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

DIXIÈME CONGRÈS
ZEHNTER KONGRESS
TENTH CONGRESS

TOKYO
SEPTEMBER 6–11, 1976

RAPPORT INTRODUCTIF
EINFÜHRUNGSBERICHT
INTRODUCTORY REPORT

PUBLIÉ PAR LE SECRÉTARIAT DE L'AIPC À ZÜRICH
HERAUSGEGEBEN VOM SEKRETARIAT DER IVBH IN ZÜRICH
PUBLISHED BY THE SECRETARIAT OF IABSE IN ZÜRICH

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International Association for Bridge and Structural Engineering

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Préface

Un congrès de l'AIPC est à la fois un événement sur le plan scientifique et technique et une possibilité de contacts que nulle lecture ou réflexion ne peut remplacer. En 1976 au Japon, du 6 au 11 septembre, il vient s'ajouter le charme d'un pays prestigieux offrant les contrastes les plus étonnants propres à obliger l'architecte et l'ingénieur à dépasser le pur cadre de son activité traditionnelle.

Selon une tradition bientôt cinquantenaire, ce présent rapport introductif présente une série d'exposés généraux donnant la base pour les contributions des participants au congrès qui paraîtront dans le rapport préliminaire. Dix-huit rapporteurs généraux (trois par thème), de réputation internationale, ont accepté de participer à ce volume. Nous les remercions très sincèrement de leur collaboration et souhaitons que leurs exposés suscitent un grand enthousiasme pour notre congrès. Conformément à notre mission, les thèmes ont été étendus aux méthodes de construction et à l'optimisation. Autre nouveauté, nous consacrons le dernier thème à des contributions de synthèse sur des sujets pour lesquels l'AIPC a joué un rôle particulièrement déterminant depuis notre dernier congrès en 1972. Ces contributions-là paraîtront seulement dans le rapport final, afin de permettre de tenir compte des ultimes développements jusqu'au moment du congrès.

Les membres qui ont l'intention de participer à la discussion préparée devront présenter avant le 31 juillet 1975 le thème, le titre et un court exposé de leur article (20 à 60 lignes dactylographiées) dans l'une des trois langues officielles de l'Association.

L'invitation du groupe japonais de l'AIPC pour notre congrès 1976 est un événement marquant dans la vie de notre société. Nous remercions vivement nos amis japonais, à la tête desquels se trouve Monsieur Inayama, de leur engagement et de leur volonté de réussite. Nous invitons nos membres à répondre massivement à tant de créativité et de gentillesse afin que notre dixième congrès contribue d'une manière déterminante à l'accomplissement de notre mission.

Zurich, janvier 1975

Le Président de l'AIPC:

Prof. MAURICE COSANDEY

Président de l'Ecole Polytechnique Fédérale de Lausanne

Les Secrétaires Généraux:

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Professeur à l'Ecole Polytechnique
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Professeur à l'Ecole Polytechnique
Fédérale de Zurich

JÖRG SCHNEIDER

Professeur à l'Ecole Polytechnique
Fédérale de Zurich

Vorwort

Ein IVBH-Kongress, wissenschaftlich-technisches Ereignis und Gelegenheit zur Anknüpfung persönlicher Kontakte zugleich, lässt sich weder durch Lektüre noch eigene Überlegungen ersetzen. Hinzu kommt vom 6. bis zum 11. September 1976 in Japan der Charme eines wunderbaren Landes mit den erstaunlichsten Gegensätzen, der geeignet ist, Architekten und Ingenieure aus ihrer normalen Tätigkeit herauszuheben.

Gemäss fast 50-jähriger Tradition bietet der vorliegende Einführungsbericht eine Reihe von allgemeinen Abhandlungen und bildet damit die Basis für die Beiträge der Kongressteilnehmer, welche im Vorbericht erscheinen werden. Achtzehn Berichterstatter von internationalem Ansehen (drei pro Thema) haben diesen Band ermöglicht. Wir verbinden mit dem Dank für ihre Mitarbeit den Wunsch, dass ihre Beiträge einen begeisterten Widerhall finden werden. Entsprechend der erweiterten Aufgabe der IVBH wurde der Themenkreis ausgedehnt auch auf die Gebiete der Bauverfahren und der Optimierung. Erstmals wird im übrigen das letzte Thema zusammenfassenden Beiträgen über Gebiete vorbehalten bleiben, in denen die IVBH seit dem letzten Kongresse 1972 eine entscheidende Rolle gespielt hat. Die entsprechenden Beiträge werden erst im Schlussbericht erscheinen, um auch die letzten Entwicklungen berücksichtigen zu können.

Mitglieder, die an der vorbereiteten Diskussion teilzunehmen beabsichtigen, sollten vor dem 31. Juli 1975 Thema, Titel und kurze Zusammenfassung (20 bis 60 Schreibmaschinenzeilen) in einer der drei offiziellen Sprachen der Vereinigung einreichen.

Die Einladung der japanischen Gruppe der IVBH für den Kongress von 1976 ist ein markantes Ereignis im Leben unserer Vereinigung. Wir danken unseren japanischen Freunden, allen voran Herrn Inayama, herzlich für ihren erfolgversprechenden Einsatz. Unsere Mitglieder sind eingeladen, dieses so ideenreiche und überaus freundliche Bemühen mit einer zahlreichen Teilnahme zu belohnen, damit der zehnte Kongress uns einen entscheidenden Schritt näher an die Erfüllung unserer Aufgaben heranzuführt.

Zürich, Januar 1975

Der Präsident der IVBH:

Prof. MAURICE COSANDEY

Präsident der Eidgenössischen Technischen Hochschule Lausanne

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Preface

An IABSE Congress is not only an event of technical and scientific importance but also offers various possibilities for making contacts which cannot be gained from merely reading or thinking about such things. The congress of 1976 will take place in Japan from 6th to 11th September. The charm of this country will inspire architects and engineers to move beyond the traditional confines of their professional activities.

In accordance with a nearly 50 year old tradition, the present introductory report contains a series of general papers serving as the basis for the contributions of the participants in the congress; these contributions will appear in the preliminary report. Eighteen general reporters of world-wide repute have stated their willingness to participate actively in the preparation of this volume (three reporters for each theme). We sincerely thank them for their collaboration and hope that their articles will evoke great interest in our congress. In accordance with our extended aims the range of themes has been broadened, now including topics on constructions methods and optimization. For the first time the last theme is dedicated to contributions on topics in which the IABSE has played an important role since the last congress in 1972. These contributions will be published in the final report only, in order to allow taking into consideration the latest developments up to the time of the congress.

Members who intend to participate in the prepared discussion should, before 31st July 1975, hand in their theme, title and a short summary of their article (20 to 60 typewritten lines), written in one of the three official languages of the association. The invitation by the Japanese group of IABSE for our congress in 1976 is a remarkable event in the history of our association. We warmly thank our Japanese friends, especially Mr. Inayama, for their promising engagement and their will for success. We invite our members to reward such creativity and kindness by active participation in the congress so that it may enable us to approach the accomplishment of our aim.

Zurich, January 1975

The President of IABSE:

Prof. MAURICE COSANDEY

President of the Swiss Federal Institute of Technology, Lausanne

The General Secretaries:

Dr.sc.techn. HANS VON GUNTEN

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RAPPORT PRÉLIMINAIRE

Le Rapport Préliminaire, à paraître en mai 1976, contiendra les articles écrits sur la base du Rapport Introductif, et acceptés par les Commissions de Travail responsables.

Les participants désirant envoyer un article pour le Rapport Préliminaire sont priés de transmettre au Secrétariat de l'AIPC avant le **31 juillet 1975**, le titre, le thème et un court exposé de leur article (20–60 lignes dactylographiées) dans l'une des trois langues officielles de l'Association.

Les auteurs dont les articles auront été retenus, enverront leur textes prêts pour la reproduction par procédé offset — en anglais, français ou allemand — au Secrétariat de l'AIPC pour le 15 janvier 1976.

VORBERICHT

Der im Mai 1976 erscheinende Vorbericht wird die aufgrund des Einführungsberichtes geschriebenen — und von den verantwortlichen Arbeitskommissionen angenommenen Beiträge — enthalten.

Die Teilnehmer, die einen Beitrag für den Vorbericht einreichen möchten, werden gebeten, dem Sekretariat der IVBH vor dem **31. Juli 1975** den Titel, das Thema und eine kurze Zusammenfassung ihres Beitrages (20–60 Schreibmaschinenzeilen) in einer der drei offiziellen Sprachen der Vereinigung zukommen zu lassen.

Die Verfasser der angenommenen Beiträge werden ihren für den Offsetdruck vorbereiteten Text auf Englisch, Französisch oder Deutsch dem Sekretariat der IVBH bis am 15. Januar 1976 zustellen.

PRELIMINARY REPORT

The Preliminary Report, to be issued in May 1976, will contain papers written on basis of the Introductory Report and accepted by the Commissions in charge.

Participants who want to send a paper for the Preliminary Report are requested to submit to the Secretariat of IABSE not later than **July 31, 1975**, the title, theme and short version of their contribution (20–60 lines of typescript) in one of the three official languages of the Association.

The authors of the papers which have been accepted, will be requested to submit their camera-ready manuscripts in English, French or German, before January 15, 1976, to the Secretariat of IABSE.

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Le thème VII fera l'objet de trois conférences, qui seront imprimées dans le Rapport Final.
Das Thema VII wird aus drei Vorträgen bestehen, die im Schlussbericht veröffentlicht werden.
Theme VII will consist of three lectures, to be printed in the Final Report.

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I

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

**Entwurfsgrundlagen und Entscheidungskriterien
für Tragwerke**

**Design Philosophy and Decision Processes
for Structures**

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Einfluss der Baumethoden auf den Entwurf von Tragwerken

Planning of Structures and its Relationship with Construction Methods

L'influence des moyens et des méthodes de construction

ANGELO POZZI

Professor für Bauplanung und Baubetrieb
ETH Zürich
Zürich, Schweiz

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

1. Einleitung

Wenn in den letzten Jahrzehnten die Architekten vor allem die ästhetischen Probleme und die Bauingenieure das Tragwerksverhalten eines Bauwerkes behandelten, so handelt es sich einfach um eine sehr starke Gewichtung von zwei der vielen Aspekte, unter denen ein Bauwerk betrachtet und bewertet werden muss. Durch die Mechanisierung der Bauwirtschaft, die wachsende Kritik der Benutzer von Bauten und die knapper werdenden finanziellen Mittel bekommen ökonomische, ökologische, technologische und zweckorientierte Aspekte ein ständig steigendes Gewicht. Nicht nur das Zielsetzen für ein Investitionsvorhaben, sondern ebenso sehr das Gestalten und Erbauen des Bauwerkes sind äusserst komplexe Vorgänge; sie sind für den Nichtfachmann immer weniger überblickbar. Er kennt normalerweise die Ausgangssituation und die Randbedingungen nicht, wertet und kritisiert lediglich das Produkt. Ob wir es wahr haben wollen oder nicht, für die Zweckdienlichkeit, die Funktionstüchtigkeit, den betriebswirtschaftlichen Erfolg oder Misserfolg, die ästhetisch ansprechende oder nichtpassende Form, den umweltfreundlichen oder umweltbelastenden Charakter eines Bauwerkes werden immer mehr die Fachleute verantwortlich gemacht. Es ist notwendig, dass wir das Bauwerk als Ganzes betrachten und das Bauen nicht nur unter ganz wenigen stark gewichteten Aspekten betrachten. Alle Aspekte sollen mit in die Betrachtung einbezogen werden und je nach der Zielsetzung für ein Bauvorhaben werden dann bestimmte Kriterien entsprechendes Gewicht erhalten. In der Folge wird der Investitionsprozess als Ganzes dargestellt und kritisch betrachtet. Auf diesem Hintergrund wird dann das Teilgebiet des Einflusses der Baumethoden auf den Entwurf der Tragwerke kurz behandelt und die Hauptprobleme für die Diskussion am Kongress in Tokio formuliert.

Für bestimmte vereinfachte Prozesse, Informationssysteme oder Entscheidungsvorgänge lassen sich selbstverständlich mathematische Modelle konstruieren und entsprechende Lösungsmethoden entwickeln. Mit diesem Bericht sollen Diskussionsbeiträge provoziert werden, die sich mit den Elementen des Problemlösungsprozesses auseinandersetzen. Von besonderem Interesse sind Beiträge, die am praktischen Beispiel die Entwicklung der Entscheidungskriterien im Zusammenhang mit der Bewertung von alternativen Konzepten zur Darstellung bringen. Es geht hier also nicht um die Darstellung von ablaufplanerischen Lösungsmethoden, sondern um die einfache Darstellung von wesentlichen Zusammenhängen im Bauprozess. Als Anwendungsgebiet soll der Entwurf und die Ausführung von Tragwerken im Vordergrund stehen.

2. Die Besonderheiten des Bauwesens

Die Bauwirtschaft unterscheidet sich nach Struktur, Arbeitsweise und wirtschaftlicher Konzeption stark von andern Wirtschaftszweigen. Die serienmässige Herstellung von Bauwerken für den Markt und die damit verbundene Werbung zur Absatzsicherung und Markteinflussung fehlen praktisch. Der rationelle Einsatz von Mensch, Maschine und Kapital aber, die Auswertung von wissenschaftlichen Methoden beim Entwerfen und Ausführen von Bauten sowie die detaillierte organisatorische Bearbeitung der komplexen Prozesse sind Merkmale der stationären Industrie, die auch in der Bauwirtschaft Gültigkeit haben. Jede Baustelle gleicht infolge der Mechanisierung einer am Bauort eingerichteten Fabrik, wobei der temporäre Charakter in der Mobilität der technischen Hilfsmittel zum Ausdruck kommt. Man könnte die Bauwirtschaft als den Wirtschaftszweig der "wandernden Fabriken" bezeichnen.

Die stationäre Industrie arbeitet nach einem weitgehend selbst gestalteten Produktionsprogramm; Arbeitsablauf und betriebliche Dispositionen richten sich über längere Zeit danach. Die Baubetriebe und das Baugewerbe haben noch wenig Möglichkeiten, die Produktion nach eigenem Ermessen zu programmieren. Die Produktion wird im wesentlichen durch die Investitionsfreudigkeit der privaten und öffentlichen Bauherren bestimmt. Sie bestimmen mit Hilfe der Dienstleistungsbetriebe des Bauwesens die technischen Daten der Bauobjekte. Erst nach Abschluss dieser Vorbereitungsarbeiten werden normalerweise die Bauproduzenten zugezogen. Nach der Offertstellung und Vergabe können die dann beauftragten Unternehmer ihre Dispositionen treffen. Es handelt sich in der Bauwirtschaft nicht um eine programmierte Produktion, sondern um eine Auftragsproduktion. Die Bauleistung ist kein kontinuierlicher Vorgang, sondern eine Summe in sich abgeschlossener Einzelvorgänge.

Die komplexer werdenden Baubedürfnisse und die zunehmende Zahl der zu erfüllenden Randbedingungen bedingen einen ständig grösser werdenden Zeitraum für die Bedürfnisabklärung und den Entwurf von Bauwerken. Dieser Zeitraum beträgt in einfachen Fällen wenige Jahre,

in komplexeren Fällen aber bis zu zehn und mehr Jahren. Aehnliche Zeiträume werden oft für die Ausführung benötigt. Diese Zeiträume sind so gross, dass sich die Bedürfnisse der Investoren und Benützer während diesen Phasen ändern können und deshalb im Entwurf mitberücksichtigt werden müssen. Beispiele dafür sind heute unvollendete städtische Expressstrassen, die infolge der raschen Aenderung der Einstellung der Betroffenen vorläufig nicht zu Ende gebaut werden. Mit Bauen ist immer ein Eingriff in die Umwelt verbunden, denn Bauen bedeutet ja letztendlich auch Gestalten der Umwelt. In den dichter besiedelten Gebieten sind diese Eingriffe besser sichtbar und spürbar. Die Zweckbestimmung und Nutzungsart, sowie volkswirtschaftliche und betriebswirtschaftliche Ueberlegungen ergeben für Bauwerke im Durchschnitt eine mehrere Generationen umfassende Nutzungs- und Lebensdauer. Der Zeitraum unserer Betrachtungen ist im Bezug zur Lebensdauer eines Bauwerkes klein, man darf deshalb davon ausgehen, dass Bauen für praktische Ueberlegungen ein irreversibler Prozess darstellt. Deshalb dürfen unsere Bauwerke nicht nur dem momentanen Bedürfnis entsprechen, sondern das bauliche Konzept muss mögliche künftige Nutzungsänderungen im vernünftigen Rahmen mitberücksichtigen. Die aus ökonomischen Gründen notwendige lange Lebensdauer der Bauwerke zwingt uns, auch bauliche Konzepte zu entwickeln, die nicht nur dem momentanen Bedürfnis der geplanten Erstnutzung entsprechen, sondern mögliche künftige Nutzungsänderungen mitberücksichtigt.

Auch wenn immer mehr Einzelteile eines Bauwerkes vorgefertigt werden und sich damit eine gewisse Standardisierung der Teilprodukte ergibt, bleiben Bauwerke an Ort gefertigte Einzelprodukte, im weiteren Sinne also Prototypen. Die ständig ändernden individuellen Bedürfnisse der Investoren und Benützer, die einmalige geographische Lage und Umwelt eines Bauwerkes, die immer wieder neue kreative Leistung der Gestalter des Bauwerkes, die wechselnde Zusammensetzung der Benützer des Bauwerkes und die zufälligen Bedingungen bei der Planung und Ausführung der Bauwerke sind genügend Begründung für die vorher aufgestellte Behauptung. Wir können deshalb davon ausgehen, dass für jeden Einzelfall eine spezifische Organisation geschaffen werden muss, die den Anforderungen in der Vorbereitung und Ausführung eines Bauvorhabens gewachsen sein soll. Spätestens mit der Inbetriebnahme eines Bauwerkes wird diese projektorientierte Organisation wieder aufgelöst.

Das Problem der Beschaffung und Bearbeitung der für das Projekt relevanten Information stellt sich bei jedem Baubedürfnis wieder neu. Es geht aber nicht nur um die Informationsbeschaffung selbst, sondern die richtigen Informationen müssen zur rechten Zeit am rechten Ort in der notwendigen Menge vorliegen, damit ein fachgerechter Entscheid durch die kompetenten Instanzen gefällt werden kann. Spezielle Probleme ergeben sich deshalb, weil die wesentlichsten Entscheide über ein Bauvorhaben zu einem Zeitpunkt gefällt werden müssen, da noch wenige Informationen vorliegen. Entscheiden bei Unsicherheit kompliziert den Bauablauf enorm, macht aber das Entwerfen und Ausführen von Bauwerken erst zur interessanten Aufgabe.

Das Entwerfen, Planen und Ausführen von Bauwerken stellt also nicht nur von der gestalterischen Seite hohe Ansprüche, sondern stellt zudem einen der komplexesten praktischen Prozesse dar. Es ist deshalb gerade hier notwendig, das Konzept des Projektmanagement zu vertiefen, damit die gestalterische Arbeit besser zum Ausdruck kommt. Es geht also nicht um das "Entweder oder", sondern um das "Sowohl als auch". Es wird immer wieder versucht, die betriebswissenschaftlichen Erkenntnisse der stationären Industrie direkt auf den Bauprozess zu übertragen. Wenn wir nach neuen besseren Wegen suchen, müssen für die Lösung unserer komplexen Probleme die vorstehend beschriebenen Besonderheiten mitberücksichtigt werden.

3. Der Bauprozess

Den Bauprozess kann man in die fünf Hauptphasen

Generalplanung
Objektvorbereitung
Objektausführung
Objektnutzung
Objektliquidation

gliedern. Sie sind natürlich nicht unabhängig voneinander, sondern durch Rückkopplungen verschiedener Art miteinander verbunden. Diese fünf Hauptphasen sind je durch eine vorwiegende Tätigkeit charakterisiert.

In der Phase Generalplanung geht es vorwiegend um die Bedürfnisabklärung, das Einordnen in allfällige Gesamtpläne, die Ueberprüfung der "Feasability" des Vorhabens und die Erarbeitung der Zielsetzung für das zu entwerfende Bauwerk. Basierend auf den durch die Generalplanung erarbeiteten Grundlagen werden in der Phase der Objektvorbereitung die Vorstellungen konkretisiert. Aus verschiedenen möglichen baulichen Konzepten wird eine Lösung gewählt und als Projekt soweit bearbeitet, dass die wesentlichen Grundlagen für die Ausführung vorliegen. Eine verfeinerte Studie über die "Feasability" des Projektes ist hier angezeigt, denn mit dem Beginn der nächsten Phase ist man praktisch bereits am "Point of no return" angelangt.

Die Phase der Objektausführung basiert auf einem Plan, der den Ablauf der Tätigkeiten vorgibt. Er enthält die zeitliche Folge der Tätigkeiten, die abhängigen finanziellen Konsequenzen und legt die notwendigen Kapazitäten fest. In der Wirklichkeit werden dann kleinere oder grössere Abweichungen vom Plan auftreten, die durch unkontrollierbare, nicht voraussehbare Einflüsse oder nicht zutreffende Annahmen entstehen. Je intensiver man sich bei der Objektvorbereitung mit den möglichen Entwicklungen in der Phase der Objektausführung befasst, desto weniger wird man durch Abweichungen vom Plan überrascht. Prototypen werden normalerweise einem Testlauf unterzogen, bevor sie für den Betrieb freigegeben werden. Dieser Probelauf muss für Bauwerke in den meisten Fällen in die Phase der Objektnutzung miteinbezogen werden.

Die ersten Jahre der Nutzung einer baulichen Anlage sind also zugleich Testzeit für das Produkt. Die notwendigen Korrektur- und Garantiearbeiten müssen deshalb während der Betriebszeit vorgenommen werden. Je nach der Zweckbestimmung der baulichen Anlage werden in der Phase der Objektnutzung mehr oder weniger Anpassungs-, Abänderungs- oder Ergänzungsarbeiten zusätzlich zu den normalen Unterhaltsarbeiten anfallen. Je besser in der Phase der Generalplanung oder Objektvorbereitung die in Zukunft ändernde Nutzungsart mitberücksichtigt werden kann, desto einfacher gestalten sich diese Arbeiten. Man wird sich oft schon beim Entwurf einer baulichen Anlage auch Gedanken über die Liquidation machen müssen, denn in vielen Fällen können hohe Kosten und schwierige Probleme damit verbunden sein.

Die vielen an einem Bauprozess beteiligten Organisationen können in einige Hauptgruppen zusammengefasst werden, dabei ist für die Gruppierung die Aufteilung der Verantwortung wegleitend.

Verantwortung Gruppe Bauherr:

- Zielsetzung für die Hauptphasen des Bauprozesses
- Treffen der entsprechenden Hauptentscheide
- Koordination der Investitionstätigkeit
- Aufbau und Auflösung der Projektorganisation

Verantwortung Gruppe Baufachorgane:

- Erarbeiten der Entscheidungsgrundlagen für die Wahl unter alternativen Konzepten und Projekten
- Entwicklung und Erarbeitung technisch-wirtschaftlich optimaler Lösungen
- Realisierung der Bauprojekte

Verantwortung Gruppe Benützer:

- Erarbeitung der betrieblichen Konzepte
- Betrieb und Verwaltung der baulichen Anlagen
- Anpassung und Unterhalt im Rahmen der eigenen technischen Möglichkeiten

Verantwortung Gruppe Staat:

- Durchsetzen allgemein gültiger Vorschriften und Normen
- Erhalten des ökologischen Gleichgewichtes

PHASEN AUFGABEN	BETEILIGTE		FACHORGANE		BENÜTZER	STAAT
	BAUHERR	DIENST - LEISTUNG	AUS - FÜHRENDE			
GENERALPLANUNG	3	2	1	1	1	
OBJEKT VORBEREITUNG	2	3	2	1	1	
OBJEKTAUSFÜHRUNG	1	2	3	1	1	
OBJEKT NUTZUNG	1	1	1	3	1	
OBJEKT LIQUIDATION	1	1	3	1	1	

Bild 1: Gewicht der Mitarbeit im Bauprozess durch verschiedene Baubeteiligte

AUFGABEN		BETEILIGTE										
		BAUHERR	PROJEKTL.	B ₃	B ₄	B ₅						
GESAMT		E	OL			•						
UNTER -	A1											
	A2											
	A3											
TEIL -	A4											
	A5											
UM -												

KOMPETENZEN:
 OPERATIVE LEITUNG
 ENTSCHEIDUNG
 MITSPRACHE
 INFORMATIONSRECHT
 etc.

Bild 2: Funktionen Diagramm

Eine Unterteilung der Gruppe Fachorgane in die Untergruppen Dienstleistung und Produktion ist durch die praktische Situation und die Problemstellung gegeben, obwohl spezifische Unternehmungen auch beide Bereiche decken. In Bild 1 ist der Zusammenhang zwischen den Baubeteiligten und den Hauptphasen des Bauprozesses dargestellt. Die Zahlen geben die Intensität des Einflusses einer Gruppe in einer Phase wieder. Wenn nun die Hauptphasen in sich weiter unterteilt werden, so erhält man in der linken Kolonne des Bildes 1 die zu bearbeitenden Hauptaufgaben des Bauprozesses. Zerlegt man zudem die Gruppen in der ersten Zeile von Bild 1 in die verschiedenen einzelnen am Bauprozess beteiligten Betriebe, Unternehmungen, Funktionäre und Lieferanten, so ergibt sich eine Aufgaben-Beteiligten-Matrix. Die Zuordnung bestimmter Kompetenzen bei der Behandlung einer spezifischen Aufgabe an einen Beteiligten wird im Funktionsdiagramm festgelegt. Dabei ist es oft sinnvoll, die Gesamtaufgabe für diesen Zweck nach andern Gesichtspunkten zu gliedern als nach dem Aspekt der logischen Folge. Dieses Funktionsdiagramm (Bild 2) bildet die Basis jeder weiteren organisatorischen Tätigkeit. Darauf aufbauend lassen sich erste Pflichtenhefte für die Hauptbeteiligten ableiten und ein Konzept für den Informationsfluss erstellen.

Im Rahmen dieser Zuordnung werden die Beteiligten immer wieder unabhängig von der spezifischen Aufgabe drei Grundtätigkeiten erfüllen:

- Informationen beschaffen und aufarbeiten
- erkannte Probleme lösen
- Entscheidungen treffen.

Ueber diese Grundtätigkeiten sind die Beteiligten miteinander und die Aufgaben untereinander gekoppelt. Bevor nicht bestimmte Informationen vorliegen, können bestimmte Probleme nicht gelöst werden und entsprechende Entscheide gefällt werden. Vielfach werden in einem späteren Zeitpunkt zur Verfügung stehende Informationen einen früher gefällten Entscheid in Frage stellen. Oft entstehen spezifische Probleme erst durch zusätzliche Informationen. Der allgemeine Informationsstand ist in der ersten Hauptphase tief, trotzdem werden die wesentlichsten Entscheide gerade in den ersten beiden Phasen gefällt (Bild 3). Man ist bestrebt, mit möglichst kleinem Aufwand einen möglichst guten Entscheid zu treffen. Es stehen auch nur begrenzte Mittel für die Vorbereitung der einzelnen Entscheide zur Verfügung. In den späteren Phasen nimmt der Einfluss der Entscheidungen bezogen auf das ganze Bauwerk stark ab, wobei der Informationsstand der Beteiligten aber ständig wächst.

Versucht man nun im konkreten Fall das Lösen der anfallenden Probleme, das Aufarbeiten der relevanten Informationen, die zu fällenden Entscheidungen unter Berücksichtigung der immer vorhandenen Rückkopplungen in eine logische zeitliche Folge zu bringen, dann ergibt sich ein sehr komplexer dynamischer Ablauf, eben der Bauprozess.

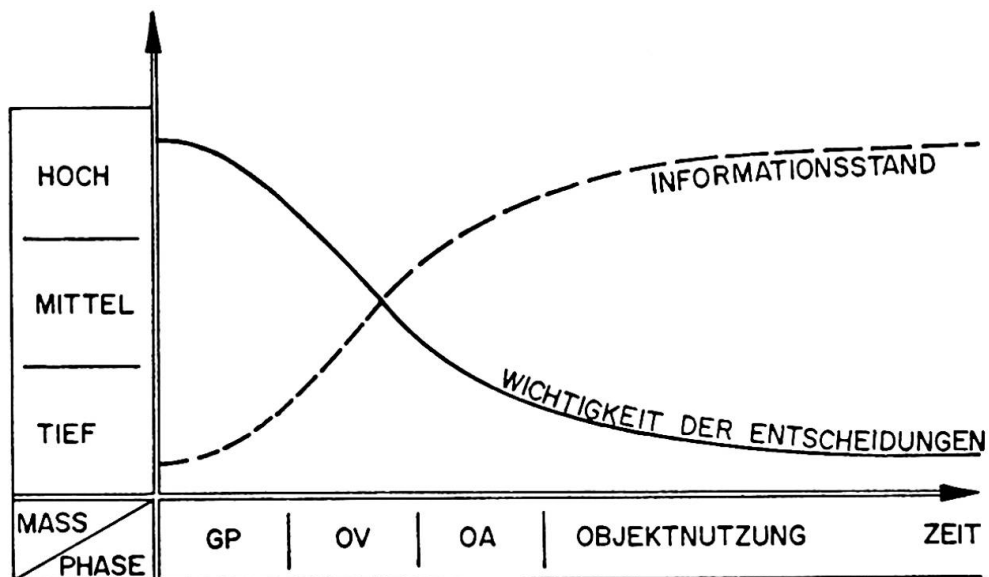


Bild 3 : Informationsstand in Relation zur Wichtigkeit der Entscheidung in Funktion der Hauptphasen des Bauprozesses

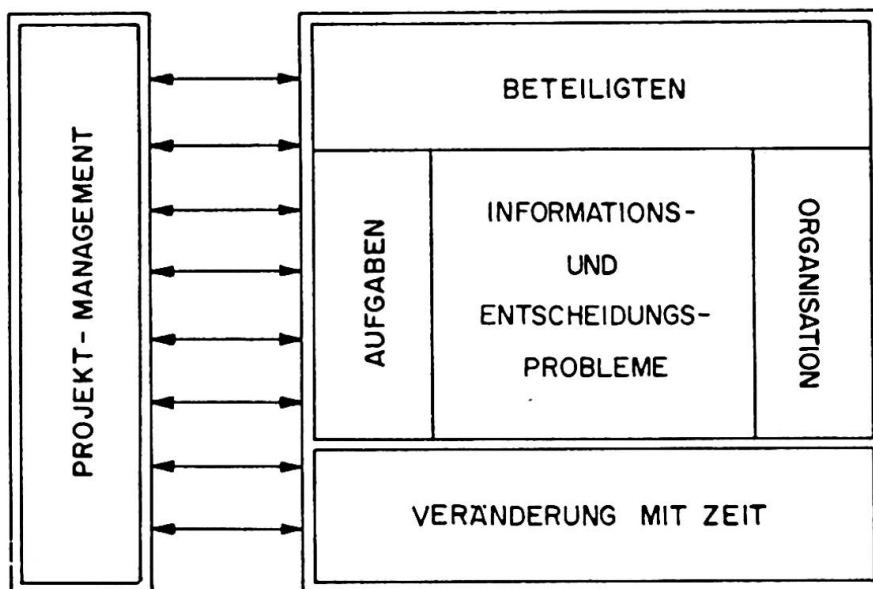


Bild 4 : Die Elemente des Projektmanagementproblems

Obwohl jedes Bauwerk für sich einen Prototypen darstellt, wird es mit der Zeit möglich sein, für bestimmte Gruppen von immer wieder ähnlichen Bauaufgaben den Bauprozess so zu standardisieren, dass damit der Praxis ein echtes Hilfsmittel zur Verfügung gestellt werden kann. Da allerdings auch bei ähnlichen Bauwerken die Baubeteiligten immer wieder ändern, entsteht hier zusätzlich ein Ausbildungsproblem. Denn solche Hilfsmittel sind nur dann sinnvoll zu gebrauchen, wenn die Beteiligten die gleiche Sprache sprechen und das verwendete Hilfsmittel verstehen.

Durch die Netzplantechnik gefördert, läuft man heute Gefahr, den Bauprozess vorwiegend als logische Kette von Tätigkeiten oder Ereignissen zu sehen. Sicher gewinnt man mit dieser Darstellung eine bessere Uebersicht über den Ablauf. Schwierigere und schwerwiegendere Probleme ergeben sich durch die vielen Entscheidungssituationen; in welcher Zeit, unter welchen Aspekten, auf welchen Planungshorizont bezogen soll eine Entscheidungssituation bearbeitet werden? Fortschritte bei der Systematisierung des Bauprozesses müssen vor allem in dieser Richtung erzielt werden. Die vielen meist voneinander unabhängigen Baubeteiligten, die vielen immer wieder unter neuen Aspekten zu bearbeitenden Aufgaben, die über die verschiedenen Phasen des Bauprozesses ständig anzupassende temporäre Organisation, der deshalb notwendigerweise komplizierte Informations- und Entscheidungsmechanismus sind die Begründung dafür, dass gerade hier ein hochqualifiziertes Management notwendig ist. Hier sind noch wesentliche Fortschritte relativ rasch möglich. Das Modell des Projektmanagement muss für unsere Bedürfnisse weiterentwickelt werden. (Bild 4)

4. Der Problemlösungsprozess

Das Problemlösen ist eine der Grundtätigkeiten im Bauprozess. Das Vorgehen beim Lösen irgendeines Problems ist immer dasselbe. Nur haben die verschiedenen Teilschritte je nach der Art des Problems eine verschiedene Bedeutung. Im Bild 5 ist ein Modell für das Vorgehen beim Problemlösen dargestellt. Wir gehen davon aus, dass der Entscheidungsträger nicht identisch mit der Sachbearbeitungsgruppe ist. Das Modell lässt sich dann in zwei Hauptteile gliedern. Im ersten Teil geht es darum, dass die Sachbearbeiter über eine umfassende Analyse des Auftrages das Problem im Sinne des Auftraggebers verstehen und auf dieser Grundlage die Zielsetzung derart verfeinern, dass ein detaillierter Kriterienplan entsteht. Eine weitere Bearbeitung des Problems erübrigt sich so lange, als der Auftraggeber und die Sachbearbeiter sich über diese erste Fassung des Kriterienplanes nicht einigen. Es ist nicht sinnvoll, eine Vielzahl von Alternativen zu entwickeln und zu bewerten, wenn man über die massgebenden Kriterien für die Bewertung nicht einig ist. Hat man sich aber geeinigt, dann sind die Grundlagen für die eigentliche kreative Arbeit gegeben. Der Entwicklung der wesentlichen Lösungsalternativen folgt deren detaillierte Analyse. Die aufgrund der vereinbarten Kriterien vorgenommene Bewertung der Alternativen er-

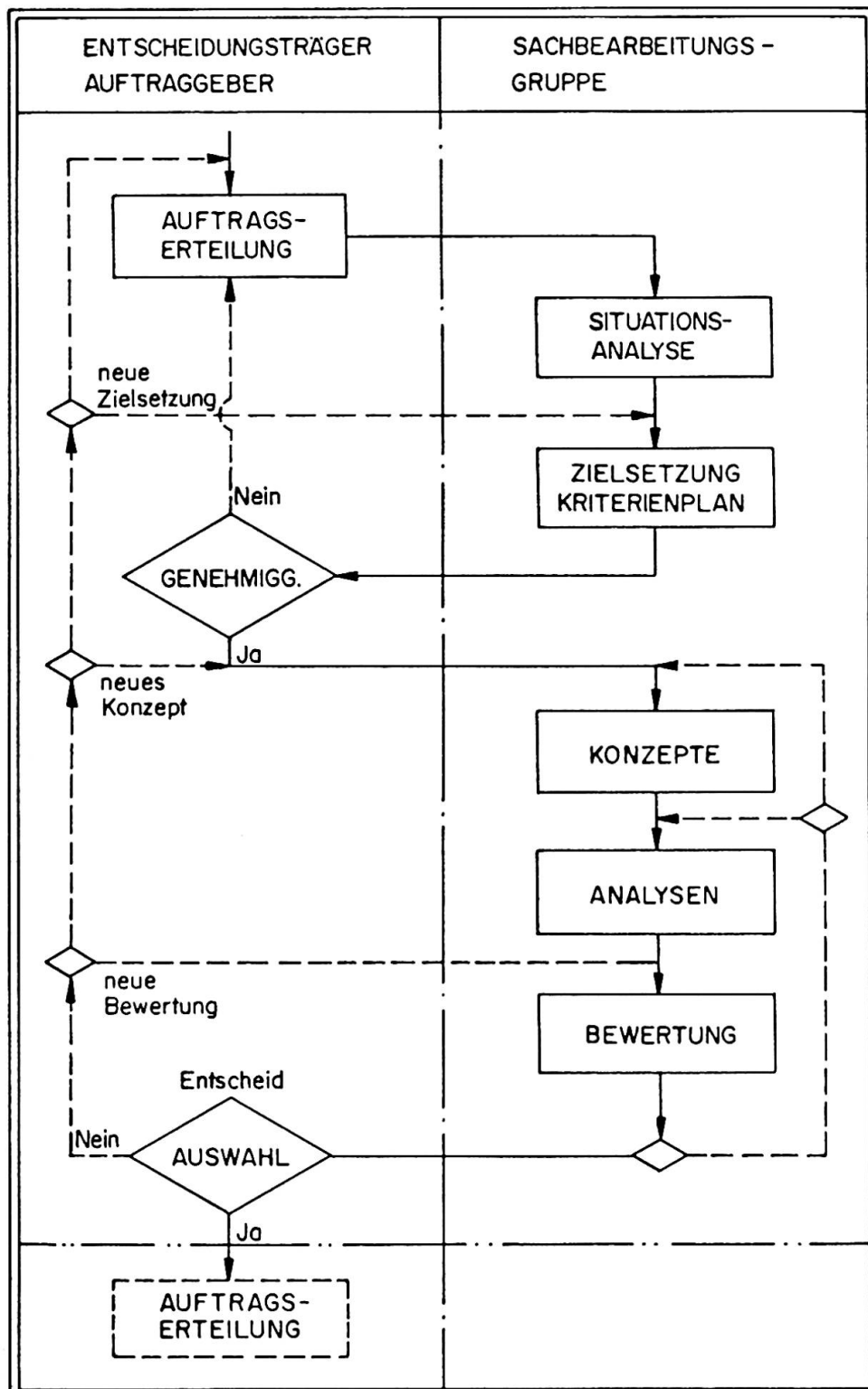


Bild 5: Problemlösungsvorgang

gibt die Entscheidungsgrundlagen für den Entscheidungsträger.

Dieses ideale Vorgehen kann in der Praxis meist nicht realisiert werden. Erst durch die Bearbeitung der Alternativen selbst erhält man oft genügend Einsicht in das Problem, um die massgebenden Kriterien zu verstehen. Auch das Entwerfen von Alternativen kann nicht ganz von der analytischen Tätigkeit getrennt werden, denn oft sieht man wesentliche Alternativen erst nach einer intensiven Analyse verwandter Konzepte. Rückkopplungen aller Art müssen also im Modell vorhanden sein. Ein Problemlösungsprozess ist dann abgeschlossen, wenn entweder der Auftrag aufgrund der Ergebnisse aufgehoben wird, oder eine Wahl für eine Alternative getroffen wurde. Wesentlich ist, dass zur rechten Zeit mit den richtigen Beteiligten eine Einigung über die anzuwendenden Kriterien herbeigeführt wird und damit klare Grundlagen für den Entwurf und die Bewertung geschaffen werden. Die Wahl ist ein Entscheid bei Unsicherheit, damit ist aber auch bei eindeutigen Vorteilen für eine Alternative noch nicht sicher, ob sich diese Wahl in der Zukunft als gut erweist.

5. Das Tragwerk als Element des Bauwerkes

Wir haben uns bisher mit dem Bauwerk und dem Bauprozess als Ganzem beschäftigt. Jene Baubeteiligten, die sich mit dem Gestalten, Entwerfen, Konstruieren, Berechnen, Produzieren, Erbauen oder Montieren des Tragwerkes auseinandersetzen, sind wie alle andern verpflichtet, ihre Aufgabe als Teilaufgabe im Gesamtprozess zu sehen und zu verstehen. Die Betrachtungen in den vorstehenden Abschnitten können deshalb ohne weiteres auf den Problembereich Tragwerk übertragen werden. Die Bedeutung der Teilaufgabe "Tragwerk" innerhalb eines Bauprozesses ist je nach Bauwerksgruppe sehr verschieden.

Handelt es sich um die Gruppe der Brückenbauten, Turmbauten, Staumauern, Off-Shore-Tragwerke, (Gruppe A) wird die Teilaufgabe "Tragwerk" praktisch zur Hauptaufgabe. In dieser Gruppe ist die Hauptfunktion des Bauwerkes eine Tragwerksfunktion. Trotzdem dürfen diese Bauwerke nicht isoliert für sich betrachtet werden, denn sie sind ja Teil eines übergeordneten Systems. Liegt dieses übergeordnete Konzept einmal fest, dann lassen sich bei dieser Gruppe einzelne Aufgaben oft so genau abgrenzen, dass sie für sich einen abgeschlossenen Bauprozess darstellen. Dieser Prozess muss lediglich auf die Zeitplanung im übergeordneten System abgestimmt werden.

Bei der Gruppe der Industriebauten, Verwaltungsbauten, Wohnbauten (Gruppe B) stellt die Aufgabe "Tragwerk" eine der wichtigeren Teilaufgaben dar. Das Funktionieren der Installationen, die Zweckmässigkeit innerbetrieblicher Dispositionen, das Erfüllen der bauphysikalischen Forderungen, das Erfüllen der ästhetischen Bedingungen als Bauwerk sind für die Funktionstüchtigkeit des Bau-

werkes ähnlich wichtige Aspekte wie das gute Tragwerksverhalten unter Gebrauchslast. Es gibt hier keine Teilaufgabe mehr, die den Rang der Hauptaufgabe übernehmen könnte und nach der sich dann alle andern Aufgaben praktisch zu richten hätten. Vielmehr hängen die Problemlösungen für die einzelnen Aufgaben stark voneinander ab. Es wird ein grosses Mass von gegenseitigem Verständnis verlangt, wenn alle Teilaufgaben im Sinne der Gesamtaufgabe gelöst werden sollen. In der Gruppe Kanalbauten, Leitungsbauten, Pistenbauten, Strassenbauten (Gruppe C) spielt die Aufgabe "Tragwerk" eine untergeordnete Rolle.

In den letzten 50 Jahren hat man im Entwerfen, Berechnen, Konstruieren und Erstellen von Tragwerken in allen Gruppen grosse Fortschritte erzielt. Der "Know how" für das Lösen dieser Aufgabe ist auf einem hohen Stand. Betrachtet man die Gesamtaufgabe oder das übergeordnete System, dann stellt man rasch fest, dass der "Know how" in bezug auf das Lösen der vielen Teilaufgaben sehr verschieden ist. Eine allen Ansprüchen gerecht werdende Brücke nützt wenig, wenn sie zu einem unzweckmässigen Strassenzug gehört. Eine hochentwickelte Tragwerkskonstruktion in einem Bürohochhaus nützt wenig, wenn die Aussenhaut die bauphysikalischen Bedingungen nicht erfüllt oder die Erschliessung des Gebäudes nicht funktioniert. Wir müssen vermehrt das Tragwerk als Teil einer Gesamtaufgabe auffassen, oder uns überhaupt intensiver mit der Gesamtaufgabe auseinandersetzen, bevor Teilaufgaben gelöst werden.

Wir haben bisher das Kernproblem, den Einfluss der Baumethoden auf den Entwurf von Tragwerken nicht besonders behandelt. Auf dem Hintergrund der vorstehenden Ueberlegungen kann man dieses Problem leicht einordnen. Im Rahmen des Bauprozesses geht es darum, wie stark der Einfluss der ausführenden Fachorgane in der Phase der Objektvorbereitung sein soll (Bild 1). Im Problemlösungsprozess für die Teilaufgabe "Tragwerk" stellt sich die Frage ob baumethodische Kriterien in der Zielsetzung aufgenommen werden oder nicht. Einen Einfluss haben diese baumethodischen Aspekte auf den Entwurf des Tragwerkes dann (Bild 5), wenn die entsprechenden Kriterien mit einem starken Gewicht in die Bewertung der Alternativen eingehen. Für Bauwerke der Gruppe A (Brücken etc.) wird das meistens der Fall sein. Bei Bauwerken der Gruppe B (Wohnbauten etc.) werden bei den baumethodischen Betrachtungen nicht nur die Aufgabe "Tragwerk", sondern auch mehrere andere Aufgaben wie "Installationen", "Aussenhaut" etc. eine Rolle spielen. Bei den Bauwerken der Gruppe C (Strassen etc.) stehen wohl die Baumethoden im Vordergrund, aber da das Tragwerk eine untergeordnete Rolle spielt, ist der Zusammenhang auf Stufe Bauwerk und nicht Tragwerk zu suchen. Es geht eigentlich weniger um den Einfluss der Baumethoden auf den Entwurf eines Tragwerkes, sondern allgemein um die Frage, wie stark sollen baumethodische Aspekte als Kriterien in der Zielsetzung für die ganze Phase der Objektvorbereitung miteinbezogen werden? Dies ist aber eine Ermessensfrage, und kann nur am konkreten Beispiel beantwortet werden.

6. Die Hauptprobleme

Im Zentrum steht das Bauwerk, das Entwerfen und Ausführen des Tragwerkes ist eine Teilaufgabe, auch wenn in einigen Bauwerksgruppen das Tragwerk eine besondere Stellung einnimmt. Diese Teilaufgabe muss im übergeordneten System und im Gesamtbauprozess integriert verstanden und bearbeitet werden.

Der Bauprozess ist ein äusserst komplexer Vorgang. Wenn wir wesentliche Fortschritte erzielen wollen, müssen wir mehr über diesen Prozess wissen, Ursache - Wirkung - Beziehungen ergründen und in theoretische Ansätze umlegen, Ziel - Mittel - Beziehungen soweit entwickeln, bis brauchbare Technologien nicht nur für die Ausführung sondern auch für die Planung zur Verfügung stehen.

Unsere wesentlichen Entscheidungen werden bei relativ tiefem Informationsstand gefällt. Umsomehr sind wir auf Methoden angewiesen, mit denen die wichtigsten Lösungsalternativen gefunden und bewertet werden können.

Die Beiträge sollen sich auf diese drei Hauptprobleme beziehen. Es werden vor allem Erfahrungen in einfacher Form für das Anwendungsgebiet Entwurf und Ausführung von Tragwerken erwartet. Bevorzugt werden Beispiele, bei denen ein Kriterienplan vorgegeben war und eine Bewertung der Alternativen vorgenommen wurde. Der Vergleich der Ueberlegungen, die zu einem spezifischen Entscheid führten, mit den Ueberlegungen, die am ausgeführten Bauwerk gemacht werden, sind von besonderem Interesse.

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8. Zusammenfassung / Summary / Résumé

Die Besonderheiten des Bauwesens werden im Vergleich zu anderen Wirtschaftszweigen herausgearbeitet. Der Bauprozess wird als komplexer Vorgang in einer Aufgaben-Beteiligten Matrix dargestellt, wobei das Informations Erarbeiten, Problemlösen und Entscheiden als die drei wesentlichen Grundtätigkeiten herausgestellt werden. Das Problemlösen wird in einem weiteren Schritt in die Grundschritte zerlegt. Die Problematik des Entscheidens bei Unsicherheit wird in diesem Rahmen beschrieben. Das Entwerfen und Ausführen von Tragwerken wird als eine Teilaufgabe in den Bauprozess eingegliedert, wobei auf die Unterschiede für verschiedene Bauwerksgruppen eingegangen wird. Die am Kongress zu behandelnden Hauptprobleme werden am Schluss beschrieben.

The particularities of the problems in civil engineering and the building industries are compared to those of other economic fields. The process governing investments in buildings and civil works is presented in a "problem - concerned - parties - matrix"; to collect data, to solve problems and to make decisions are the three basic activities in this process. In a second step the elements of the activity "to solve a problem" are described, the question of decision under uncertainty will be discussed in this context. The design and realization of structures is shown as one task in a very complex process. The most interesting problems to be discussed at the Congress in Tokyo are listed at the end.

Les caractéristiques des problèmes de génie civil et de l'industrie de la construction sont comparées à celles d'autres domaines économiques. Le processus conduisant au choix d'investissements dans les constructions et les travaux publics est présenté selon une matrice "devoirs - personnes concernées", les trois activités essentielles étant l'analyse des données, la résolution de problèmes et la prise de décisions. Dans une deuxième étape, les éléments concernant la résolution de problèmes sont analysés. La question de prises de décision lors d'incertitudes est présentée dans ce cadre. Le projet et l'exécution d'une structure représente une seule activité dans le processus de la construction, cette activité étant différenciée selon les groupes d'ouvrages. La liste des problèmes-clés à discuter lors du Congrès de Tokyo est donnée en fin d'article.

Achievement of Safety and Economy in Design and Construction

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

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

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Introduction

Over the years much literature has appeared on the subject of "Structural Safety". A first tangible result of this was the presentation of the semi-probabilistic concept of safety which has been adopted by the CEB, the FIP, the CECM and the ISO.

The primary advantage of this semi-probabilistic concept of safety was its appeal to structural engineers and designers; it opened their eyes to the non-deterministic character of safety. They were accustomed to thinking in absolute terms: allowable stresses, code load requirements and factors of safety based on allowable stresses. Nevertheless most of the literature on safety remained a closed book to them. This literature was usually too mathematical and theoretical for the practicing engineer. The Model Code of the CEB opened up the prospect of a statistical approach to safety to them, it widened their horizon. Most building structures are designed by these practicing engineers with their rather conventional approach. Since the results of scientific research during preceding years has been incorporated in each new edition of Guides of Practice and Building Codes, these documents provide this large category of engineers with the strongest stimulus for selfstudy and development. The most obvious result of this semi-probabilistic concept of safety is in the practice of testing of materials, especially concrete, where quality is not defined as a mean value anymore, but as a characteristic value.

This is about as far as we have come with Guides of Practice. Finally the margin between characteristic strength and characteristic load is established as the product of some factors, but the relation with the probability of failure, let alone with economy, is not only unclear, this relation hardly exists. There are many problems here.

The symposia of the IABSE, the joint committee on structural safety CEB-CECM-CIB-FIP-IABSE, the joint committee on Tall Buildings, all the work done by several CEB-committees and research at various universities supply more and more material and show interesting developments. But all of this does not reach the great majority of the practicing engineers. The rapid development of scientific research causes a wide gap between researcher and practitioner.

The author of this Introductory Report belongs to the second category: for 20 years he was a structural engineer before he accepted a teaching position at the Delft University of Technology where he is trying to give architectural students some insight into the relation load-resistance-economy of building structures.

In that position you have to philosophize about the unsafeness of the world we live in with its tremendous "natural" catastrophes at the one hand and at the other those calamities we bring upon each other and ourselves, senseless destruction in the "solving" of controversies, unsafety due to crime, to traffic, to evergrowing pollution which so often accompanies prosperity in many countries.

How much can and will a community spend to give its members the feeling that their built environment is a safe place to live, work, recreate, etc.?

The line where safety is considered to become excessive is partly determined by the economy but also by politics.

Can we make this line clear?

The subject of this Session is: "Achievement of safety and economy in design and construction".

Our investigation of this subject should eventually lead to guides of practice and building codes. Safety cannot be tied to a number, in the way allowable stresses, factors of safety or loadfactors can. Only a probabilistic approach can give some real insight. But even then only a relative insight, in a way that "this is safer than that". If we can link safety to economy this might enable us to talk about safety in absolute terms. Is not the price of a structure partly determined by its measure of safety, but also — in the opposite direction — by the insurance premium?

The normal built environment

Now I propose that at this Session we concentrate on the safety and economy of "normal" structures. We shall not concern ourselves with prestige objects — economy and status contradict each other. Nor with the economy of special objects such as large bridges, tunnels, off-shore constructions, pipelines, etc. These projects are big enough to let specialists advise on the safety and economy of each individual object; a general code hardly applies here.

We shall concern ourselves with the economy of that category of structures — by far the biggest in volume — that constitutes the normal built environment of man, i.e. houses, schools, office-buildings, factories, nursing-homes, simple civil engineering constructions, etc.

Can we formulate rules that guarantee an optimum combination of safety and economy in these normal structures?

Before answering this question I should like to submit the following general observations:

1. Safety — probability of failure

Safety has to do with the probability of the occurrence of an event. This can be the probability of unacceptable cracks or deformations, of a fire of certain intensity, of excessive loads, of corrosion, etc. Here we confine ourselves to a concept of safety that is based on the probability of failure of an element of a structure and the consequences of this failure.

2. Loads

For the design of structures a lot of information on loads must be gathered.

a. Dead loads and other sustained loads

If we have already chosen the materials and made a preliminary calculation and also have decided on non-structural elements and finishes, then these loads are fairly accurately defined. When making the final calculations all of these data must be checked for possible changes and controlled till the day of delivery. What happens after this day? Where does the responsibility of the designer end: at the delivery of the building when he must tell the owner that after this day nothing should be changed anymore without the designer's consent? Or should he not trust the owner too far in this respect and allow for these possible changes in his calculations?

b. Live loads

Through field surveys we can obtain statistical data about a number of live loads, especially for those often occurring spaces like in houses, office-buildings, etc. Our surveys always show what is, not what may be.

Can we project our observations of the present into the future? How popular will waterbeds become? Or should we decorate the walls of each space with the data on which our calculations of its structure are based and leave the responsibility to the user? In this way we could give each distribution function a clear upper bound.

How much do we know of possible future changes in the use of spaces? If we can set clear limits to those loads over which the user has control and leave the consequences of overstepping these limits to him, then we can make considerable gains in this respect. Is this realistic and if not entirely so then perhaps to a certain extent?

As long as we confine ourselves to normal building structures we have reasonably accurate data at our disposal about the climatic loads wind and snow. What however do we know about "loads" like fire and gas explosions and how do we project this knowledge into the future? How careful are we in the use of new materials (calamity on the Isle of Man in 1973)? The installation of fire-resisting materials is a statistical-economic problem.

For the CUR-committee "Safety"^{*} the IBBC-TNO-Institute in Delft has done some research on a concrete floorslab with a span of 4.25 m and a thickness of 141 mm. In accordance with the Code Requirements the amount of reinforcing steel was found in a deterministic way. By considering all data of the slab itself and of the load as stochastic quantities a theoretical value of 1.4×10^{-14} for the probability of failure was found. By repeating the process, but now including the possibility of fire, the theoretical value increased to 3.6×10^{-4} . If we keep the amount of reinforcement and the thickness of the slab the same but increase the cover, the probability of failure due to overloading will grow, but due to fire will diminish. For this concrete slab the optimum was found for a cover of 33 mm, the probability of failure being 1.5×10^{-7} .

^{*}Members of the CUR-Committee A16 "Safety": D.Dicke, H.v. Koten, J. Kuipers, F.K. Ligtenberg, J. Strating, A.W.G. Thijssse.

The big increase in the use of natural gas makes the occurrence of gas explosions much more likely and can therefore influence the probability of failure.

This influence can be considerably reduced by appropriate measures. Summing up we can say that some of the types of live loads — as they are now — are fairly accurately known. Projection of data into the future is difficult but not impossible; these data cannot be defined as clear distribution functions. "Loads" like fire and explosions have a measurable influence on the probability of failure and must be taken into account. If we do not want to do this then we must take measures to eliminate or greatly reduce their influence on failure. These measures cost money: it is a question of economy.

c. Simultaneous loads

Information on this possibility must also be gathered. In applying deterministic methods we are often required to examine combinations of different loads or a different disposition of loads. From a probabilistic point of view the combination that causes the highest stresses or failure does not necessarily have to be the most dangerous combination: it is conceivable that the chance of this combination occurring is particularly small.

d. Forced deformations

These deformations, caused by differences in temperature or by shrinkage and creep usually hardly influence the probability of failure, but can cause cracking and deformations which may lead to damage.

3. The shape of the structure

The engineer must develop the overall shape of the structure in close collaboration with the designer of the building. This shape itself has a bearing on the safety and economy of the structure. Are there members of the structure that may cause extensive damage when they fail? Is there a chance of progressive collapse? Can we work towards an optimum combination of safety and economy in the design stage? Yes, that is possible with the help of the probabilistic concept of safety.

4. Materials

The materials selected for the structure must be closely related to its overall design. Here quality control plays an important role. Close inspection upon delivery may allow us to reckon with a lower bound in the quality distribution function. In the case of concrete poured in situ, quality depends largely on the builder and the supervisor. Very often, especially in cases where bending is predominant, the strength of reinforced concrete depends entirely on the quality of the steel. Here stringent (and expensive) requirements regarding the quality of the concrete do not make sense, unless they are necessary for other reasons.

5. Elements

Out of the selected materials the elements of the structure must be manufactured.

The care given to this process, the expertise of the people involved and the effectiveness of the inspection determine the strength of these elements to a large extent. A weak element will remain a weak element during the whole lifespan of the structure.

6. Joints

The elements of the structure must be joined together. The character of these joints, monolithic or not, can not only influence the safety of the elements themselves, but also the safety of larger portions of the structure. In case of monolithic joints the detailing of the reinforcement in the joint may play a role.

7. Calculations

The actual dimensions of the elements of a structure must be determined by calculations. To this end we reduce the real structure to a "model" which allows us to apply the principles of Mechanics to it. In this way we find the forces working in these elements, their resistance against these forces and their deformations. In these models we also schematize the loads and the properties of the building materials.

These models are rather primitive and we often find big differences between calculated and measured values for the deformations and the distribution of forces which is derived from them. As the conditions for equilibrium are usually fulfilled, the use of computers in attempts at making these models more true to reality does not influence the calculated probability of failure very much. Constructions and elements under compression and bending (like columns) however become very sensitive to errors in "schematisation" of the properties of the material and of their boundary conditions when the load approaches its critical value. If we deal with large series of the same type of buildings then Computer Aided Design might offer an extra possibility to find an optimum combination of safety and economy. Refinements in calculation, intended to produce safer structures, may make the calculation and also the working drawings more laborious: 10% higher costs here mean a costincrease of 2% for the whole building. Lowering the factors of safety in the present Codes by 5% would mean a cost-decrease of 2% for the whole building.

One more observation: a calculation is made only once, but its discrepancies with reality last the building's lifetime.

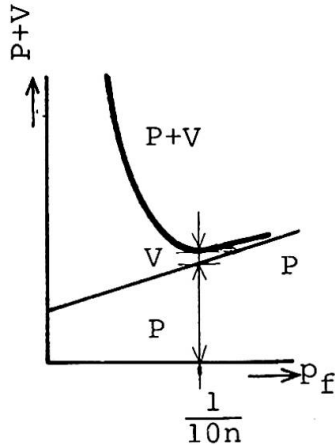
8. Use

After the delivery of the building, people live and work in it. This not only affects the loads (cf. 2a and b) but also the preservation of the building. How well will the owner take care of his building? Does he maintain it well, does he immediately notice extreme deformations, cracking, corrosion, ageing? Vreedenburgh once remarked that - had steel braces been put around the Campanile in Venice one hour before its collapse in 1902 - the tower would still stand as it was.

Optimum combination of safety and economy

The information gathered in items 1 through 6, with projections into the future, combined in the calculation (item 7) should enable us to determine the probability of failure.

Assuming that we should succeed, then we can certainly expect the question: "Is this safe enough?" Ligtenberg* has pointed out a possible criterion which establishes a direct relation between safety and economy.



If we call n the ratio of the total damage caused by the collapse of one element and the price P of that element, then the sum of the price P and the fictitious insurance premium V shows a minimum for approximately

$$p_f \cdot n = \frac{1}{10} \quad (p_f \text{ is the probability of failure}).$$

The difficulty of this otherwise simple and clear criterion is not establishing the material damage but translating human grief into an amount of money.

How do we "price" those who are killed, who are crippled for life, the wounded; the exasperation, loss of irreplaceable goods, loss of reputation, emotional and political consequences? As we confine ourselves to the great mass of relatively simple structures, this should not pose an insurmountable problem for a great many elements.

Let us take floors of houses as an example. We assume floors to be designed in such a way that collapse without warning is out of the question. Then the human aspect is reduced to some annoyance and inconvenience. Here failure takes on the meaning: inability to serve its purpose due to large deformations.

Let us suppose that the cost (including inflation) of replacing such a floor by a new one would be ten times the original cost ($n = 10$), then according to Ligtenberg we should design this floor with

a value $p_f = \frac{1}{100}$. This is the probability of failure, no matter if this is caused by excessive loading, fire or explosions. The probability of fire in a house during its lifespan also equals about $\frac{1}{100}$. If we know the coefficient of variation of the load and of the

resistance and also the influence of fire on failure, then we can establish a value for γ_k (i.e. the ratio of the characteristic values

of resistance and load). In this case the value would be 1.2 at the most. At such a low value for γ_k it is not unthinkable that other

requirements prevail. One might base the design on acoustical requirements for instance, and check deformations, cracking (in the case of concrete), stresses and economy afterwards.

Fire hardly influences the probability of failure in this case.

Starting from the simple rule of Ligtenberg $p_f \cdot n = \frac{1}{10}$ we can

arrive via a probabilistic concept of safety, at deterministic calculations. The question is: is it possible to classify the most common elements (as we have just done in a somewhat simplistic manner), to formulate a number of boundary conditions and to establish a value for γ_k ?

This could be a first step on the way to put the probabilistic philosophy of safety into full practice.

*ir.F.K. Ligtenberg: Discussion Summary, Vol.D.S. Planning and Design of Tall Buildings, page.437

Making the practicing engineer aware of the backgrounds of this method together with much research can pave the way for a more differentiated computer-programmed use of this method.

Systematic investigation and registration of all cases of serious damage would provide the facts and the insight to extend the application of this method to a wider range of elements.

I have not discussed loads brought on by tornados, earthquakes, floods and violence. Normal rules do not apply here. Elements exposed to these loads must be dealt with in a separate discussion. The relation we used between the probability of failure p_f and the ratio $n = \text{total damage} / \text{price}$ is an optimum relation which can only be influenced by the appraisal of damage that is hard to put in terms of money — the human aspect.

Probabilistic approach and public relations

A general belief in the absolute safety of our buildings still prevails. The probability of failure is not yet accepted. Whenever a building structure collapses it is a foregone conclusion for many people that this was either caused by the engineer's or contractor's negligence or blunder, or by an act of God. If we want to bridge the gap between scientific research and daily practice, we must explain the philosophy of safety to the practicing engineer. A probabilistic approach must be accepted. The author of this Report found this out when the notes of his student lectures on Safety were published as CUR-Rapport 63^{*}. There was an obvious need for a simple approach to new concepts of safety. This may form a basis for further study necessary for acceptance and application of these concepts.

SUMMARY

The idea behind this Introductory Report is this: it must be possible to gather sufficient data about the most common types of building structures to enable us to derive a design method from the probabilistic concept of safety. This method, using optimum economy as its criterion, will be deterministic in appearance, but will have a highly variable factor of safety. Possible causes of failure, other than overloading, should also be taken into consideration.

Moreover this Report appeals for making the probabilistic concept of safety more accessible to the practicing engineer.

RESUME

L'idée à la base de ce rapport introductif est la suivante: il doit être possible de réunir suffisamment d'informations sur les types les plus courants de structures, en vue d'en tirer une méthode de calcul basée sur un concept probabiliste de la sécurité. Cette méthode utilisant le critère d'une économie optimale, sera déterministe en apparence, mais offrira un facteur de sécurité extrêmement variable. Des causes possibles d'effondrement autres qu'une surcharge, devraient également être prises en considération.

En outre, ce rapport souhaite qu'un concept probabiliste de la sécurité soit rendu plus accessible à l'ingénieur praticien.

*Those interested in this Report can order a copy in English from: Secretariaat CUR, Postbus 61, Zoetermeer, The Netherlands.

ZUSAMMENFASSUNG

Folgende Idee liegt diesem Bericht zugrunde: Es muss möglich sein, genügend viele Daten der gebräuchlichsten Typen von Bauwerken zu sammeln, um daraus eine Methode abzuleiten, die auf dem probabilistischen Sicherheitskonzept beruht. Diese Methode, deren Kriterium die optimale Wirtschaftlichkeit ist, wird, äusserlich besehen, deterministisch sein, weist jedoch einen ungewöhnlich stark variablen Sicherheitsfaktor auf. Neben der Ueberlastung, sollten auch andere Versagensursachen in Betracht gezogen werden.

Zudem will dieser Bericht dem in der Praxis stehenden Ingenieur das probabilistische Sicherheitskonzept zugänglicher machen.

Serviceability and Maintenance

La serviciabilité requise et l'entretien

Nutzung und Unterhalt

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Life Cycle Considerations for Structures

This paper presents an overview of the various life cycle aspects and procedures applicable to structures. Treatment is in broad terms as regards types of structures. Discussion covers the life of a structure from conception, through planning, design, construction and implementation of use, to the longer period of operation and maintenance, ending in demolition or abandonment. Orientation is from the point of view of how the structural engineer -- be he planner, designer, materials supplier, equipment specialist, builder or administrator -- can best serve the interests of the owner and the affected public, while working with other professions engaged by the owner and regulatory agencies of government, to improve the creation and operation of a project. It is concluded: that the optimization of the primary aspects -- functional service, capacity to serve, environmental effects, time schedules, and cost to benefit relations -- are objectives within reasonable reach; and that favorable serviceability and maintenance depend directly on adequate planning, design, and construction.

Serviceability and Maintenance -- what are they and why talk about them. The practitioner of structural theory can well say our structures are excellently designed to carry specified loads. The researcher in materials can equally well state satisfactory materials are being incorporated into structures. The builder can claim with assurance that structures are built as designed. Everybody knows that most structures last a lifetime. They are torn down when no longer needed; practically never fall down. New generations of structures have benefited from improvement in design theory, in better materials built into the structure, and in more ingenious construction methods.

There can be no quarrel with any of those statements from the point of view of the structural engineer. Certainly structures of the past have given good service, the vast majority far beyond the call of duty. Certainly maintaining them in serviceable condition has been no more than a routine requirement except in isolated cases.

Such a discussion could easily backslide into great detail. Examples of former less than perfect practices which have been corrected could be cited -- elimination of pockets which did not drain and rollers which did not roll; introduction of "weathering" steel to eliminate the need for painting, particularly where service was affected; elimination of expensive false work by segmental or slip-form procedures. Examples of adequacy and progress include: carrying loads with surprising reserve capacity; greatly improved corrosion resistant coatings; welding to achieve economy, cleanness of detail and quiet; and so on ad infinitum. It is not the purpose of this paper to discuss such details. They are different for almost any structure, varying with location and climate, type of structure, kinds of materials used, service requirements, and accidental phenomena.

In passing, the point is made that no structure is better than its details. Attention to detail is therefore essential, most particularly from serviceability and maintenance aspects. This explains in part why the best designs have usually had the benefit of participation by individuals experienced in functional design, construction, operation and maintenance. Experience is a marvelous teacher of practicality.

The intent of this paper is to discuss aspects of serviceability and maintenance from other than the engineering point of view. After all, the structural engineer participates primarily in the creation of the structure. Others establish when and where it will be built, pay for it, take care of it, use it, and eventually dispose of it. Many of the design criteria are fixed by non-engineers. They all have important points of view with respect to serviceability and maintenance, both of a direct and indirect influence.

It is worthwhile to examine these other points of view and their influence on the work of the structural engineer. A structure is a complex mechanism. An influence exerted in one area is very apt to permeate the whole complex. The better the structural engineer understands, works with, and can put himself in the position of the other decision makers the more professional and satisfactory will be his services.

As the peoples of the world more keenly realize that resources are finite, as demands on resources inexorably increase both quantitatively and qualitatively, and as need for leadership in responsibility becomes ever more essential there emerges a greater and greater need to consider more fully for any structure its basic life cycle aspects. The five primary aspects of concern are functional service, capacity to serve, environmental effects, time schedules and cost to benefit relations. In each specific case these simply stated primary aspects are comprised of many elements. The elements of significance and the weight to be

assigned to each in a given case must be selected with due regard to responsibilities of all types -- human, social, economic, and monetary -- as well as technical. The value of a structure to society is measured by its performance.

The addition of non-technical considerations to the more usual technical engineering activities will be new to some structural engineers. To those who have had overall responsibility for projects and have had to develop justification for them it will be less strange. There will be those who will take the position that attempts to evaluate the above named primary aspects over the life of a structure will be futile because the time factor introduces too many variables which cannot be precisely evaluated. They can cite such items as wages, material prices, interest rates, taxes, and general economic level of activity. Certainly long-term projections cannot be precise. They are more useful on a comparative basis than on an absolute basis.

However, lack of precision is not a valid reason to avoid making the best possible analysis. As structural engineers know so well, any structure abounds with secondary stresses which cannot be precisely evaluated but this does not deter the designer from making his best possible analysis on the basis of known factors, theory, and his experience and judgment. It would be folly to merely guess at the size of structural members; and the same applies to the primary aspects named above, including their life cycle evaluation.

Almost every structure for constructive purposes (and we are not here discussing monuments whose sole purpose is to extol past achievements artistically) is related to one or more of the following:

- o Housing for people, business, commerce, industry, military or cultural activity
- o Production or processing of food, clothing or other items to serve people
- o Transportation of people or goods to support or to benefit from production, commerce, education, health care, recreation or communications
- o Utilization or control of natural resources for the benefit of people
- o Disposal of the by-products of civilization.

It follows that any new knowledge, process or procedure which improves the primary aspects of a structure has a long-range significance for people. It contributes to the ways and means of improving the quality of living through enhancement of a learned art for the benefit of people. In the final analysis that is what structural engineering is all about.

The key role of the structural engineer: from the immediate point of view is to create structures superior in every respect; and from the long-range point of view is to improve the lot of mankind at a price mankind can afford to pay.

Associated with a structural project are always four main categories of people which in fact comprise a project corps. Each category deals primarily with a different set of elements as is illustrated by:

- o OWNER - functional needs, regulatory controls, financial sources, amortization of cost, project timing
- o DESIGN TEAM - design criteria, construction materials and procedures, equipment, environment, design methods, construction contract documents, costs and benefits, time schedules
- o BUILDER - materials suppliers, labor, construction equipment, sequential schedules, weather
- o USERS - functional service operations, maintenance procedures and costs, contributions to quality of living.

All of these ramifications in no way lessen the demands on the structural engineer. To the contrary, the requirements thus thrust upon him are heavy additions to the need to be completely up to date on all aspects of his primary concern -- the physical design which appropriately translates the agreed upon functional services into a structure which will advantageously provide those services. Nevertheless, the added requirements cannot be ignored. History is replete with examples of needed projects not built because of misunderstandings; and of structures built and not serving as effectively as possible.

It is therefore appropriate that I.A.B.S.E. has broadened its interests to include cognizance of economics, construction procedures and environmental, social and political factors which impinge directly or significantly on structural design criteria and thus his practice. By broadening its base of operations, structural engineering will be following more closely the systems approach which, in plain language, means considering all the indirect as well as the direct angles and consequences to achieve proper mutual and full understandings with the decision makers before irreversible action is taken at any stage of a project.

It falls to the professional designer to provide the leadership that holds together a well rounded design team that makes timely and efficient progress in the formative period leading up to an appropriate master plan thoroughly understood and agreed to by all the elements of the design corps. This profes-

sional designer may be advisor to the owner, he may be a consulting engineer, or he may also be the builder. Whatever his position, his responsibilities differ only in detail. Actually, a professional engineer should be in all of the positions named on any sizable project. The engineer's strength lies in his thorough grounding in the relations between cause and effect -- various causes, multiple effects, all kinds of ways and means of solving apparently impossible problems within the limitations of almost immovable prerequisites. His training is primarily methodological; secondarily specific. He is realistic about costs and benefits. He is geared to making progress. His whole attitude is application of knowledge to produce a useful facility.

He can cope with concepts of appropriate parameters which have been added to design criteria for design of a facility. Reference is here made to a "facility" instead of to a "structure" because many structures are a part of a system to which these new parameters apply. The structural designer is, therefore, more often than not now a member of a design team composed of many types of professionals whose various aspects of design must be integrated into a composite facility. Although to safely carry required loads and to do so economically over a long time are still primary functions for most structures, such functions may become secondary to the full purpose decided upon for the system of which the structure is but one part.

These added considerations have arisen in recent years as the result of the public becoming involved in the planning and programming of those facilities which may affect environment, the ecology, and society. The public's greatest concern to date has been with respect to visible features. In the early stages of this awakening, desires were often only partially and unclearly stated and frequently reflected an uninformed background. Only recently has the public been forced to come to grips with such practicalities as funding, finally realizing that seldom can added characteristics be provided in a facility without adding costs and that cost factors must be considered when establishing the design criteria. Such realizations have served to temper the environmental, ecological, and social demands but have properly not eliminated them. An appreciation by the public that time is of the essence is still to come. The same applies to establishment of priorities.

This evolution which has changed the decision makers from a concentrated group of specialists to a non-homogenous group has, of course, lengthened the decision-making process. Time is of the essence in the planning, design and construction for any needed structure or facility. Repeated reviews, particularly partial and merely negative ones, false starts, change orders and stop orders add materially to time and costs in two ways and for no good reason. The records show that projects delayed in start, subjected to prolonged periods of construction, or changed in concept during design or construction always cost substantially more, sometimes by as much as one or two magnitudes. The record also shows that delay in receiving services from a needed project always subjects

the potential users to added costs for the reason that the existing facilities or lack of any are more expensive than those to be obtained from the new project -- otherwise why build the new one.

Some designs which met earlier simpler criteria for serviceability are, in the light of recent more sophisticated criteria, now considered inadequate. And, of course, since the public did not participate previously in the decision making, the public blames others for those inadequacies. During the transition, the public has been exercising authority without bearing the relevant responsibility. To continue in such a manner would lead to public bankruptcy. By brute force of budgeting realities that relationship is being normalized. Nevertheless it is still necessary for the professional structural engineers to act in a positive and constructive manner that will restore the confidence and respect of the public in the professional services they render.

Engineers cannot restore their credibility by talk alone. They must again earn that status by placing their actions in agreement with their talking, and do so in a manner which the press and the public cannot misunderstand. The cost and the time required to achieve beneficial use of the facilities created by designers, and the interactions between all parts of the system of which a structural design becomes a part have a definite bearing on regaining credibility.

Greater social responsibility for engineering based facilities is here to stay. It is in the interests of engineers to adjust to these new requirements quickly, reasonably and, as is most necessary in all parts of their work, intelligently. Frankly we have added participants in the normal interplay between cause and effect. Those causes and effects which are related to human nature can be as influential in a structural design as those related to Mother Nature.

This points up another characteristic of the structural engineer's work. The designer himself, for example, cannot be expected to become expert in detail in all aspects affecting design today. However, it has been shown by experience that the engineer as the principal will be charged with full responsibility for the work of others in his team, including design criteria dependent upon data or often only opinions developed by social, political, and economic "scientists."

The use of empirical data and of assumptions in formulating designs is not new to engineers. Such use has been required in order to formulate mathematical relationships needed to solve for the unknowns. In the past, assumptions and empirical data have been used only when rigor was not possible by reason of the state-of-the-art. Always the use of assumptions and empirical data was founded on relevant experience and applied with careful judgment to meet the unrelenting realities of the physical world. The so-called social, political, and economic scientists in stating their

design criteria have often (a) paid more attention to their concept of what the future should be than to experience, and (b) have often indulged in wishful thinking. In contrast the engineer knows that sky hooks will not support loads and, therefore, never tries to employ them. His failures are too quick and visible. Much benefit would result if the same certainty of consequence would become readily visible with respect to design criteria related to environmental, ecological, time schedule, and cost considerations.

A fair amount of rigorous analysis can be attained by conscientious adherence to established relations between cause and effect in human matters. The social, political, and economic scientists should recognize these controls or cease to call themselves scientists. Often their half-truths, innuendos, and unrealistic conclusions have telling economic and functional impact on the serviceability of engineered facilities for which the design criteria have been impacted by significant influences rooted in environmental, social, political, and ecological niceties which may or may not be feasible. A statement credited by the Xerox Corporation to Joseph C. Wilson is apropos:

"Organized human endeavor can be lifted an order of magnitude through leadership if it is inspiring. The springs of inspiration lie deep in the knowledge of all that is worst and best in men and in the wholehearted acceptance of that worst and best. To lead well is to know people and to know, above all, that they are always people. The roots of that knowledge are in the sturdy minds and noble souls of the centuries."

Adequate definition of the proper functions of the public and of the private sectors and assigning to each sector its proper role founded on rational analyses of cause and effect is necessary. Goals for environmental excellence depend upon such accomplishments. In addition, major progress toward excellence in the primary aspects of structures -- functional service, capacity to serve, time schedules, and favorable cost to benefit relations -- can be achieved through design excellence at little cost if such cooperative efforts could be pursued diligently in serious manner. One must realize that the leverage of such possibilities is great. Good and thorough design need be only a really small portion of the cost of a structure; but the savings and benefits of good design as reflected in the construction, operation and maintenance costs are many times greater than total design cost.

Structural engineers have a particular responsibility to frame the relevant influences of their expertise in form understandable by non-technical decision makers and, further, to see that the message is received. Talking about costs is a good way of getting the public's attention.

Cost considerations related to serviceability and maintenance include:

- o Money for the physical creation of the project
- o Environmental and community impacts
- o Resource utilization
- o Money for operation and maintenance throughout the life of the project.

The direct money values are readily arrived at once a definitive master plan and time schedule are actually adopted and frozen.

The environmental and resource factors are not yet susceptible to such specific determination, as has already been cited, being founded more on learned opinion than on evident fact. Here each critic, self-appointed or official, measures the impacts with his own elastic tape measure. Until such time as such critics resolve to work in the public interest instead of to show how smart they are, the reaching of the necessary mutual decisions will continue to be difficult.

In the transition, it can be demonstrated that any lack of adequate consideration of environment and of resources in the past was as much dictated by statute and criteria of owners emphasizing money economy as it was from neglect of the design professions. More recently, the criteria of owners, particularly government, have swung to the other extreme, practically ignoring money economy and timely design and construction. This brief recital of responsibility is solely for the purpose of emphasizing that the arrival at satisfactory design criteria with respect to the primary aspects of the structure and with due regard to all types of economy is a multidisciplinary responsibility. In the interests of true efficiency and economy the needed understandings, decisions and agreements should be reached expeditiously with adequate recognition of the state-of-the-art rather than requiring that one group propose and another denounce thus causing a long and wasteful iteration. Perhaps a lesson can be taken from the arts. Music, art, and drama critics are seldom successful composers, artists, playwrights or performers. Similarly, the public should recognize that the critics of structures usually could not themselves do the work of the creators in economical and useful manner. However, no one is perfect and informed constructive criticism can be very helpful.

Almost any structure or facility of significant proportions is a complex mechanism. Each aspect is interrelated. Changing design criteria will change all aspects to greater or less degree. That is why it is practically impossible to discuss any one aspect in isolated manner. For example, almost everything done affects cost, maintenance, and serviceability in some way. Costs accumulate from the time of initial thought of conception of a structure until the time it is no longer used, requires care, nor influences the environment. The sum total is referred to as life cycle cost. The subject merits mention in the context of serviceability and maintenance.

In passing it is observed that services which have been provided by a structure may still be needed even after the structure is 100 or more years old and has outlived its efficient usefulness. In such case, demolition and replacement costs enter the picture. This is a common event in the industrial and manufacturing areas and the result is more visible because it happens within a short time for many items of equipment. In the field of structures the time cycle is usually so great and the future social, political, and economic climates so uncertain as to make such consideration at the time of design of little merit in any way other than to amortize the cost of the structure being designed. In the same way, neither the design engineers, the owners, nor anyone else can peer into the future and focus on the route the economy and the style of living will follow. Therefore the usual practice for structures is to assume that economic, social, and political climates will retain their characteristics except that population growth and distribution, industrial growth, tax increases, and inflation will continue their historical movements.

The relative weight of life cycle cost elements will be quite different for different structures. All of the primary aspects of a facility are involved and to some extent can be varied to suit the owner's convenience and purposes. For example, initial capacity can be kept low with provision for expansion as the future may require within a range of alternates. Thus initial construction costs can be decreased while future costs will be increased, probably by more than is initially saved. Even so, there may be perfectly valid reasons for such a course.

This paper will not attempt to become specific in such matters. On the other hand most of the various elements will be present in nearly every case. A look at the forest will be helpful. Individual engineers will need to establish their own specific parameters for their own separate projects. In doing so they will give due consideration to the fact that governments often erroneously ignore certain cost areas such as rent, interest, taxes, administrative costs, fringe benefits for employees, insurance, and general overhead. Nevertheless such cost elements are important long-term cost factors especially when comparing alternate solutions, including public vs private sector participation, to arrive at the master plan. Such cost factors don't just go away even though they may not show in the departmental appropriation. The people still pay for everything they get; and sometimes for what they do not get.

For illustrative purposes the following partial list of cost and benefit elements which enter into life cycle evaluations will suggest the breadth of that approach to consideration of the five primary aspects of a structure with respect to return on the investment in terms of services and maintenance:

- (A) Surveys and projections to determine character and magnitude of existing functional needs and their future growth rates;

- (B) Preliminary studies and planning prerequisite to defining the master plan through analyses of alternates for the structure, including the design criteria and provisions, if any are needed, for future growth, relationships between time schedules and costs, analyses of environmental impacts, costs and benefits, and impacts on the economy of the community local to the structure;
- (C) Feasibility studies to compare (a) estimated long-term benefits to be received as the result of services to be provided by the new structure as contrasted to existing ways and means with (b) the estimated expenditures required to provide, operate, and maintain the new structure and abandon the old one, if any exists, as contrasted to maintaining the existing facility. These estimates should be detailed rather than lumped into large units. Such practice will assist accuracy and facilitate understanding by the decision makers of comparative advantages and costs, of alternate possibilities for various parts of the structure as well as for the whole structure, and with respect to phased construction as needed rather than providing capacity for future growth at the outset;
- (D) Financial planning and funding; including interest, amortization, debt service functions performed by trustees, lawyers, and consultants;
- (E) Obtaining permits and other required authorizations;
- (F) Hearings, reviews, audits, and similar checking operations which interfere with productive efforts;
- (G) Final design and completion of all construction contract documents;
- (H) Site selection and acquisition, including all associated secondary items;
- (I) Legal fees;
- (J) Accounting and auditing costs;
- (K) Insurance of various types, including all risk, liability, workmen's compensation, health, unemployment, loss of use and occupancy;
- (L) Administrative and management costs of owner's staff;
- (M) Resource use and preservation;

- (N) Construction and inspection of construction;
- (O) Interest earned on unexpended fund balances;
- (P) Interruption of on-going operations during construction, including inconveniences to others as well as to owner;
- (Q) Salvage values of discarded existing facilities;
- (R) Start-up costs, including operating manuals, selection and training of operating personnel, accumulation of inventory, and shake-down trials;
- (S) Taxes and/or loss of tax ratables;
- (T) Operation and maintenance including staff, equipment, materials and supplies, energy and other utilities, overhead, depreciation, and demolition.

A truth too infrequently appreciated in the initial concept stage, long before tangible details can be determined on a reliable study basis, is that some of the most important and far-reaching decisions with respect to serviceability and maintenance are required at that stage. In the absence of specifics, these decisions must be based on the judgment of experts trained by experience as well as in theory. Such judgments must reflect an awareness of local pertinent factors and influences. Examples of such early questions are: does the project appear feasible to an extent warranting careful exploration; what goals and objectives shall be established for the project; what is the order of magnitude of the costs and of the benefits over the years and how are they distributed over the time span; which of two or more projects should be selected when financial resources are limited; what time frame should be established; is it best to start with a low estimate or a high estimate of cost; is it better to organize for high initial rate of cost to gain low annual costs subsequently or vice versa? Many such questions could be more precisely resolved after extended study (and expense therefor) but practicality precludes endless study. The number of alternates to be investigated must be kept to a reasonable maximum even though for the usual structure these study costs are a miniscule portion of the total cost.

The key to this aspect of service with economy is a combination of theory, experience, and judgment. Hence the need for accumulated reliable data of benefit to every one, most particularly the ultimate consumer. Another requirement for coming up with the best solution to the continuing questions and problems is to pick the best qualified project corps at the very beginning and then keep it as it gains additional useful experience with the specific project and the manner in which it has developed. Such a life-line should flow through a job from concept into routine operation and maintenance long after implementation of service. Accumulation of written data cannot replace the human element in such cases.

It will be very helpful to develop a system of life cycle evaluation utilizing a universally understood terminology and viable methodology. Only in such manner can accumulated experience be intelligently applied. To emphasize the need for this universal language of communication between all parties concerned (owners, users, bankers, lawyers, designers, government) it is only necessary to recall the wide-ranging concepts now in use for one secondary cost element, namely overhead. Differences on the order of several magnitudes result from differing definitions which in turn affect evaluations of other elements.

It should be kept in mind that we are not here speaking of some new device. The experience of the authors includes evaluations similar to those being discussed on a wide variety and size of projects over a period of twenty-five years on facilities for which the aggregate construction cost has been several billion US dollars. The results have been remarkably good. It is significant that most of the projects concerned could not have been built without such analyses.

Approval of budgets and accomplishment of financing in timely fashion are more readily and economically accomplished for any projects if the rigorous feasibility procedures perfected for revenue bond projects are followed, thus providing independent certification in formal manner for all the engineering, legal, and financial data including purposes to be served and estimated return on the investment.

A cash-on-the-barrel-head-type of revenue supported financing through private channels has been used over many years very successfully for a wide variety of public projects, e.g. highways, bridges, airports, power generation and distribution systems, water supply and waste disposal systems. Not very different procedures have served for business and commercial projects, e.g. office buildings. The authors are convinced that the public interest would be served in additional public works projects, including structures, if similar rigorous procedure could be applied. Benefits to accrue could include elimination of unjustified projects, and establishing reasonable priorities for needed projects.

Engineers have a responsibility to encourage use of such supplements to their technical services as a means of providing needed services at reasonable cost for the benefit of the public. The best part about such improvements is that everybody benefits, including the engineers. Not the least of his gains will be improved stature and respect for this work.

At the same time, owners have a responsibility to realize that adequate evaluations for the primary aspects of a structure or project cannot normally be made until after (a) designers have had time to develop mutually with the owners full definition in

technical terms of functional services to be provided by the project; (b) the designer has studied various alternate solutions and acquainted the owner with the results to an extent sufficient to convey full understanding thereof; and (c) both have agreed upon the characteristics of the master plan to be finalized. Meaningful life cycle evaluations cannot be made in casual manner and are not an exact science. They must be done with care and developed in an atmosphere of mutual confidence.

In recent years there has been a spate of novel procedures suggested by administrators as panaceas for too costly structures, taking too long to build, and not meeting desired service requirements when finished. That such difficulties have existed is true but the usual cause has been violation of the basics set forth above. The suggested cures do not treat the causes of the disease: insufficient authority at lower levels of administration, incomplete planning, inadequate design criteria, decisions made too late and changed too often.

The odds favoring thorough planning, adequate design criteria, and efficient management during planning, design, and construction are much too great to ignore. The time and cost are far less than for mediocre analysis and design; the resulting serviceability far better. The most complete and expert services in this stage cost but a small amount of the total life cycle cost and they exert a powerful leverage on the remainder. The actions required of the owner to reap such benefits are not difficult; they are within easy grasp. Structural engineers can assist the owners in achieving desirable goals and have a responsibility to do so. After all, it is the results of engineering work that can make the difference if given appropriate opportunity.

Government has a unique responsibility in connection with serviceability and maintenance of structures. These aspects are influenced, sometimes even controlled, by building codes, zoning restrictions, and inspection procedures promulgated and administered by government to protect and benefit citizens. Today, these controls as they exist and are administered, frequently prevent the project corps -- owner, design team, builder, and user -- from achieving optimum results. The controlling documents are usually seriously out of date. The administration of the requirements is often by persons having insufficient knowledge and very little authority except in the negative. Any serious attempt at constructive progress via such controls is a rarity. It can also be observed that deliterious influences stem from selfish demands and restrictions sponsored by labor and by materials producers with the consent of government. It makes no difference whether that consent is in the form of silence or endorsement. Unfortunately the political well being of elected officials and the achievement of maximum benefit to cost ratios for public structures often appear incompatible when viewed solely in the context of the moment as contrasted to the long-term public interest.

It is certainly true that what has been accomplished in the past with structures in spite of all difficulties is very commendable and considerable. All concerned can take justifiable pride. Nevertheless, no human ever did anything perfectly. What could be accomplished would be unbelievable if all parties concerned cooperated constructively in full accord with the state-of-the-art with respect to life cycle evaluations of functional service, capacity to serve, environmental effects, time schedules, and cost to benefit relations. Serviceability and maintenance would be greatly improved and our structures would cost very much less. The public would be well served.

A major step forward in improving serviceability and maintenance of structures should be made by engineers taking the initiative to improve communications between owners, designers, builders, and users on a multi-lateral basis. Improvement of mutual understandings of the needs and problems of others could accomplish wonders. As one example of great opportunity, structures to house adequately a large proportion of the peoples of the world are prime candidates for such progress.

SUMMARY

Serviceability and maintenance are generally treated in the context of a systems approach during formulation of design criteria, master planning, design and construction. Emphasis is on continuity of considerations, need for achieving thorough mutual understandings between owner and designer, and flexibility and importance of cost-benefit relations.

The conclusion is that in many cases improvements at relatively small cost are a reasonable possibility.

RESUME

La serviciabilité et l'entretien sont en général pris en considération dans l'approche globale d'un système, lors de l'établissement du cahier des charges, du projet général, du projet d'exécution et de l'exécution. L'accent est mis sur la nécessité d'une analyse permanente et d'une collaboration totale entre le maître de l'oeuvre et l'ingénieur. Les relations coûts-avantages doivent être prises également en considération.

Dans de nombreux cas il est ainsi possible d'apporter des améliorations pour des coûts relativement faibles.

ZUSAMMENFASSUNG

Nutzung und Unterhalt werden im allgemeinen in einer gesamten Systemstudie betrachtet, sei es während der Festlegung des Pflichtenheftes, des Vorprojekts, des Ausführungsprojekts oder der Verwirklichung. Die Notwendigkeit einer permanenten Analyse und einer totalen Zusammenarbeit zwischen Bauherr und Ingenieur wird unterstrichen. Eine Studie der Kosten-Vorteile Beziehungen ist ebenfalls aufgeführt.

In vielen Fällen ist es so möglich, Verbesserungen zu relativ kleinen Kosten anzubringen.

II

Progrès dans l'optimisation structurale

Fortschritte in der Optimierung von Tragwerken

Progress in Structural Optimization

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Optimization Concepts and Techniques in Structural Design

Concepts et techniques d'optimisation

Grundlagen und Methoden

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I. INTRODUCTION

Since the early 1960's structural optimization has established itself as an important area of research in structural engineering and it is now gradually becoming an integral part of the technological expertise of practising designers. The phrase "structural optimization" conjures a very specific impression in the minds of most engineers relating to the application of numerical search techniques to mathematical models of certain types of structures. Whilst this impression is correct in detail it is by no means the whole story. The history of architecture and structural design has been characterized from the very earliest times by efforts to design structures which were in some respect "better" than their predecessors; to use materials and resources more efficiently and to design structures which were as well-suited as possible to their functions. This continual search for improvement is essentially an optimization process, to use the term in its widest sense, and it pervades all present-day engineering activities.

On a national and regional scale governments seek to invest in new industries, resources and communications so as to obtain maximum benefit from a limited investment. At the project level an engineer strives to produce efficient, cheap and reliable designs for projects to satisfy functional requirements. The contractor wishes to use his resources of manpower, machinery and capital in the most efficient manner and the client is concerned whether his structure will be economical to run, to use and to maintain. The whole process of project planning, design, construction and operation is governed by the need to produce the best possible solution. Traditionally, emphasis has been placed on the intuitive skills and ability of experienced planners and engineers to produce near-optimal solutions to problems. However, the demands of an increasingly technologically-based population now require that projects are larger, more expensive and more complex. Financial pressures towards cost economy are increasing. Restrictions imposed by aesthetic, social, environmental and technological factors are ever more stringent. A result of this increasing complexity of the design process is that traditional reliance upon the skills of individual designers must change to meet present-day circumstances.

Consequently, more assistance is now sought from the digital computer, with its speed and power to solve complex problems rapidly, to meet these demands.

Since the digital computer was first used on structural engineering problems over twenty years ago it has completely transformed methods of structural analysis, so that now the use of matrix stiffness and flexibility methods as well as finite element techniques is commonplace in structural design offices throughout the world. However, as it became possible to analyse increasingly complex structures it also became more difficult to interpret the results of the analysis in a logical way for design purposes. It was, therefore, a natural trend in research to try to use the computer to produce design-oriented information rather than analysis-oriented information for the designer to interpret. Thus over the last ten or fifteen years research into aspects of computer-aided structural design has considerably increased. Once it was recognized that the engineering planning and design process is an optimum-seeking one it was also natural to see whether the computer could not also be used to produce not merely design information but optimum design information so that the structural engineer could play his part effectively in the planning and design of efficient, cheap and reliable structures. The computer, therefore, afforded the possibility of helping with the two basic problems mentioned above of the increasing technological complexity of structural design and the growing pressures towards cost control and economical designs.

However, just as the computer revolutionized structural analysis by demanding new techniques suited to its abilities a similar revolution was needed in design techniques if the computer was to be of real use in this area. In the early 1960's Schmit¹ and others gave a first insight into a new computer-oriented approach to structural design by examining the application of newly-developed techniques of mathematical programming and optimization to problems of structural design. Thus structural optimization in its modern form was born. With the passing of time it is perhaps difficult to realize that in the early 1960's the philosophy of structural optimization was really very novel. The early research referred to above clearly demonstrated what had not previously been self-evident: That the largely ad hoc processes of structural design which appeared to have little formal logic may in fact be expressed formally and in mathematical terms, and that there was as much rigour and logic in solving an optimum structural design problem as there was in solving a structural analysis problem. Perhaps the largest contribution which the study of optimization has given to structural engineering is that it has put structural design on a formal, mathematical basis and by so doing has unified a previously fragmentary and ill-disciplined subject.

Coming some fifteen years after Schmit's original work this paper examines some of the more recent developments in structural optimization techniques. It shows that there are still many difficulties to overcome for the full potential of optimization to be realized and it is hoped that it will give an insight into the way in which structural optimization creates a greater understanding of the nature of the design process and of structural behaviour.

2. A HIERARCHY OF STRUCTURAL OPTIMIZATION PROBLEMS

The rationalizing and formalizing influence which optimization has had upon structural design now means that structural optimization problems may be conventionally expressed as:

$$\begin{array}{ll}
 \text{Minimize, or Maximize} & g_0(x_i) \quad i = 1, \dots, N \\
 \text{Subject to constraints} & g_j(x_i) \begin{cases} < \\ = \\ > \end{cases} 0 \quad j = 1, \dots, M \\
 & x_i \geq 0 \quad i = 1, \dots, N
 \end{array} \quad (1)$$

The variables in this problem, x_i , $i = 1, \dots, N$, usually represent physical parameters of the structure to be designed such as dimensions, spacings, bar sizes, plate thicknesses, etc. These variables are under the designer's control and he wishes to find "best", or optimal values for all of them. The objective function $g_0(x_i)$ which is to be extremized represents some evaluable criterion of efficiency of the structure. The efficiency criteria most frequently used are such things as minimum structural weight, minimum cost, maximum factor of safety, etc. The choice of objective function will be commented upon later. The M constraints $g_j(x_i)$ may be equalities or inequalities and they originate from many sources. They specify, for example, the mechanical behaviour of the structure under load, the known properties of the materials used, requirements of relevant codes of practice, fabrication requirements, geometric and layout requirements, etc. All relevant restrictions and requirements upon the structure must appear among the constraints. The non-negativity condition upon each variable is necessary to ensure that values are obtained for all the problem variables which are real and feasible in an engineering sense. Sometimes integer values or values from a discrete set may be required.

Generally structural optimization problems are large and non-linear except for all but the simplest structures. This is seen if a single, very much simplified structural element is examined. Fig. 1 shows the cross-sectional shape of a typical reinforced concrete T-beam.

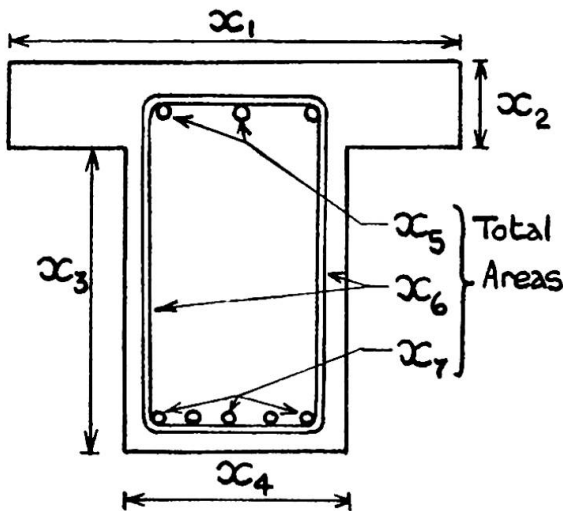


FIG. 1

To design this beam of known length and loading so as to minimize a simple cost function involves finding optimal values for the seven variables shown. The cost function g_0 is fairly simple to write down. It involves the cost of the concrete and of the steel, both roughly proportional to the volumes of concrete and steel, and it also involves the cost of shuttering the beam during pouring. This cost depends upon the perimeter of the cross-section. The objective function is then a simple one but is non-linear in the variables. Many constraints are necessary to ensure that the concrete and steel can adequately carry the bending stresses in the beam and also resist shearing stresses. Codes of practice prescribe maximum permissible values for these stresses and also prescribe a large number of other permiss-

ible values for such things as deflections, crack widths, bar spacings, etc. There may be fabrication constraints which limit, for example, the bar areas to be not less than some available size. The designer may also add constraints himself if the beam has to fit some restricted location within a structure. Typically to fully describe the optimum design problem, between ten and twenty constraints may be needed in the seven variables and because of the functions involved in these constraints they are almost all highly non-linear ones.

Problems of this size and complexity can now be solved fairly rapidly by a variety of methods which will be outlined later in this paper. However, this example is merely a single beam element. Most real-world structures have many elements such as beams, columns, slabs, panels, etc., and to fully describe the optimum design problem for such real-world structures may require many hundreds of variables and constraints. The general characteristics of structural optimization problems are therefore that they are large, multivariate, non-linear, constrained problems.

Because problem size and complexity become enormous when all variables and constraints are lumped together in a single problem a hierarchy of problems has developed. The hierarchy is as follows:

1. Topology of the structure
2. Geometry of the structure
3. Overall sizes of structure members
4. Detailed design of elements

The logic and implications of this hierarchy can be demonstrated by reference to the beam example which falls into category 4 of the hierarchy as it is a detailed design of an element. Suppose this beam was one of a known number of beams supporting a deck slab longitudinally. By optimizing the cost of each beam in turn cost savings would accrue over a non-optimized design and may be considerable. However, the inquiring designer will ask himself whether the specified number of beams is itself optimal. If more longitudinal beams were used the load to be carried by each would be reduced and so the necessary size would also be reduced. The cost savings to be gained by using a larger number of smaller beams or a smaller number of larger beams might well be far greater than anything achieved by merely paring down upon detailed design sizes. This type of problem falls into category 3 of the hierarchy.

If the deck and beams form part of a bridge over several piers then the interested designer will soon find himself considering the cost savings to be made by varying the distances between piers to arrive at an optimum geometric arrangement in category 2 of the hierarchy. This promises the possibility of even greater cost economy. Finally category 1 which holds out the greatest savings of all, is concerned with topology. Was the decision to build a deck on concrete beams over supporting piers itself optimal? Would not an alternative structural form be more efficient? Why not use a steel box-section deck?

This example demonstrates the logic of the hierarchy fairly well and also demonstrates two general features of it. Starting from the lowest category 4, the higher up the hierarchy that optimization can be used the greater the potential for economy becomes but also the more difficult optimization becomes to implement. This second feature is reflected by the fact that very little work has been published on the optimum topologies of real-world structures, the work of Michell² being of theoretical interest rather than practical use. Indeed, the present state of the art of structural optimization is that a vast amount of research work has been published on methods for category 4 problems, and it can be stated that most problems in this category can be solved fairly rapidly. Much work has also been done in category 3 but success there has been less general and new techniques are required. The available literature in category 2 is fairly small but has significantly increased in the last two or three years. Much more work remains to be done in this category and in category 1 where significant practical literature is almost non-existent.

3. APPROXIMATION CONCEPTS

Very often approximations are used in the formulation of optimum design problems for two reasons. Firstly they can be used to hold problem sizes down to a level at which computer solution does not become inordinately expensive. Secondly they can be used to decompose a large design problem into a series of smaller problems. Another simple example demonstrates this. Consider the problem of the optimum elastic design of the beam/column framework of a multi-storey building such as that in Fig. 2. Each member of this framework may have

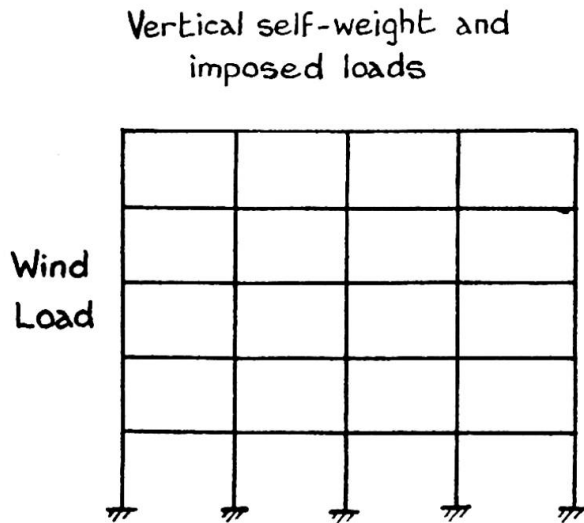


FIG. 2

up to ten detailed dimension variables and perhaps ten to twenty constraints. A complete optimum design problem for such a structure including both overall member sizes and detailed dimensions (i.e., categories 3 and 4) would therefore involve hundreds of variables and constraints and would be impossibly expensive in computation time.

Approximations can be used to effectively decompose or separate out the large problem into a single category 3 program and many simple category 4 problems. This is done by selecting for each beam or column element of the structure a single parameter and relating the cost, weight and behaviour of the element to this single parameter. In this

particular example a suitable parameter might be stiffness, $E I$. Each element has thus been approximated and obviously in problems generally care must be taken to ensure that the approximate element really does behave as the real element does. The framework is then reassembled using approximate elements and a single category 3 problem may be formulated. This would consist of finding a complete set of optimal stiffness values for the framework which minimizes the cost of the framework while satisfying as constraints the equations of equilibrium and compatibility for all the applied load cases.

Having found the set of optimal stiffness values an analysis of the frame using these values determines all the beam and column moments and forces. Each element may then be designed separately as a category 4 problem so that its cost is minimized over all five to ten detailed variables. The loadings are those obtained in the analysis and the constraints would be the normal detailed constraints plus an additional one that the stiffness of the element must be equal to the value obtained for the approximate stiffness parameter in the over-all size optimization.

Decomposition by means of approximations such as this is very widely used with considerable success. Frequently, in the aerospace industry very complex structures such as wing boxes, fin and tail structures, etc., are approximated as assemblies of membrane plates and shear panels for which rapid minimum weight design programs are available. One note of warning however; the last stage of any optimum design problem should always be a detailed analysis of the optimum design to ensure that approximations have not caused errors to be introduced. Indeed, on a more general basis it should perhaps be made clear that optimization is not intended to replace the designer. The objective is to provide him with information on what the most efficient solution to the problem posed might be. Only the designer can know whether the problem he posed is a complete one

and whether it was representative of the real-world structure. Optimization can only suggest a possible design which the designer is then at liberty to modify or reject or accept after further analysis. There are always factors in the mind of a designer which he cannot quantify but which nevertheless influence his designs. In using structural optimization to assist him the designer should attempt to formulate a representative problem only. According to how much he feels has been put into the problem or omitted he alone will know how much reliance to place upon the suggested optimum design and also how much he must alter it to satisfy himself regarding the unquantifiable factors such as aesthetics. Structural optimization produces preliminary design information and should never be expected to produce a final design.

In this section on approximation concepts it is useful to comment further upon the choice of objective function to be extremized. In purely technical problems involving reliability, dynamic response, etc., the correct objective function is usually fairly obvious. In more general problems cost or weight usually provide a direct measure of the efficiency of a design. In the aerospace and shipbuilding fields weight is generally of paramount importance partly because it directly reflects an element of cost but more because it directly affects the operational efficiency of the vehicle. In civil engineering weight is perhaps not so important and cost seems to be the vital factor in design. Some structural design applications of optimization in the civil field are frequently criticized because weight has been used as an objective function or, if cost has been used then not enough of the elements of cost arising from fabrication, erection, labour, etc., have been used or the cost coefficients are invalid in an inflating economy. Sometimes these criticisms are valid but frequently the solution is relatively insensitive to variations in the objective function. What is important in constructing an objective function is to ensure that all the variable major elements are included with coefficients of the right order. If, in the future the price of steel was to reach the present price of gold then designs for steel structures would change only minimally since labour costs would likewise have escalated along with costs of all other materials. Cost objective functions are almost always approximations in themselves and as was stated earlier the important thing is to ensure that they are truly representative approximations.

To conclude this section on approximation concepts the occasional requirement that variables must have integer values or values picked from a discrete set is examined. These requirements considerably complicate the solution of problems. A useful way of handling them is first of all to ignore them and solve the problem using continuous variables. The solution of this problem will then be in the approximate area of the solution for integer or discrete variables in most cases. Pathological examples can be constructed mathematically in which the integer/discrete optimum is completely different from the continuous optimum but this does not occur in real-world structural optimization. Having found a continuous optimum integer programming may be used to find the integer/discrete optimum in that region, or, as is perhaps more appropriate, the designer in his analysis, checking and modification can round the continuous solution to suitable discrete values. This continuous approximation to discrete functions is most useful by virtue of the fact that it has reduced the area of search for a discrete optimum to a small region around the continuous optimum. Had this discrete search been made over the whole feasible design space it would be a very lengthy procedure.

In the field of steel civil engineering structures geometric similarity may be used to construct continuous functions representing the section properties of rolled steel sections which are available only in a range of discrete sizes. Consider, for example, the range of available wide flange beams. Select a characteristic cross-section parameter such as section area, A . The assumption is now made that all beams in the range of available sizes are

geometrically similar in shape but differ only in scale. Then with this assumption it can be shown that the section modulus Z of a beam in this range is related to its area A by the relationship:

$$Z = C_1 A^{\frac{3}{2}} \quad (2)$$

Similarly the moment of inertia, I and the depth d are given by:

$$\left. \begin{aligned} I &= C_2 A^2 \\ d &= C_3 A^{\frac{1}{2}} \end{aligned} \right\} \quad (3)$$

and similar relationships hold for all section properties which may be related to a single characteristic parameter. The coefficients C_1, C_2, C_3 , etc., are all constants for the particular range of sections and may be found by examining any discrete beam within the set.

This continuous approximation has essentially replaced the set of discrete beams by a single variable A to which all section properties are related by known functions (2),(3), etc. If variable A is used in an optimization problem the optimal value A^* will correspond very closely to perhaps two or three beams within the set from which a discrete member may be selected. The assumption of geometric similarity is not absolutely valid for all sets of available rolled sections but is sufficiently accurate for the purpose here.

Thus far this paper has examined some of the major concepts of structural optimization and some of the general techniques involved in problem formulation for real-world structures. Very often it is difficult to formulate a representative mathematical model for the design of some structures particularly in the architectural area where the component of subjectivity in design is far greater than objectivity. However, having formulated a problem in the general form of problem (1) it is necessary to be able to solve it, and so solution methods for structural optimization problems will now be considered.

4. SOLUTION METHODS IN STRUCTURAL OPTIMIZATION

It is important in any survey of methods of structural optimization to differentiate between mathematical and structural optimization techniques. Although problems of optimum structural design can be expressed in mathematical form (problem (1)), the ideal solution technique from the viewpoint of a structural engineer is quite different from what a mathematician would consider ideal. Basically the structural engineer is interested in the structure which the problem represents rather than the problem itself. He is interested in the results of the optimization rather than the means whereby they were obtained. Any optimization method for use in engineering problems must therefore be flexible enough to solve as wide a range of problems as possible - frequently problems will alter several times as new constraints and variables are introduced by the engineer to more accurately represent his real-world design problem. The method used should be robust in operation and reliable - the engineer wants useable results and is frustrated by a solution method which in operation is very sensitive to the mathematics of a problem. Any method should be comparatively easy to use and should require a minimum of pre-solution computer programming and preparation. The engineer is comparatively little bothered by the need to differentiate between global and local optimality of solutions since a great many structural optimization problems display very flat, plateau-like optima. Similarly, extreme accuracy of the solution is not necessary since the mathematical problem itself is only an approximation to a real-world structural design and the designer is aware that the optimum solution is only a guide for him and will probably require modification in accordance with factors not included in the mathematical formulation.

The mathematician, however, has a different viewpoint and is interested in the means whereby results are obtained. He is happy to develop a method which solves a very limited class of problems if it is efficient on those problems. Flexibility of a method is of comparatively little interest. Methods are often reported in the mathematical literature which have low reliability - they are efficient on some problems yet fail to produce results for purely mathematical reasons on problems which to all intents and purposes look very similar. Such methods are of little interest to an engineer who wants results all the time. By training, the mathematician concentrates upon such factors as solution accuracy, speed and accuracy of convergence to the solution, and differentiation between local and global optima.

It is not the intention to disparage the approach of the mathematician in developing optimization techniques. Without this work structural optimization would not be at its present stage of development. It should be stressed however that it is very necessary to examine all mathematical techniques carefully to determine whether they are suitable for the needs of the structural engineer. Very often the structural engineer uses relatively simple and crude techniques to solve problems, not out of ignorance of more sophisticated methods but because he can place reliance upon the results obtained. This point is often mis-understood but is an essential difference between mathematical and structural optimization.

The methods now described are those which, after careful examination and thorough testing by structural engineers, have been established as suitable for optimum structural design problems. The criticisms levelled at them are likewise based upon the performance of the methods on problems arising in the design of real-world structures. At this stage it is useful to restate the basic characteristics of most structural optimization problems which are that they are large problems with many variables and a large number of non-linear constraints.

4.1 Unconstrained Methods

It may seem odd to commence a survey of methods of structural optimization with methods for solving unconstrained problems since structural design problems almost always have a large number of constraints. However, many of the concepts of unconstrained methods are useful in constrained problems and also there are methods which transform problems having constraints to unconstrained problems. Unconstrained optimization can be formally expressed as:

$$\text{Minimize } g_0(x_i) \quad i = 1, \dots, N \quad (4)$$

It is not necessary to consider maximization separately since this can be effected by minimizing the negative of $g_0(x_i)$. The N variables x_i represent an N -dimensional infinite design space in which all values are feasible. In order to find the minimum value of the function g_0 the classical theory of optimization examines all stationary points of g_0 , i.e., solves the set of equations:

$$\frac{\partial}{\partial x_i} g_0(x_i) = 0 \quad i = 1, \dots, N \quad (5)$$

The minimum must be one of the set of possible solutions of (5) and it can be found by substitution of all solutions of (5) into (4), the lowest result being chosen. However, there are very many circumstances under which this classical approach just does not work. Typically, it does not work if $g_0(x_i)$ is a non-analytic function or if some of the derivatives in (5) are discontinuous. The science of optimization theory stems from the very frequent failure of the classical approach to solve problems.

In the absence of a successful classical approach to problem (4) a logical method of finding the minimum value of g_0 would seem to be to evaluate g_0 at a series of trial sets of values of the variables and to numerically select the lowest result. This can be done by imposing a 'grid' over the design space and evaluating g_0 numerically at each grid intersection point. However, if the likely range of each variable x_i is divided into equal divisions so as to give 10 trial values for each variable the total number of values of g_0 which must be evaluated is 10^N . If there are many variables, i.e., if N is large, this number is very large and the method uses too much computer time. A random search in which trial points are selected according to a statistically random sequence is likewise inefficient because of the large number of trial evaluations of g_0 which must be made even to locate a point near to the optimum.

A much better search strategy is to try to ensure that each trial evaluation of g_0 is made according to a set of rules which give a good likelihood that g_0 will be reduced. A vital concept in this context is that of the gradient of g_0 . The gradient of g_0 with respect to some variable x_i is simply the first partial derivative of g_0 for x_i , i.e.,

$$\frac{\partial}{\partial x_i} g_0(x_i)$$

Obviously if the gradient of g_0 for variable x_i is negative then if the value of x_i is increased g_0 can be expected to decrease. If, at each new evaluation of $g_0(x_i)$ all the first partial derivatives can also be evaluated then a new trial point at which the value of g_0 could be expected to decrease can easily be found by either increasing or decreasing each value of x_i according to the sign of its partial derivative. Very many numerical search strategies are based upon using gradient information to produce a new search direction in which new trial evaluations may be made with maximum likelihood that g_0 will decrease.

The steepest gradient method is one such strategy. The N gradients of g_0 at a particular trial point represent an N -dimensional plane which is exactly tangential to the surface of g_0 at the trial point. The steepest gradient method finds that direction upon the tangent plane in which the slope of the plane is maximum. Then by placing a new trial point somewhere along this direction in a decreasing sense g_0 can be expected to decrease in value by more than if the trial were placed in any other direction. The steepest gradient method is a frequently used one in engineering design because of its reliability and its ease of implementation. However, though it always finds a minimum it can often converge very slowly. This is because the tangent plane is essentially a linear approximation to the surface of g_0 which is exact only at the trial point. Once a new trial point is selected, even in the direction of the steepest gradient, at some distance from the original trial point the non-linearity of g_0 may render the tangent plane approximation inaccurate, leading to very slow convergence.

For this reason directions other than the steepest gradient direction are often used to form the basis of a search procedure. Methods based upon conjugate directions are typical of these. Within the scope of this paper it is not possible to examine these methods in detail but their objectives should be stressed. The purpose of such methods is to improve the rate of convergence of methods like the steepest gradient which, although logical, is sometimes very slow.

All methods of search which used gradient information as well as trial evaluations are collectively termed first-order methods because they require first partial derivatives of g_0 . In order to improve upon the efficiency of these methods many second-order methods have been proposed and used. These methods use the second partial derivatives of g_0 , (i.e., information about the local curvature of g_0) in order to speed convergence. Once again space precludes

further examination of these methods other than to comment that because they use more information about the local behaviour of g_0 they are consequently more efficient. For a very readable account of many different methods for unconstrained minimization reference should be made to the work of Sargent⁴.

Several comments should be made on the general applicability of zeroth-, first- and second-order methods to structural optimization. From the point of view of speed and efficiency second-order methods are obviously strong candidates. However, these methods require the prior and recurring evaluation of not only the function g_0 but also all its first and second partial derivatives. This can be very time consuming. If g_0 is a function of 50 variables x_i , $i = 1, \dots, 50$, then there is at each trial point a single function evaluation, 50 first partial derivative evaluations and the matrix of second partial derivatives requires the evaluation of 2500 elements. The computer time and space required for preparatory work and general housekeeping operations can be considerable in the implementation of second-order optimization methods. First-order methods also suffer from this criticism but less so. Carpenter and Smith⁵ have compared the performance of zeroth-, first- and second-order methods on a selection of simple structural optimization problems by the SUMT method⁸. This comparative study is valuable as it brings out very clearly the advantages and the disadvantages of each method. They conclude that on the problems they examined the first-order method of Fletcher-Powell⁶ was preferable for larger problems, Newton's second-order method for small analytic problems and the behaviour of Powell's method⁷, a frequently used one, was generally poor. However, it is also fair to comment that although this sort of information is very useful it is inevitably problem-dependent. For some problems the first and second partial derivatives may be easy to obtain while for others they may be obtainable only by numerical difference techniques which can be very laborious. The choice of methods is therefore a complex one but, bearing in mind that Carpenter and Smith compared only three methods which have been available for at least ten years, their conclusions form a very useful guide.

4.2 Penalty Function Methods

The above section on unconstrained methods is necessary for an understanding of penalty function methods which solve a constrained problem by means of a sequence of unconstrained problems. Penalty function methods have been widely used in structural optimization and are among the more popular methods. Consider the equality-constrained problem:

$$\begin{array}{ll} \text{Minimize} & g_0(x_i) \\ \text{Subject to} & g_j(x_i) = 0 \end{array} \quad \left. \begin{array}{l} i = 1, \dots, N \\ j = 1, \dots, M \end{array} \right\} \quad (6)$$

The penalty function approach replaces problem (6) by the unconstrained problem:

$$\text{Minimize } F = g_0(x_i) + \sum_{j=1}^M P_j [g_j(x_i)]^2 \quad i = 1, \dots, N \quad (7)$$

In which values of P_j , $j = 1, \dots, M$, are positive constants. The function F is therefore composed of the original objective function g_0 plus the value of each constraint multiplied by a penalty factor P_j . Starting with some known set of factors P_j , F is minimized using unconstrained techniques. All values of P_j are then considerably increased and another unconstrained minimization of F is performed. This process continues for increasing values of the penalty factors and has the effect of forcing each of the constraint functions g_j towards the value zero. Thus as values of P_j are increased the results of the sequence of unconstrained minimizations of F tend towards the solution of problem (6).

Penalty function methods are also applicable to inequality constrained problems. If the constraints g_j in problem (6) are written as:

$$g_j(x_i) \leq 0 \quad j = 1, \dots, M \quad (8)$$

a suitable interior penalty function problem is:

$$\text{Minimize } F = g_0(x_i) - P \sum_{j=1}^M [g_j(x_i)]^{-1} \quad i = 1, \dots, N \quad (9)$$

where P is a positive penalty factor. Starting with a large value of the penalty factor P an unconstrained search is carried out from a feasible starting point, (i.e., values of x_i which do not violate any of the constraints - hence an interior or feasible point). The solution of this search cannot be at a point which causes any constraint value g_j to be zero otherwise F would be infinite. This time the effect of P is to keep the solution away from the constraint boundaries. Further unconstrained searches are carried out using a sequence of decreasing values of P thus the process can progressively approach any constraint boundary, where $g_j = 0$, if it wishes or can remain feasible, $g_j < 0$, if this is advantageous. The results of the sequence of problems thus converges to the minimum of g_0 with constraints given by (8).

Much work has been published on penalty function methods and both interior and exterior methods (the search is always in the infeasible region) have been widely studied. A popular method is the SUMT method of Fiacco and McCormick⁸ to which reference should be made. Lootsma⁹ has given a comprehensive review of the topic and its use in structural optimization has been championed by Fox¹⁰ and Moe¹¹ in particular.

The chief disadvantage of the approach is that although it converts constrained optimization problems to the much simpler unconstrained form, it requires a considerable amount of time to solve the unconstrained problem many times. Furthermore the composite unconstrained objective function F contains all the constraint functions g_j and since we have seen that structural optimization problems usually have many constraints F can be very large. In solution of the unconstrained minimization if first-order or second-order methods as described in section 4.1 are used, partial derivatives of all the constraint functions g_j must be evaluated since they appear in F . Thus considerable time is necessary for the evaluation of derivatives and for this reason the penalty function approach cannot really be deemed suitable for large structural optimization problems. It has been used very effectively on the detailed design of structural elements or components such as beams, plates, panels, etc., where the number of variables is perhaps a maximum of 15 and the number of constraints is of the same order. For more complex problems its efficiency can sometimes be rather poor.

4.3 Constrained Numerical Search

In section 4.1 methods of unconstrained numerical search were considered. Here the feasible region is infinite. When constraints are present, however, they limit the feasible region which is hedged around by constraints which must not be violated. The only way of knowing whether a particular trial point is or is not feasible is to evaluate all constraints at that point and check them for violation. If methods of gradient search are to be used it is likewise vital to know if a particular search direction points into an infeasible region. This too involves checking all constraints and the derivatives of constraints. It is obvious, therefore, that the presence of many non-linear constraints which is characteristic of structural optimization problems causes considerable difficulties for any numerical optimization method. The success and efficiency

of any search technique depends upon the nature of the constraints.

The simplest case is where all constraints are linear equalities. Here as many variables may be eliminated (expressed in terms of other variables) as there are equality constraints and the problem is then reduced to an unconstrained one in a reduced number of variables. The linear equality constraints positively help the solution process. When the problem is constrained by linear inequalities slack variables may be added to convert the constraints to equalities and the solution can then be carried out as above by unconstrained methods. Linear constraints can therefore be handled quite easily and efficiently.

In structural optimization the constraints are usually and unfortunately non-linear inequalities. It is the author's opinion that no numerical search method for non-linear inequality constraints has yet been developed which can be advocated on grounds of reliability and efficiency as suitable for anything but the smallest structural optimization problems, i.e., about 5 - 10 variables and a similar number of constraints. It is almost always possible to solve problems more efficiently by the penalty function approach of section 4.2 or by the methods to be outlined in the following sections. Many methods have been proposed and they founder generally on the need for many trials and derivative evaluations. These are necessary because of the difficulty experienced in locating and following a non-linear boundary in N-dimensional space. The only direct search method for non-linear constraints which could possibly be an exception is that based upon the Simplex method of Nelder and Mead¹² which is a zeroth-order method making trial evaluations at the vertices of a N-dimensional regular figure which 'spins' through the feasible design space.

4.4 Linear Programming

Throughout this paper it has been emphasized that structural optimization problems are generally highly non-linear. There is, however, a major exception to this generalization which arises in the optimum plastic design of structures. Consider the beam/column framework of a multistorey building such as that shown in Fig. 2. The design of such a framework on a fully-plastic basis consists of finding a set of fully-plastic moments M_p for all members of the framework so that a prescribed factor of safety against collapse is achieved. The optimum design is one in which the set of M_p values also minimizes the weight or cost of the frame. For a frame of given layout member lengths are known and it is possible to approximate the cost function for the frame as a linear function of the M_p values of all the members. It should be noted that this is only an approximation but is a very reasonable one to make in order to solve the design problem which is in category 3 of the hierarchy described in section 2.

The constraints upon the problem are those of structural mechanics: It is necessary to ensure that in any possible collapse mode the work done by the factored applied loads does not exceed the energy capacity of the rotations at plastic hinges in the frame. This requirement leads to constraints in which linear functions of the M_p values are bounded above by a known set of constants. For completeness there should be one linear constraint for each possible collapse mode. In this problem, then, both the objective function and all the constraint functions are linear and linear programming may be used to find the optimal set of M_p values.

Optimum fully-plastic design has received much attention in the past since linear programming may often be used for which efficient and reliable computer package programs are available. However, the objective function is not truly linear and from a structural point of view the constraints are incomplete since several non-linear effects have to be omitted. For instance, elastic instability of the frame, the reduction of plastic moment capacity due to axial load, and

change of geometry effects cannot be included. There are also considerable problems involved in ensuring that the actual collapse mode of the frame is present among the constraints. Much research effort has been devoted to overcoming some of these difficulties which is ample testimony to the ease of use and popularity of linear programming.

Linear programming deals with problems similar to problem (1) in which all functions g_j , $j = 0, \dots, M$ are linear. Almost any textbook on optimization or operational research gives several solution techniques so no specific reference is made here. Several important features of linear programming can be mentioned, however. The first is that very large problems can be solved very efficiently. Many thousands of variables and constraints can be handled effectively. It is perhaps for this reason alone that linear programming is popular as a sequential method, (see section 4.5), for structural optimization. It is very worthwhile trying to force a problem into a linear format if possible because very large problems can then be solved. Secondly the duality theory of linear programming has the advantage that it gives insight into the nature of the problem which normally remains obscure. Linear duality may be summarized as follows:

<u>Primal Problem</u>	<u>Dual Problem</u>
Minimize $W = \sum_{i=1}^N C_i x_i$	Maximize $Y = \sum_{j=1}^M b_j \lambda_j$
Subject to	Subject to
$\sum_{i=1}^N a_{ji} x_i \geq b_j \quad j = 1, \dots, M$	$\sum_{j=1}^M a_{ji} \lambda_j \leq C_i \quad i = 1, \dots, N$
$x_i \geq 0 \quad i = 1, \dots, N$	$\lambda_j \leq 0 \quad j = 1, \dots, M$

Thus each primal linear programming problem in variables x_i has a dual problem in variables λ_j such that the solution of either one is exactly equivalent to solution of the other. A problem with few variables and many constraints has an equivalent dual problem with many variables but few constraints, and so the easier of the two problems may be solved.

Space precludes any further consideration of duality other than to mention that the physical interpretation of dual variables for a real-world structural design problem remains a very fertile area of research. Many authors^{13,14} have examined linear duality for problems arising in fully-plastic design of structures and as a result have contributed greatly to an understanding of the basic behaviour of structures. As a final advantage of linear programming it should be noted that restrictions that variables take integer values or values from a discrete set can be accommodated by linear programming in a rigorous manner. This does not generally apply to other methods, apart from Dynamic programming.

4.5 Sequential Linear Programming

Because linear programming is an efficient and reliable technique which can solve very large problems it has been used widely in a sequential manner for the solution of non-linear problems. The way this is done is as follows. Consider problem (1) with all functions g_j , $j = 0, \dots, M$ being non-linear. Take a feasible trial point $\bar{x}_i$, $i = 1, \dots, N$, and evaluate at this point the values of all functions g_j and the values of all first partial derivatives of all functions g_j . Each of the functions g_j may then be replaced by a linear approximation which has the same value and gradients at the trial point. The

approximating functions can be easily derived from a first-order Taylor series expansion about the trial point. The resulting approximate problem is now a linear programming one and may be solved as such. The result of this linear programming problem then gives a new trial point at which a new approximating linear problem can be constructed. The solution of problem (I) is then approached by means of a sequence of linear programs.

The sequential linear programming method can be criticised in several ways. Firstly, there is no proof that the sequence of linear approximations will converge to the optimum of problem (I). Conceptually, convergence is often assumed but if the degree of non-linearity in the original problem is high convergence of the sequence may be very slow. Also, a linear approximation to a non-linear problem may be very inaccurate and the results of each linear program may be highly infeasible for the original non-linear problem. In order to ameliorate this move limits are often used which add extra constraints to prevent the linear search from going too far into infeasible regions. This increases the number of cycles of iteration and has the effect of imposing convergence to a point which is not necessarily the optimum of problem (I). The use of linear programming implies that the optimum will always be found at a vertex of the linear constraints but this is not necessarily true of general non-linear problems.

However, despite these criticisms sequential linear programming remains a very popular method of structural optimization and although its performance sometimes leaves much to be desired it has been successfully used on a very wide range of large and complex structural design problems. Pope¹⁵ has given a good description of sequential linear programming in AGARDograph 149 which is entitled 'Structural Design Applications of Mathematical Programming Techniques' and gives excellent background material in the area of structural optimization. A big advantage of the SLP method is that it can tackle large and complex problems unsuited to any other technique. It has frequently been linked to finite element analysis programs to give an iterative optimum design capability. In such cases considerable amounts of computer time are necessary and the final results are sometimes only approximate optima but this method is the only way in which complex problems can be solved. Examples in the field of structural design are many but two may be referred to as they typify the performance of the method on very complex problems^{16,17}.

4.6 Dynamic Programming

Dynamic programming solves a very special class of problems in which the objective is to extremize the performance of a serial system.

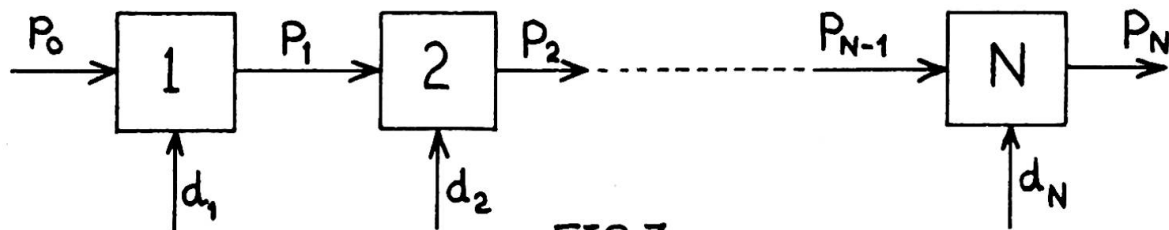


FIG.3

In Fig. 3 each box represents a stage in the serial system. Consider box 1. Input to this stage is the value of the performance p_0 . By giving different values to the decision variable d_1 the output performance p_1 of stage 1 is varied. This output p_1 of stage 1 is the input to stage 2 and the system performance at the end of stage 2, p_2 , is modified from the input value p_1 by decision variable d_2 . By the final stage N , the final performance criterion p_N is therefore a function of the initial performance p_0 and the N decision variables

$d_1, \dots, d_N$. The objective is to find values for p_0 and all the N decision variables which extremize the final performance p_N . Dynamic programming is a very efficient technique for solving such problems and was developed by Bellman¹⁸.

The dynamic programming method itself is not studied here but some characteristics of the problem should be noted. First of all it is necessary to have a serial system such as shown in Fig. 3 in which decisions taken at a particular stage affect only performance after that stage and not before it. No 'looping-back' is possible. Although Fig. 3 shows only one decision variable at each stage it is possible, though more expensive in time, to have multiple decisions at each stage. The reason that dynamic programming is mentioned in connection with structural optimization is that it is very rapid and efficient in solving problems which fall into the suitable class. Unfortunately very few structural optimization problems have the necessary sequential decision characteristics. Palmer^{19,20} has applied the method to optimizing the geometry of transmission towers and frameworks with some success. As a general comment upon the dynamic programming method for structural optimization it can be said that it works very well indeed on suitable problems but few structural problems are suitable.

4.7 Geometric Programming

Geometric programming is perhaps the most recently developed mathematical programming method to appear on the structural optimization scene. Like linear programming and dynamic programming, geometric programming solves a particular class of problems but unlike these other methods the class of suitable problems is quite large and many optimum structural design problems are suitable. Geometric programming in its simplest form solves the problem:

$$\begin{array}{ll} \text{Minimize} & g_0(x_i) \\ \text{Subject to} & g_j(x_i) \leq 1 \\ & x_i \geq 0 \end{array} \quad \left. \begin{array}{l} i = 1, \dots, N \\ j = 1, \dots, M \\ i = 1, \dots, N \end{array} \right\} \quad (10)$$

with the restrictions on the mathematical form of the functions g_j that:

$$g_j(x_i) \equiv \sum_{t=1}^{T_j} C_{jt} \prod_{i=1}^N x_i^{a_{jt+i}} \quad j = 0, \dots, M \quad (11)$$

$$\text{and } C_{jt} > 0 \quad \text{for all } j, t \quad (12)$$

The class of suitable problems is therefore governed by the requirement that all functions be sums of terms, each term involving products of variables raised to known powers. Generally all constraints are non-linear inequalities and such problems can be very difficult to solve directly. Geometric programming does not attempt direct solution but uses theorems of geometric duality, described by Duffin et al²¹ to construct an equivalent dual problem. Using dual variables δ the geometric dual problem exactly equivalent to (10), (11) and (12) is:

$$\text{Maximize } V(\delta) = \prod_{j=0}^M \prod_{t=1}^{T_j} \left(\frac{C_{jt} \lambda_j}{\delta_{jt}} \right)^{\delta_{jt}} \quad \left. \right\}$$

$$\begin{aligned}
 &\text{Subject to the constraints } \sum_{t=1}^{T_0} \delta_{0t} = 1 \\
 &\sum_{j=0}^M \sum_{t=1}^{T_j} a_{j+t} \delta_{jt} = 0 \quad i = 1, \dots, N \\
 &\text{in which } \lambda_0 = 1 \\
 &\lambda_j = \sum_{t=1}^{T_j} \delta_{jt} \quad j = 1, \dots, M \\
 &\delta_{jt} \geq 0 \quad \text{all } j, t
 \end{aligned} \tag{13}$$

The important feature of the geometric dual problem (13) is that the constraints are linear equalities. As was mentioned in section 4.3 linear equality constraints actually assist in maximizing $V(\delta)$ and methods of unconstrained search can be adapted to solve the dual problem numerically. The geometric programming method therefore uses duality theorems to convert problems of a difficult type to solve directly into problems with linear equality constraints which are much easier to solve. Since the appearance of Duffin, Peterson and Zener's book²¹ in 1967 the method has been considerably extended and requirement (12) that all coefficients must be positive has now been removed in more recent versions of the method. Also constraints of reversed sign can be handled. Templeman and Winterbottom²² have summarized these recent developments in a paper which describes a computer program for geometric programming and its application to structural design.

Advantages of the method are that it is a non-linear one which works effectively on highly non-linear problems. The class of suitable problems is clearly defined and is as easy to recognize as linear programming. A standard format of problem input is available which makes solution by means of computer package programs relatively easy. It has been shown that very many structural optimization problems are naturally suitable for solution by geometric programming. The detailed design of almost all types of structural elements (category 4 of the hierarchy described in section 2) can be expressed as geometric programming problems. Equations (2) and (3) show that the physical properties of a beam or column can be related to a single variable, A , raised to different powers. Terms such as these typify category 4 problems and they are in the form of (11) which is ideal for geometric programming. Templeman²³ gives several examples of the optimum design of structural components such as beams, corrugated plates and integrally-stiffened compression panels and also shows how geometric approximations may be made to more general problems in structural optimization. Non-standard problems can then be solved by a converging sequence of geometric programming problems in a conceptually similar way to sequential linear programming as described in section 4.5. This has the potential advantage that more representative non-linear approximations rather than linear ones are made to non-linear problems. A final advantage of the method is that the dual problem sheds new light on the optimum design and much insight can be gained by attempting to interpret physically the dual of a structural design problem. This point was made previously in connection with linear duality and it seems likely that research now being carried out in several centres into interpretation of primal/dual systems could in the future be of great benefit, leading to new structural optimization techniques and enhanced understanding of the design process and of structural behaviour.

Finally, the disadvantages of the geometric programming method should be mentioned. Firstly, it is the most difficult to understand of the methods described here. Secondly, if a computer package is not readily available to a prospective user of geometric programming then the effort involved in programming the method is very large. At present there are only a few suitable computer programs available. Thirdly the method is still relatively in its infancy, its use is not widespread and much of its great potential is still to be realized. Finally, the performance of the method on very large non-linear problems is dominated rather by computer storage and hence also by run-time. Factual evidence of this is as yet small but it may be that there is a fairly high limit upon the size of geometric programming problems which can be solved within normal economical limits.

5. OPTIMALITY CRITERION METHODS

All the methods described in section 4 attempt to solve structural optimization problems of the general form of problem (1) by mathematical and numerical search methods. The philosophy adopted is that nothing is assumed about the location or nature of the optimum. The optimum is reached by some purely numerical search which is based upon the mathematical form of problem (1) rather than the real-world structural design which problem (1) represents. The structural engineer, however, sometimes feels that he knows much more about the optimum structure than is present in its mathematical equivalent and that this knowledge might be useful in deriving a search method based upon structural principles rather than upon a mathematical abstraction. The term optimality criterion methods covers such approaches. Optimality criterion methods solve problems such as problem (1) by search methods so strictly this section could be numbered 4.8 and considered with all the other methods. The concept is so different, however, as to warrant a separation from the other methods.

Optimality criterion methods are absolutely problem-dependent and a particular criterion applies only to the optimum design of a particular type of structure under very specific conditions. The philosophy of the optimality criterion approach is first of all to investigate the nature of optimum structures of some specific type to try to establish a condition or set of conditions satisfied only by the optimum structure and which are not satisfied by any other, non-optimal design. For example such conditions might be that for a particular type of structure under restrictions on stresses and displacements the optimum structure always has a recognizable distribution of some form of energy among its components and that this distribution is peculiar only to the optimum design. Having found some structural criterion of optimality it is then necessary to devise some iterative algorithm which, starting with a non-optimal structure, will successively redesign the structure so that a structure which satisfies the optimality criterion and hence is optimal will be found.

For particular classes of structures the optimality criterion approach solves the mathematical problem (1) very indirectly by completely replacing the problem by an analogous one of iteratively redesigning a structure so that it satisfies some pre-established criterion of optimality. In a sense optimality criterion methods are dual methods in that problem (1) is solved by solving a completely different but equivalent 'dual' problem. The nature of this duality between mathematical programming and optimality criterion methods is almost totally unexplored but holds out considerable promise for further research work.

In developing an optimality criterion method for a particular class of structures there are two distinct phases. First of all a relevant and unique criterion of optimality for the class of structures must be found. This is generally very difficult to do since such criteria are rarely obvious. Had optimality criteria been obvious then mathematical programming would never have

been necessary for optimum structural design. It is the requirement that the criterion be unique to the optimum structure which is perhaps most difficult to satisfy and indeed at the present time the technical literature holds examples of many so-called optimality criteria which are not optimal at all in that they can also be satisfied by non-optimal structures. The second phase is that of developing a recursion relationship which will produce an iterative redesign algorithm so that the optimality criterion can be satisfied. This too can frequently be difficult to develop and it runs the risk like any numerical search method of being unwieldy to operate or slow to converge. The optimality criterion approach is a logical one which has great appeal to structural engineers since it is based upon structural rather than mathematical principles. It holds great promise for the future although it can never entirely replace mathematical programming methods but, like some of the methods described earlier, it is still in its infancy and it still has to realize much of its potential.

At present very few rigorous, well-tested optimality criterion methods exist although many have been proposed and the range of optimum structural design problems which they cover is small. Generally they apply to overall sizing of structural members in a multimember system such as a truss or frame, i.e., category 3 problems in the hierarchy of section 2. Some authors, notably Prager²⁴, have developed optimality criteria for structural components such as beams, sandwich plates, etc. For such structures the optimality criterion generally is concerned with energy distributions in the parts of the structure. Prager has concluded that volume integral of energy density in each part is proportional to the volume of the part in many cases with single constraints. When multiple constraints are present the optimality criterion becomes more complicated but this energy distribution pattern is still an optimality criterion although in a modified form. Variations upon this energy-density optimality criterion have been proven to be applicable to truss structures, notably by Venkayya²⁵ and many others. Indeed many types of structures may be designed by optimality criteria methods with constraints upon stresses or displacements or dynamic stiffness, and the big potential advantage which the optimality criterion approach has is that very large problems can be designed this way, given a suitable criterion and algorithm, whereas mathematical programming methods are often suitable only for relatively smaller structures. A major difficulty is encountered when structures are to be designed subject to multiple constraints of different types. For example, a truss may be required to satisfy both stress constraints for which a criterion is available and multiple displacement constraints for which another criterion is known. Both criteria are not usually satisfied simultaneously and although a composite optimality criterion can be devised it is considerably more difficult to devise a rapid redesign algorithm and the solution process can be very slow.

There are therefore many difficulties associated with the optimality criterion approach and these are not always immediately obvious. The idea of developing structurally based rather than mathematically based optimization methods is, however, very appealing and holds out great hope for the future but much more work remains to be done in order to realize this potential.

6. CONCLUSIONS

Structural optimization is at present a thriving area of research and development. The philosophy is so very obviously right since the structural engineer has historically been guided by the need and desire to produce structures which are in some respect 'better' or more efficient than those which have gone before. The electronic computer has enabled design and optimum design to be put on a more formal and rigorous basis and is the means by which the goals of structural optimization may be achieved. However, it is fair comment to say that the actual methods of optimization at present available are not entirely

adequate for very many of the structural optimization problems which surround us today. Mathematical programming methods can sometimes be laborious in operation and are often restricted to only small to medium sized problems. Some methods offer more promise for the future than others and mathematical programming is still very much alive. In the author's opinion better methods will be developed in the future perhaps based upon duality. Optimality criterion methods have great appeal to the structural engineer yet at present they too are in their infancy and require much more development.

In section 2 of this paper a hierarchy of structural optimization problems was discussed. Research up to the present time has tended to be concentrated mainly in categories 3 and 4 of this hierarchy - the easier problems. Categories 1 and 2 are as yet little-explored and the potential rewards offered for methods of solving such problems are very great. By about 1970 a watershed had been reached in structural optimization. Prior to this date research had concentrated upon marrying existing solution methods to structural design problems and it became evident that the marriage was only partially successful. Very many pressing problems remained to be solved. Since 1970 progress in structural optimization has been along new lines of approach. The simpler problems are now things of the past and only the harder ones remain. It is significant that interest in these harder problems of structural optimization is unabated. Over the last five years many new lines of approach have been opened up and although progress has often been slow the full potential of the new methods can now be clearly seen as a future goal. To achieve this potential is the object of structural optimization today.

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SUMMARY

The paper considers the economic objectives of structural optimization and shows that it has put design on a formal and rigorous basis. The size and type of structural optimization problems is then examined and a hierarchy of problem categories is discussed. Approximation methods are considered which enable complex structural problems to be posed concisely. Seven mathematical programming methods are described and critically discussed in the context of a specification for a suitable optimization technique for engineering use. Optimality criterion methods are then examined and the paper concludes that presently available structural optimization techniques have yet to achieve their full potential.

RESUME

L'article traite des buts économiques de l'optimisation structurale et montre que le dimensionnement est posé sur une base formelle et rigoureuse. Le type et l'importance des problèmes d'optimisation structurale sont examinés; une hiérarchie des catégories de problèmes est discutée. Des méthodes d'approximation permettent de poser des problèmes structuraux complexes de façon concise. Sept méthodes de programmation mathématique sont présentées et comparées dans le cadre de directives pour une technique d'optimisation appropriée à l'usage de l'ingénieur. Des critères de méthodes d'optimisation sont discutés. L'article conclut que les techniques actuelles d'optimisation structurale peuvent encore être améliorées.

ZUSAMMENFASSUNG

Der Beitrag behandelt die wirtschaftlichen Aspekte bei der Optimierung von Tragwerken und zeigt, dass die Bemessung auf einer formalen und strengen Grundlage beruht. Grösse und Typen baulicher Optimierungsprobleme werden untersucht und eine Rangordnung der Probleme diskutiert. Es werden Näherungsmethoden untersucht, welche gestatten, komplexe bauliche Probleme rasch zu lösen. Sieben mathematische Programmierungsmethoden werden beschrieben und hinsichtlich des Anwendungsbereiches kritisch verglichen. Sodann werden verschiedene Optimierungskriterien untersucht; der Beitrag schliesst mit der Feststellung, dass die gegenwärtig erzielte Optimierungstechnik ihre vollen Möglichkeiten noch nicht erreicht hat.

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

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

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

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1. INTRODUCTION

Many discussions in the I.A.B.S.E. proceedings have considered structural optimization. For example, Professor Courbon defined optimization as designing and construction a structure at the lowest cost, with the object of fulfilling a well defined purpose.(1) In particular, cost consideration must also be given to safety, service life, maintenance and future adaptability. Since all of research and practice in structural engineering is aimed towards such a goal the activity known as structural optimization must be defined in a unique way. That is, the development and application of a priori directed and automated techniques for improving designs within well defined cost contexts and recognized constraints. Within this definition, therefore, questions of design creativity and ingenuity are put aside in favor of quantitative comparisons among a vast array of acceptable yet competing designs. Thus, in much the same way as computer methods of structural analysis, the techniques of structural optimization become an aid to the designer for rapidly proportioning structural details and evaluating design alternatives to obtain the best design among given choices. In this way, when the engineer arrives at comparing quite different conceptual designs for the same application he is fairly certain of intelligently comparing these alternatives and not unfavorably biasing one alternative by poor proportioning of its details.

In the field of structural optimization the computer becomes central as a tool for searching and sorting through the similar design concepts and proportioning the element details for the most economical design. Naturally, it arrives at a design which the engineer could equally have obtained if he were prepared to invest the time and money to directly

search among the design alternatives. The principal advantage of the optimization methods is therefore its saving in design time and cost. A further consideration with the expanding usage of computer analysis programs to calculate structural behavior is that an optimization program can eliminate much of the input-output and the costly data handling effort. Since the methods result in a proportioned structure which satisfies applicable codes it becomes unnecessary to review the vast amounts of structural analysis output. Instead, the optimization program produces design details as an output and in some cases could be further programmed to produce drawings and material and fabrication specifications.

2. RAISING THE DESIGN HIERARCHY

Since structural optimization over the last decade and a half has successfully concerned itself with computer aided proportioning of design details it is easy to ignore further applications. In fact, some recent work has shown the possibility of introducing into the automated computer procedures design variables which had previously been thought to be either in the realm of creative decisions or else difficult to program for computer selection. These additional design variables have described geometry and shape of structures, material choice, complete building design including comparison of basically different element types and design selections including overall fabrication costs and material availability. Before embarking on a description of several such examples it is worth considering from this framework the historical developments of optimization applications.

There is in this regard an analogy between computer developments in both structural analysis and optimization. When digital computers first became available civil engineers who were among its early intensive users simply programmed classical methods of analysis such as slope deflection and moment distribution. In a similar way the first structural optimization applications were programs using such well known iterative design methods as structural index, stress-ratio, fully stress and other optimality criteria.(2)

A second development of computer applications were matrix analysis

for specific structural types such as trusses, frames and grillages. Paralleling this was specialized optimization programs for these same structural types using methods such as gradient directions or other heuristic design search procedures.(3) The current stage of development includes general purpose computer programs usually available from machine manufacturers or time-share agencies. In analysis this often means finite-element packages for linear and non-linear behavior. The same evolution for design has been program packages each able to handle a wide array of different structural elements and systems. It is important to investigate in detail these current developments in optimization since they include the methodology for extending applications in both structural system and geometry optimization.

3. GENERAL PURPOSE OPTIMIZATION PROGRAMS

To explore the available methodology for structural optimization the author prefers to divide iterative structural decision problems into three categories. These are: 1) element design, 2) interconnected structural systems and 3) discrete decision variables. Each of these categories will now be considered. It is recognized that other areas could be added particularly as we move further into conceptual and creative decision variables but the three categories will suffice to cover the needs of a computer optimization library for a typical design practice.

Element Design - This design problem is characterized by a well defined code of practice for the constraints and a relatively direct method of calculating the element loading such as moment, shear, torsion and axial load. Some examples of application are shown in Figure 1 including welded wide flange, unsymmetrical box girder and prestressed beams.(4-6) Design variables are typically depths, thickness, shape, reinforcement ratios and other design details which are often part of the tedious aspects of design and for which economic selection rules are not always available. Other published examples include welded columns, gabled frame, stiffened ship plates, shear walls, prestressed plates and reinforced concrete beams.(7-11) These examples typically have a small number of independent design parameters (say less than 10)

but relatively complex functional code constraints such as allowable tension, compressive and shear stresses, buckling and displacement constraints. In one example using the A.I.S.C. code the constraints were due to lateral buckling and were discontinuous representing a transition from elastic to inelastic behavior.(4) In some applications it becomes necessary to repeat the design for a large number of different elements. For example, in the case of box girder sections a particular overhead crane manufacturer using such sections needed over 5000 specified designs.(5) This obviously required an efficient design program.

Several computer packages have been developed to automate the solution to element optimization. The author prefers to use programs based on the penalty technique which combine the criteria function (cost or weight) and the constraint into a single expression to be minimized.(12) This transforms the more difficult non-linear programming problem with cost and constraints into a more tractable unconstrained minimization for which many straightforward solution methods are available. Other methods have also been successfully used on some problems including geometrical programming and linear programming.(13) As was mentioned earlier, element design is also characterized by a direct calculation of the forces on the elements. This may either be for statically determinate structures or even for complex frameworks in which a matrix structural analysis is used to solve for element forces. It is assumed in the latter case that the element size does not affect the force distribution within the structure; if this effect is significant then several cycles of iteration may be necessary to converge both the force distribution and element design. Since an element design optimization usually involves comparing many alternatives it would be computationally difficult to repeat the force analysis each time a design parameter were changed.

System Optimization - This refers to structures where there is a major physical interaction between different elements or there exists design constraints based on total structural behavior such as stability, stiffness, vibration and dynamic responses. Furthermore, system optimization arises when some of the design variables relate to more than

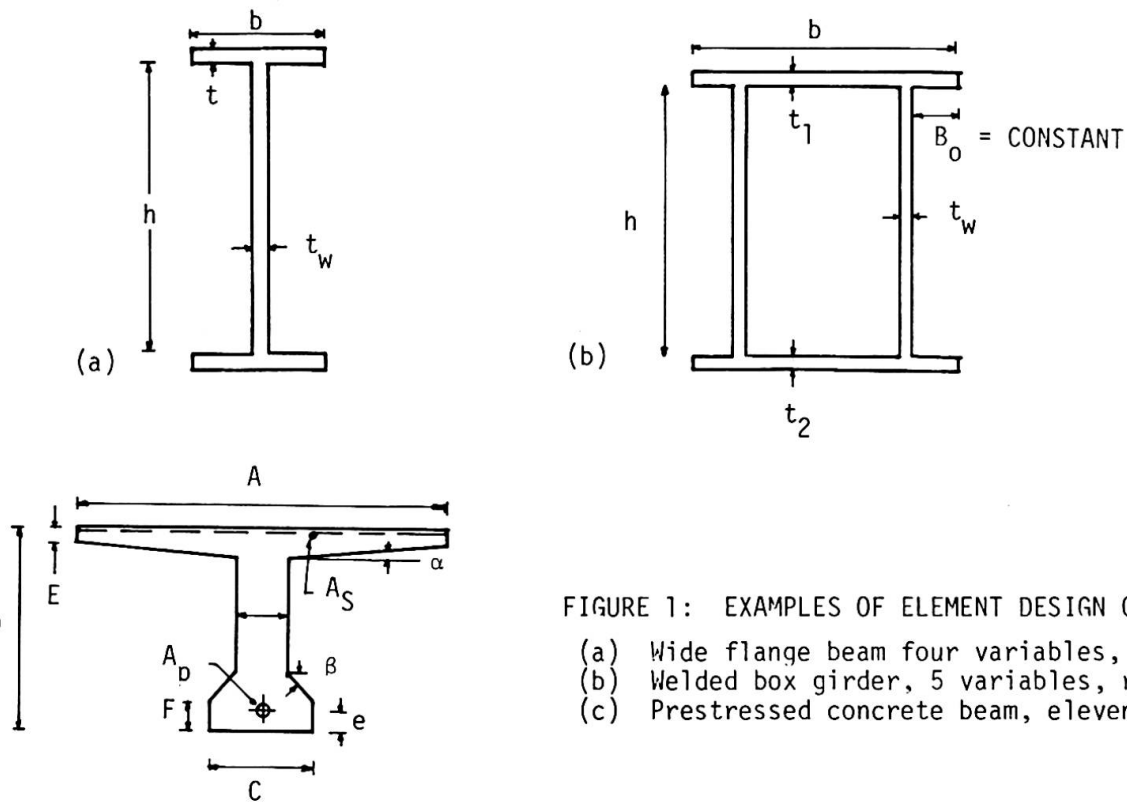


FIGURE 1: EXAMPLES OF ELEMENT DESIGN OPTIMIZATION

- (a) Wide flange beam four variables, ref. 4
- (b) Welded box girder, 5 variables, ref. 5
- (c) Prestressed concrete beam, eleven variables, ref. 6

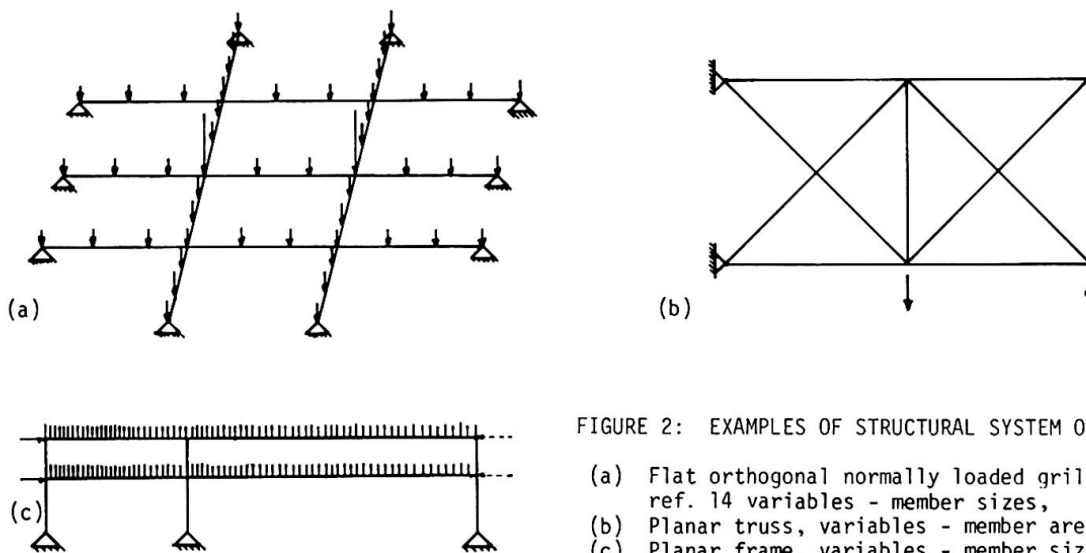


FIGURE 2: EXAMPLES OF STRUCTURAL SYSTEM OPTIMIZATION

- (a) Flat orthogonal normally loaded grillage, ref. 14 variables - member sizes,
- (b) Planar truss, variables - member areas, ref. 15
- (c) Planar frame, variables - member sizes, ref. 16

one element such as geometry, material or topology. In these cases, the force distribution invariably requires a matrix or finite element analysis. Computer costs usually restrict the number of possible trials that the computer can sequentially examine as it searches for the optimum to often under 10 trials. Some cases that have been optimized include foundation or ship grillages, trusses and frames and are illustrated in Figure 2.(14-16) The most efficient search techniques for systems usually utilize some form of linear programming steps since in this way large changes in design may be made with only a single analysis interaction.(16,17)

Discrete Selection Parameters - This category of optimization is considerably more complex than either the two first categories. Discrete design variables are not always clearly defined or constant in number and the cost function may be extremely complex and discontinuous. The discrete nature of variables often requires heuristic or intuitive search methods of solution which make general purpose programs irrelevant. One method however, which has solved a variety of such problems is the dynamic programming technique which is easy to program if the basic problem formulation meets its definition.(18) Among the reported structural design problems solved by dynamic programming are the minimum cost of continuous coverplated highway bridge girders, single story building selection of different roof, column and foundation elements, spacing of supports of multi-span girders, thickness variations in ship plate components, reinforcing bar arrangement in continuous reinforced concrete beams and girder selection for minimum material, detailing and fabrication costs.(19-24) These examples have in common discrete variable selection and more important a sequencing of decisions into stages which satisfy the dynamic programming criterion. As an example in Figure 3, the single story optimization starts separately with the roof and then includes the column and finally the foundation and bay spacing. Since roof cost is independent of the supporting columns and foundation this sequencing of decisions is possible. The same notion of sequencing is true of the other dynamic programming examples mentioned.

4. GEOMETRY AND SHAPE OPTIMIZATION

In extending the design optimization hierarchy beyond the variables associated with design details the variables associated with shape and geometry have arisen. This is a natural continuation of much of the early optimization work on truss and frame structures which optimized

the force distribution within the structure or element optimization which finds the best details in a specific cross section. Naturally much greater cost savings can be made by optimization of geometry variables than by member selection alone since the force distribution is relatively insensitive in many cases to the latter variables.

An example of this is illustrated in Figure 4 with a section of a transmission tower.(25) With a fixed initial geometry as in Figure 4a little savings can be realized in structure weight by optimizing the element force distribution. The difference between total optimized weight and say the weight obtained by a traditional direct iterative analysis and design approach is quite small even over a wide array of load conditions and even displacement constraints. However, when geometry variables are introduced the structural weight is reduced 18% as in Figure 4b. The geometry variables were the location of joints and the width of the base support. In this shape optimization the geometry design variables were separated from element design variables. For each change in geometry the minimum element design was found by a direct design method such as stress ratio. The changes in geometry were found by gradient methods.

A broader generalization of this search for optimum geometry or "best shape" structure was reported by Zienkiewicz and Campbell.(26) Starting with a finite-element analysis program an extension was programmed to automate the calculation of derivatives of structural behavior such as stresses and displacements with respect to various structural shape parameters. Optimum changes in geometry were then carried out by linear programming. Applications of this approach have been reported for arch dam geometry, dam cutouts and plates. Vitiello reported a similar program for beam shapes, gravity dams and seismic loading.(27) By properly fitting the structural behavior with polynomial functions of the geometry he was able to do the minimization with a penalty function program. An example showing the gravity design variables and the finite element modelling is given in Figure 5.

Ramakrishnan and Francavilla also using a linear programming approach found optimum shape designs for plates, pressure vessel end closures, and a gravity dam.(28)

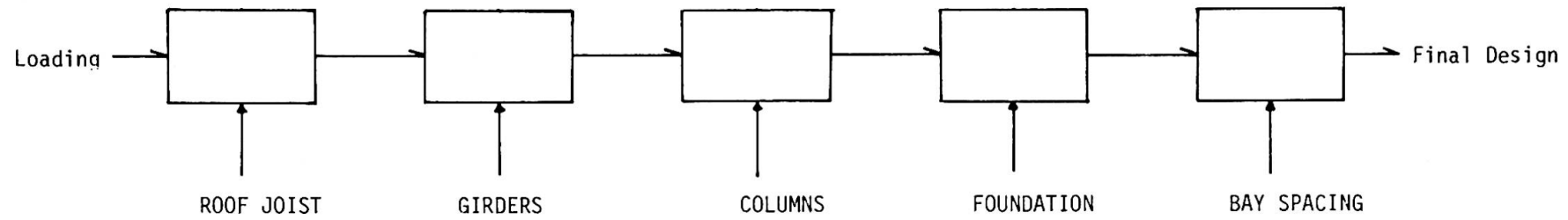
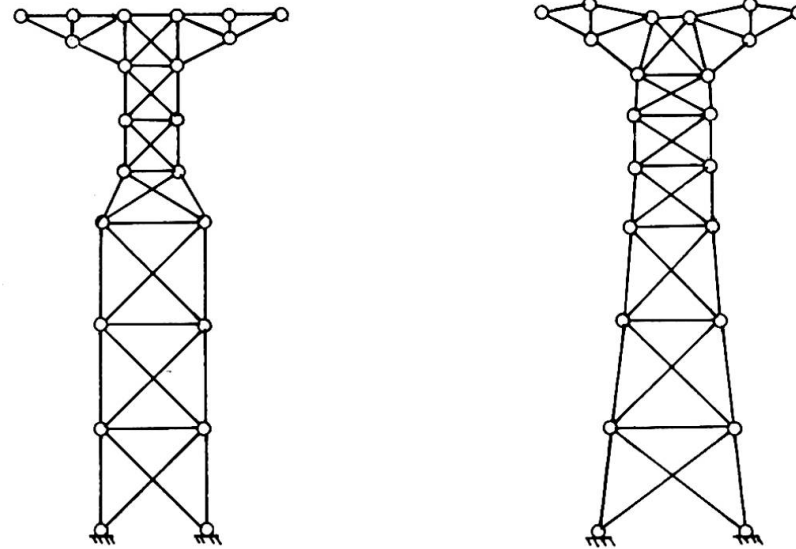


FIGURE 3: DISCRETE DESIGN OPTIMIZATION

Single story building ref. 20 and extensions by the author.



(a) Weight = 2306

(b) Weight = 1910

FIGURE 4: OPTIMAL GEOMETRY DESIGN OF TOWERS (ref. 25)

- (a) Starting Design
- (b) Optimal Geometry

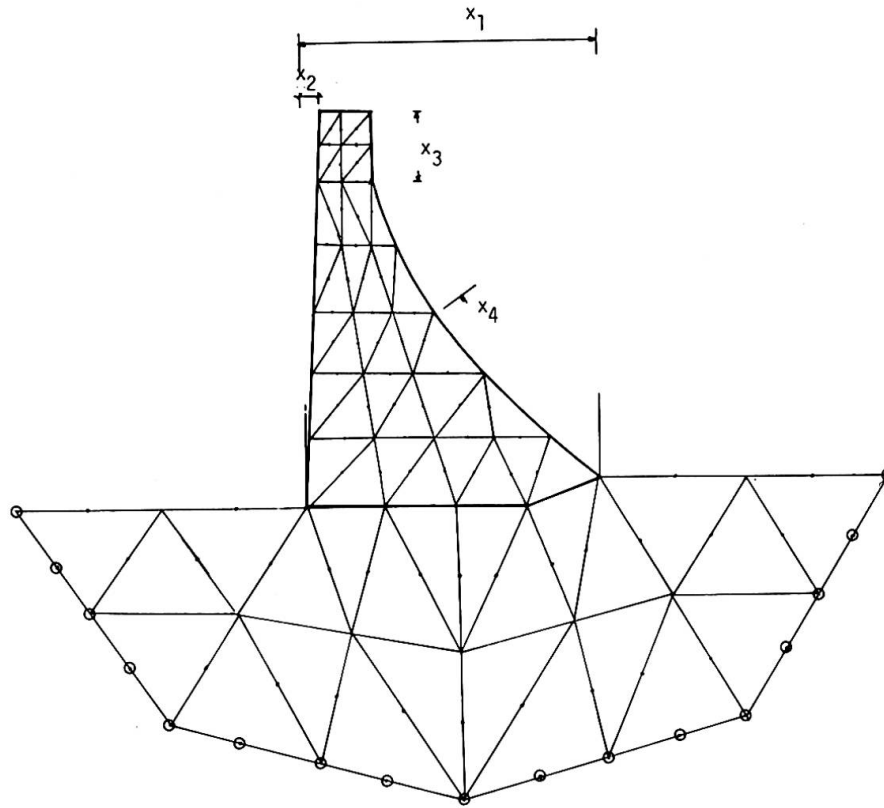


FIGURE 5: OPTIMUM SHAPE OF GRAVITY DAM,
64 Element, 262 D. of Freedom F.E. Analysis,
4 Design Variables, ref. 27.

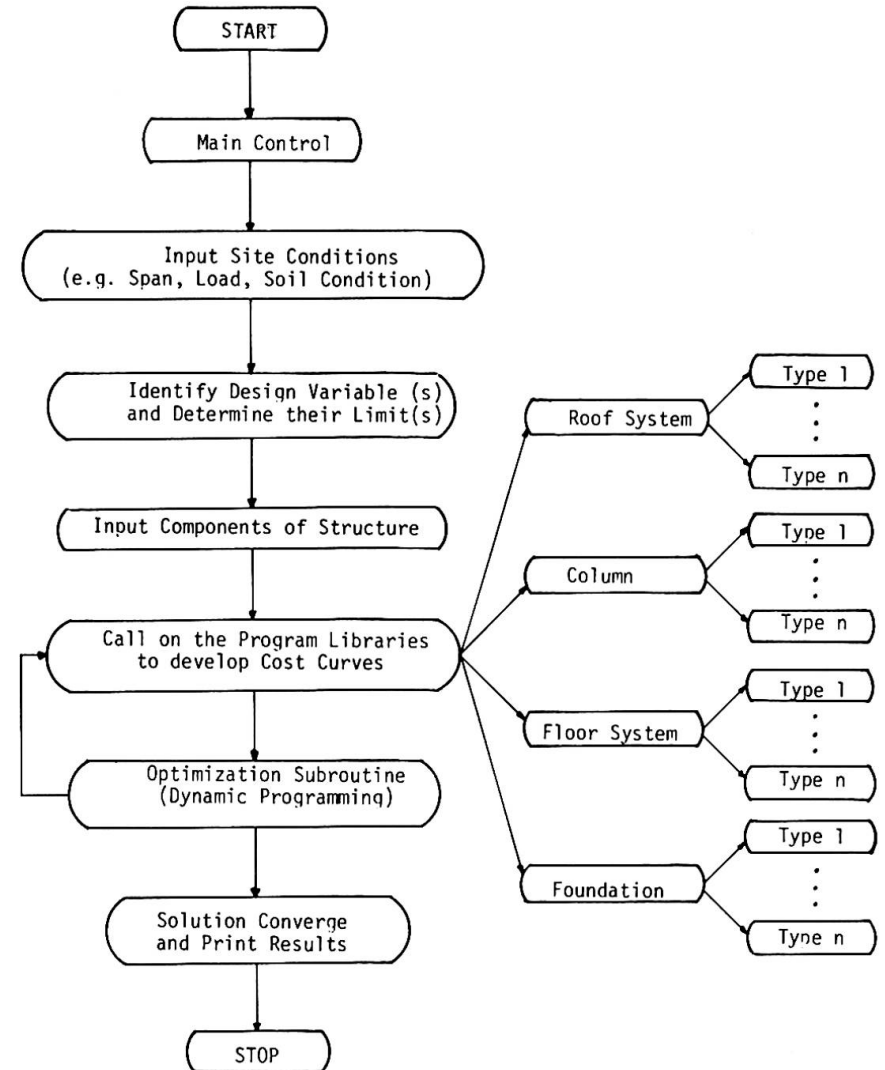


FIGURE 6: FLOW CHART FOR GENERAL SYSTEM PROGRAM FOR BUILDING OPTIMIZATION

It is apparent that despite some of this work described in shape optimization that much remains to be done and would accordingly be very profitable. Certainly, once the geometry of a structure is fixed there may be little amount of cost or material saving to be achieved by optimization techniques. It is therefore important that the geometry variables be considered as part of the automated design search. The most fruitful application appears to be three dimensional heavy structures such as towers, dams, pressure vessels, containment structures and storage tanks for which indeed form may follow economy and structure represents a major part of the total investment. In buildings, except perhaps for high-rise or other structures in severe seismic or climatic zones the potential material savings in the structure represents only a small percentage of the total building cost and therefore cannot dictate geometry in conflict with other architectural considerations. However, even in buildings particularly in industrial applications there may still be some gain to introducing geometry variables concerned with bay dimensions or location of shear walls and stiffening trusses.

5. SYSTEM OPTIMIZATION

The above discussion has almost exclusively centered on design variables which describe a structure but are in the main continuous variables such as element sizes or geometry variables. An exception has been the example in Figure 3 illustrating a single story building with discrete variables. What is unique about this latter example is the selection among alternative structural elements to perform the same function. Thus not only, say rolled steel beams but welded girders, reinforced concrete or prestressed beams and even trusses are sequentially compared while finding the optimum. Similarly, for other components of the structural load carrying system. Thus, the hierarchy of possible designs entering an automated optimization is considerably expanded.

Figure 6 shows a flow diagram of a general purpose program for performing the sequential comparison and optimization. It has been applied to design of single story buildings but may also be used for high-rise buildings. The various components required for the structure i.e., foundation, columns, girders, roof joints etc. are treated independently. A table is developed which obtains the minimum cost of

each component as a function of the loads, with the range of possible loadings automatically established based on the input data. Each component in such a table may be found only after comparison of different types of elements to carry out the same function. These individual elements may be from a stored list of optimum designs or else generated by element design programs for a specific structure. Any convenient programming method either non-linear optimization or direct design may be used for finding the element table. The entire program is then controlled with a dynamic programming type minimum cost selection scheme to choose the best combination of elements. The input data specifies which element types should be candidates for a particular structure and new elements can be added to a program library.

To make such general programs more accessible to designers simplified programming languages are needed. The input data must be in the form of basic geometrical dimensions and load data. The connecting of components must be inputted in a direct manner as well as specifying the possible element types which are design candidates. Since the variables are so general including element selection and geometry the output need not be a detailed design specification but rather a broad general indication of the element types and geometry which give the minimum cost structure.

6. LIMITATIONS ON OPTIMIZATION APPLICATIONS

Since computers and procedures for automated optimum design have been with us for some time it may be appropriate to reflect on why such methods have not always entered routine office practice. To be sure, a similar question may be raised about matrix analysis procedures that despite readily available programs many structures are built today after being analyzed with only crude approximate techniques. A major reason, at least in the United States, is the lack of incentive for designers to utilize computer methods which cost them money but save the client construction cost. This is one reason why many of the extensive applications of computer optimization have been in areas of design-build operations where there is stiff competition and hence a desire to reduce construction cost in order to obtain a job. Other optimization applications have been to bridge construction where many bridges are designed by state highway departments who are in effect the owners of the structure.

Another limitation has been that many of the computer programs have required detailed knowledge of mathematical programming and computer software techniques. This is changing and as in more recently developed structural analysis programs, the optimization routines in some of the examples cited above do not require the user to be at all familiar with such programming procedures. In fact, in many cases, the user finds these optimum design programs easier to utilize than traditional design tools or programs. It is this attraction, rather than the construction cost saved by optimization which has often decided the user in favor of this approach.

7. CONCLUSIONS

1. Advances in automated computer techniques for design have reached the stage where many types of detailed design and selection between alternatives for minimum cost can be carried out. Such design tools could be used for lowering cost, increasing standardization of elements and evaluating effects of changing constraints.

2. Success in achieving programs for element design has suggested that the design variable search be extended to include more significant variables of material and geometry. In particular shape optimization has been used, particularly for massive concrete structures such as gravity and arch dams and containment structures.

3. System programs capable of data manipulation and automated design of a wide variety of different structural schemes can be expected during the next few years. This should make possible the application of automated design by engineers with little background in programming and software techniques. At the same time there will still be demands for special purpose programs which more efficiently automate the design of a single type of structure. This will be done by organizations which have repeated need for a particular structure and are prepared to invest time and money in computer applications. An example of this latter approach is the GAD system developed by Professor Goble at Case Western Reserve University for the design of continuous welded plate girder highway bridges.(5.19) The program has been in use by the Ohio Department of Transportation for several years. The program reflects the cost data, design details, code specification and construction practices of that organization. However, due to the number of bridges of this type which are built the investment in computer programming was justified.

4. Since the cost of developing optimization programs may be large and the incentive for the design firms to use these programs may be relatively small the advances into practice of such techniques may be thwarted. A mechanism such as a central agency is needed to develop, document and disseminate such programs to insure wide practical utilization.

8. ACKNOWLEDGEMENTS

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SUMMARY

Structural optimization is defined as directed computer techniques for improving designs within well defined cost contexts and recognized constraints. Applications are divided into:

- a) element design characterized by code constraints of practice;
- b) system optimization involving large numbers of elements and
- c) discrete decision variables.

Solutions and examples are presented for all three categories. Geometry and shape optimization as well as general programs for optimizing a variety of different structures is discussed in detail.

RESUME

L'optimisation structurale a pour but d'améliorer le dimensionnement de structures au moyen de techniques appropriées d'ordinateur, dans des limites de coûts et de contraintes bien définies. Le domaine d'utilisation en est le suivant:

- a) dimensionnement d'éléments conformément aux règlements de construction
- b) optimisation de systèmes composés d'un grand nombre d'éléments
- c) variables discrètes de décision.

Des solutions et des exemples sont donnés pour ces trois catégories. L'optimisation de la forme et des dimensions est présentée en détail; des programmes généraux applicables à diverses structures sont également discutés.

ZUSAMMENFASSUNG

Die Optimierung von Tragwerken wird definiert als unmittelbare Anwendung der Computertechnik zum Entwurf und zur Berechnung von Konstruktionen bei genau umschriebenen Nebenbedingungen hinsichtlich Baukosten und zulässigen Spannungen. Die Anwendungsmethoden werden aufgeteilt in:

- a) Bemessung von Einzelelementen nach den geltenden Normenvorschriften,
- b) Optimierung ganzer Systeme bestehend aus einer grossen Anzahl von Einzelelementen,
- c) diskrete Entscheidungsvariable.

Für alle drei Kategorien werden Lösungen und Beispiele angegeben. Die Optimierung der Form und der Abmessungen sowie allgemeine Programme für die Optimierung von verschiedenartigen Bauwerken werden eingehend besprochen.

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Examples of Computer-aided optimal Design of Structures

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

Beispiele des Computer-Einsatzes bei der Optimierung

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Applied Structural Optimization

I. INTRODUCTION

The total process for the design of a sophisticated structure is a multistage procedure which ranges from consideration of overall system requirements down to the detailed design of individual components. While all levels of the design process have some greater or lesser degree of interaction with each other, the past state-of-the-art in design has demanded the assumption of a relatively loose coupling between the stages. Initial work in structural optimization has tended to maintain this stratification of design philosophy, although this state of affairs has occurred, possibly, more as a consequence of the methodology used for optimization than from any desire to perpetuate the delineations between design stages.

In recognition of this stratification, a possible hierarchy of design variable classes has been postulated.⁽¹⁾ The partitioning implied in this manner is not rigid but is representative of possible or probable design capabilities compared to total design requirements.

The hierarchy is

- a) Member Sizes
- b) Configuration
- c) Material Properties
- d) Construction or Topology

In the first class, all geometric details of the structure are fully defined and only member sizes are to be chosen by a design process. Although apparently a very restricted class of problems, this actually represents (i) the limit of most of the optimization capability available to date and (ii) an extremely wide class of structural problems. It is a fact that in many structures the location and configuration of a great deal of the primary structure is mandated by nonstructural considerations. Likewise materials and construction will frequently be dictated by environment, design codes, cost, etc. There are many other structures for which the above does not apply.

By configurational variables, location but not number of principal components is implied. Hence, the first two members of the hierarchy may be regarded as continuous variables*, whereas the latter two categories clearly involve discrete noncontinuous variation of parameters. Because of the difficulties encountered in dealing with noncontinuous variables within a mathematical framework, principal attention has been generally confined to the first two classes of variables, with maximum attention on the member sizes.

The principal approaches to the optimization of structural systems for minimum weight in the past have been based upon the use of a combination of mathematical programming or other rigorous numerical search techniques and an equally rigorous structural analysis method. There have been many variations on this theme, but the essential combination of methods has remained the same. For analysis, finite element methods have been the most frequent choice, while the numerical search techniques have run the gamut from linear programming to Monte Carlo.^(2,3,4) While this type of combination of methods is valid and appropriate for certain classes of problems, within the individual strata of the overall design process, it has led to certain intractable situations.

The rigor and sophistication of both the analysis and search procedures inevitably mandate numerical complexity and large computer costs for the optimization of anything approaching a representative large scale system. This, in turn, has cast considerable doubt upon the economic value of some optimization concepts. While many difficulties have been encountered using traditional methods of mathematical programming, there have been significant developments in new approaches to structural optimization which have overcome some of these difficulties for selected classes of problems.^(5,6,7,8)

While mathematical programming methods are fairly rigorous and extremely general in their range of applicability, computer programs developed along these lines tend to be effective for the optimization of small scale systems only. When expanded for the optimization of realistic large scale structures, such approaches tend to become excessively costly and also of doubtful reliability and accuracy. The major problems seem to arise from a large increase in the number of analysis iterations with increase in the number of design variables. In addition, the explicit or implicit need to calculate numerical approximations to derivatives of constraints with respect to all variables means that each iterative step itself becomes lengthy.

Some new developments in mathematical programming have tended to overcome some of the difficulties but others remain. In addition, the possibility of further new developments in both analysis and numerical search techniques cannot be overlooked.

*It is recognized that in many branches of structural engineering, principal members may only be selected from standard sizes and are not strictly continuous variables. This problem is usually treated by considering section properties as continuous variables and then selecting the nearest standard sizes for the final designs.

One approach to the problem which apparently avoids many of the pitfalls of mathematical programming is through the use of optimality criteria formulations.^(5,6)

The basic concept behind optimality criteria is the rejection of the generality of mathematical programming and the utilization of the physical characteristics of the structural optimization problem to generate an approach of somewhat limited applicability but of the greatest computational efficiency.

In the optimality criteria approach, preconditions regarding the optimum structural system are generated based upon a physical, mathematical or even intuitive understanding of the problem. A simple search procedure is then developed to find the design satisfying these specified criteria.

A full discussion of this approach to one facet of structural optimization is given in Section II along with examples of the applicability.

Even with the development of optimality criteria programs and other similar approaches, these methods still suffer in many cases from severe limitations with regard to class and ranges of design parameters which can be treated as variables in a search for an optimum system.

It is this latter fact which has tended to maintain the stratification of the design process. It has been simply not possible or practical to mix variables of the different hierarchy classes in any rigorous search procedure. The major handicap has been the lack of continuity of variation of some parameters. While the concept of fixing configuration, mode of construction and materials at the outset of design may be acceptable for some structures, it will certainly fall far short of a goal of overall system optimization. Attempts have been made, with varying degrees of success, to incorporate configurational variables.^(2,3) Generally, the stumbling block to the use of configurational and other variables (apart from computational costs) has been the requirement for continuity of variation in the parameters, due to the need for derivatives to provide search directions in a continuum space. With configurational variables this may be marginally possible provided the topology is undisturbed but to effect continuous variation in such concepts as material properties, construction mode and topology is beyond the capabilities of the vast majority of mathematical programming techniques.

For the optimization of large scale systems where many or all of the above parameters are initially undefined, more flexible and more general approaches have been sought. An additional consideration has been to develop an approach which would avoid the high computational costs of the more rigorous formulations, providing thereby an economic tool for ready use in design trade-off studies.

One new approach to the determination of the minimum weight of complex structural systems involving material, constructional and configurational variables in addition to the more conventional design variables has been developed and is labelled the "sieve-search" technique.⁽⁹⁾ In this new procedure, which sacrifices

some degree of rigor for economy and generality of application, an attempt has been made to consider the effects of detailed design on the overall configuration of the total system and thus tie together hitherto uncoupled design stages.

In performance of optimization studies using the sieve-search technique, the guiding philosophy is the generation of an optimal arrangement of pre-optimized components. In this approach, the detail components of a structure are optimized first using local loading conditions and then the major configurational parameters are varied in order to find the optimal arrangement of the locally optimized components. The optimal design is obtained by a sequential comparison of the individual designs based on discrete values of configurational and constructional design variables. The above procedure is labeled a sieve-search since all nonoptimum designs are eliminated by the sequential comparisons leaving only the least weight design. The process can be labeled "discrete" in contrast to the more classic approaches wherein continuous variables are treated.

The sieve-search method was developed initially for an applied to the design of an extensive class of surface effect ships. Section III discusses the basic philosophy behind this approach to structural optimization using the surface effect ship as a prime example. The extension of the procedure to other classes of structural design problems is both possible and economically attractive. Its potential use for bridge design is also discussed in Section III.

II. OPTIMIZATION USING OPTIMALITY-CRITERIA

As discussed previously, there are a number of basically different approaches to the problem of overall structural optimization. While some of the variations in the approaches stem from differences in the classes and types of systems which are being optimized, there are also problems for which two or more methods of solution are available.

A classic problem, of great practical interest, is the optimization of a structural system whose overall geometry is fully defined and fixed by a set of external conditions but whose member sizes are to be selected optimally. The structure will usually be subjected to a multiplicity of loading conditions (no one of which is uniquely critical) and in addition to known limitations on the strengths of individual components, stiffness of the system may be of critical importance. Also fabricational constraints or other codes may mandate minimum sizes for constituent members.* For this type of problem which is encountered frequently in engineering design, the primary approaches to optimization developed during the 1960's were based upon the use of mathematical programming

*In discussing a structure, the concept of an assemblage of individual elements is used. This is generally consistent with the idea of a finite element model which is usually used for the actual structural analysis. If a continuum is considered, it, too, would be represented as an assemblage of discrete elements, which may be viewed as separate variables in an optimization process.

formulations. (4) Because of the computational difficulties encountered with mathematical programming, this approach has been largely abandoned for large scale structures and newer methods based on the concepts of optimality criteria have been developed.

The underlying concepts behind optimality criteria methods can best be illustrated by considering the contrasts between optimality criteria and mathematical programming. In mathematical programming approaches, sets of rules are established for numerical search procedures which will determine an optimal solution in a strictly empirical manner. The set of search rules will guarantee a continuous and monotonic decrease in a prescribed merit function, essentially without regard to the physical (or sometimes even mathematical) nature of that merit function. The search will regard constraints, if such exist, and will continue searching until no further improvement in the merit function is possible. No preconditions concerning the nature of the optimum are specified beyond the criterion that it is impossible or uneconomic to determine a further design which will be an improvement on the present design. This approach may be labelled post hoc since the optimum is identified essentially only on an after-the-fact basis. Both the strength and weakness of mathematical programming reside in this concept. The strength is the generality which this independence of mathematical formulation imparts, with the resulting wide range of applicability; the weakness is that no use is made of any of the physical characteristics of the problem and hence frequently an unnecessarily long and costly solution process results. The antithesis of this situation arises in an approach which recognizes the physical nature of the structural optimization problem per se and sets out to take fullest advantage of the restricted class of problem. In this approach, some conditions are established initially concerning the nature of the design which will be regarded as optimal. These conditions, which are defined before initiation of the redesign process, may be rigorously exact, approximate or even intuitively assumed. The essential requirement is that their application will lead to a relatively simple (usually iterative) algorithm for a redesign process converging on the design which satisfies the initially prescribed criteria. This approach is then labelled a priori, since the characteristics of the optimal system are specified initially.

The classic and most obvious example of an optimality criteria approach is the time-honored fully-stressed-design. Every practical engineer is fully aware of and would probably support the basic idea that the most efficient (optimal) design is one in which every member is used to its fullest extent under at least one loading condition. Prior to the advent of computers and the development of advanced methods of structural analysis using finite elements, generations of structural engineers have traditionally attempted to inject some degree of optimality into designs by analyzing a trial structure, using some appropriate quasi-classical procedure, and adjusting member sizes to eliminate over- or under-stressing. The more ambitious engineers might even have re-analyzed and re-sized the structure one or more times. Probably very few practicing engineers ever wondered whether anything is invalid with this rather natural 'calibrated-eyeball' approach. Accurate analysis of indeterminate structures presented a difficult problem prior to

the introduction of computers, discouraging the repeated use of more elaborate schemes, while approximate analyses were somewhat insensitive to the crucial effects of rerouting internal force distributions resulting from resizing iterations.

With the appearance of computers in the fifties, the first attempt at automated optimum sizing was the computerized version of the above procedure, initially still relying on time-honored approximate analysis methods. The ensuing development of the finite element methods by the early sixties made rather accurate analyses possible for indeterminate structures of virtually any form or shape. Instead of just two or three resizing cycles now a much larger number of cycles became feasible, at least for numerical experimentation by researchers, even if not in practice.

This simple and intuitive concept was eventually formalized as the fully-stressed-design (f.s.d.). To achieve f.s.d., the most commonly used algorithm, although not the only one available⁽¹⁰⁾, is the simple stress-ratio. In the stress-ratio algorithm, it is assumed that the gross forces in any member of the structure will not vary with member size and hence the member properties may be adjusted directly in the ratio of the actual to the allowable stress. In indeterminate structures, changing member properties generally effects some redistribution of internal forces, so that an iterative process is required to achieve a f.s.d. The most important feature of the stress-ratio, and other similar algorithms is, that, in marked contrast with direct numerical search procedures, the number of re-analyses needed to reach an apparently converged design is usually small and independent of the size of the problem. This intuitive approach fulfilled a need for automated sizing for strength requirements and the strength optimization problem seemed to be under control.^(11,12) No such simple and efficient method existed at that time for stiffness related problems.

In the late fifties, nonlinear programming methods were introduced as the correct framework for the general structural optimization problem.^(13,14) With the development of these more rigorous methods, which were applicable to both strength and stiffness constraints, it was shown that f.s.d. is not necessarily the correct optimal solution for indeterminate structures. On the other hand, it was also shown that f.s.d. may indeed frequently be a correct solution, or more importantly from an engineering viewpoint, may be a close approximation to the correct solution. Thus with f.s.d. a very efficient but invalid method of strength optimization is provided. Fortunately not too many practicing engineers are inclined to question the rigor and validity of f.s.d. and merely welcome its advantages.

The standard f.s.d. stress-ratio redesign algorithm tends to drive a structure towards a design with the stiffest routing of internal force flow, which may or may not coincide with the optimal force flows. This trend may not become apparent if only a few resizing cycles are performed and because they do usually tend to produce a succession of improved designs, they are of great value to the engineer.

The potential sources of problems with f.s.d. are quite easy to point out, but the extent to which they are present in any given situation is extremely difficult to assess. The difficulties can be demonstrated in two small example research problems, where comparison with correct solutions, obtained by numerical search, is possible. The two examples may be regarded as somewhat pathological but even for these problems it is not entirely clear what the true nature of the pathology is. Hence, it is not possible to state categorically that any real system does not contain the same disturbing influences. In the stress-ratio algorithm, only the constraints (stresses) themselves are considered and no reference is made to the factors of relevance to the merit condition, such as density. Thus f.s.d. is completely insensitive to favoring structural elements according to their strength to weight ratios. Therefore, f.s.d. tends to break down in structures which contain materials of different densities or markedly different allowable stresses. The first example (Figure 1) is of two parallel bars sharing a single load. One bar is of steel, the other is of aluminum but both have the same allowable stress. The stress-ratio algorithm will increase the size of members with higher material stiffness and/or lower allowable stress. In this example the aluminum bar will vanish and the steel bar will be retained. Clearly this is a f.s.d. but not a minimum weight design. If both bars are made of the same material, but with different allowables, the algorithm will eliminate the higher strength bar, again a poor design. It should be noted that the optimal solution for these two problems is the other bar fully stressed. The difficulty here lies with the stress-ratio algorithm, rather than the concept of f.s.d.

A second more elaborate example is the 10-bar truss shown in Figure 2.⁽¹⁵⁾ The truss has a single loading case and initially the stress limit in all members is ± 25000 psi. The f.s.d. obtained using stress-ratio weighs 1593 lb which is known to be optimal. Successively raising the allowable stress in bar No. 10 to ± 30000 psi, ± 50000 psi and ± 70000 psi and again using a stress-ratio, designs of 1545 lb, 1725 lb and 1725 lb, respectively, are generated. The 1545 lb design is also known to be optimal but the last two solutions of 1725 lb are clearly unreasonable and considerably in error. In these two cases stress-ratio has tried to eliminate the high strength bar, resulting in the poor designs. Using mathematical programming techniques^(8,16), the optimal design for the two high strength (50000 and 70000 psi) cases is known to be 1497 lb. Further examination of the problem reveals the interesting fact that, in both the stress-ratio (1725 lb) and the optimal (1497 lb) designs, all members are either at their full allowable strengths, or at their minimum sizes, except for bar No. 10. In each case the stress in bar No. 10 is 37,500 psi, although both designs are radically different. Assigning an allowable stress of $\pm 37,500$ psi to bar No. 10 and again applying stress-ratio, results in a third fully-stressed-design, weighing 1568 lb, which is quite different from the other two. Clearly the whole field of f.s.d. needs further research. Some studies have been conducted and variations on the stress-ratio algorithm have been proposed⁽¹⁷⁾, but with limited success.

On the other side of the coin, large scale programs have been developed, basically using f.s.d. and these programs have been successfully applied to the design of real structures. Whether or not such structures are truly optimal is somewhat academic when it is realized that such designs obtained at moderate computer cost are undoubtedly superior to those generated by hand.

Figure 3 presents the computer-generated plot of the finite element idealization of a complex wing structure. The total model had 5397 finite elements, 4104 displacement degrees of freedom and 20 separate loading conditions. Redesign studies were performed on the inboard half of this structure starting from various initial designs. The model considered had 3275 finite elements (design variables) and 2520 displacement degrees of freedom. Only two loading conditions were considered critical for sizing. In all cases only three iterations were performed showing acceptable convergence. Six iterations would have been sufficient for accurate production work. The program used for this optimization was ASOP(18) and for the three iterations required 6000 seconds CPU time on a CDC6600 computer.

The preceding discussion has dealt rather extensively with f.s.d. because this is the classic example of an optimality criterion, and it is an approach to optimization which is widely recognized and accepted. It is, nevertheless, very limited in its use. Its role as an optimality criterion, per se, would probably not have been recognized, if there had not been a pressing need for the development of suitable and efficient optimization procedures for stiffness constraints. The driving motivation for the exploration of optimality criteria methods for stiffness constraints was the excessive cost of using direct numerical search methods. What was sought was an approach as simple as stress-ratio but for displacement constraints. Optimality criteria were investigated since such concepts, by definition, contain gradient related information as a result of their derivation. By taking full advantage of the special structural properties of the problem, these criteria should lead rapidly and efficiently to the solution.

The actual development of a practical method for stiffness constraints was a multistage process in which many researchers individually contributed key concepts(19,20,21,5). It is not of relevance here to discuss all the stages in this development progression; a fuller description may be found in Reference 22.

The essential step in the development of the currently used approach to stiffness constraints was the formulation of a single displacement constraint problem using a Lagrangian multiplier. In a structural system with fixed geometry, A_i , the characteristic sizes of constituent members, are considered to be design variables. If $W(A_i)$ is the merit function for the structure and $F(A_i)$ is a single displacement which is to be constrained to have a magnitude C , then values of A_i which minimize W , while satisfying the equality constraint can be determined by use of a Lagrangian multiplier formulation. The expression

$$W^* = W(A_i) - \lambda \left[F(A_i) - C \right] \quad (1)$$

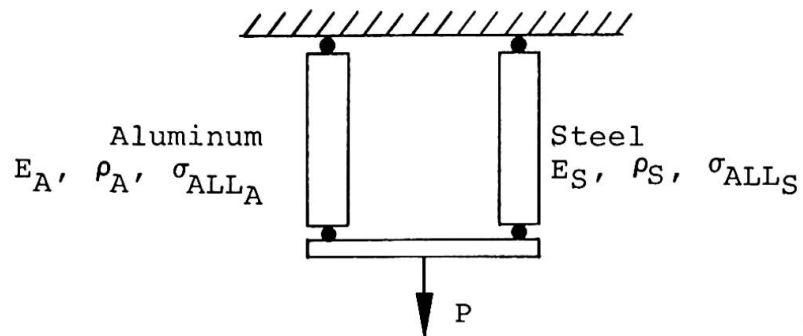


Figure 1. Parallel Bars of Aluminum and Steel

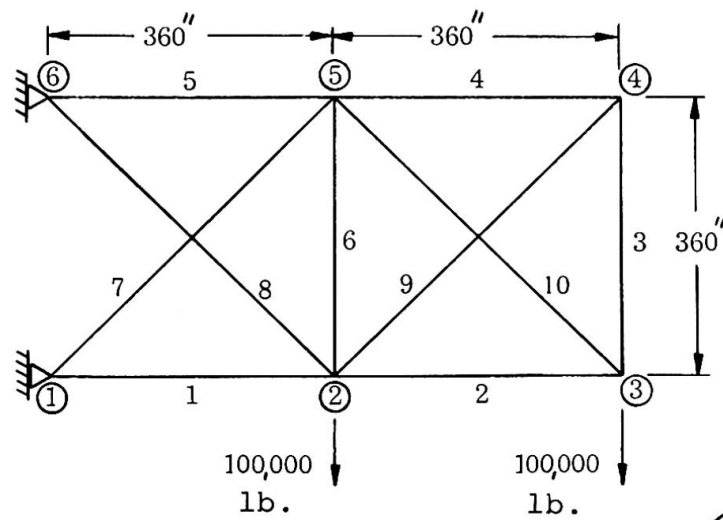


Figure 2. 10 Bar Truss

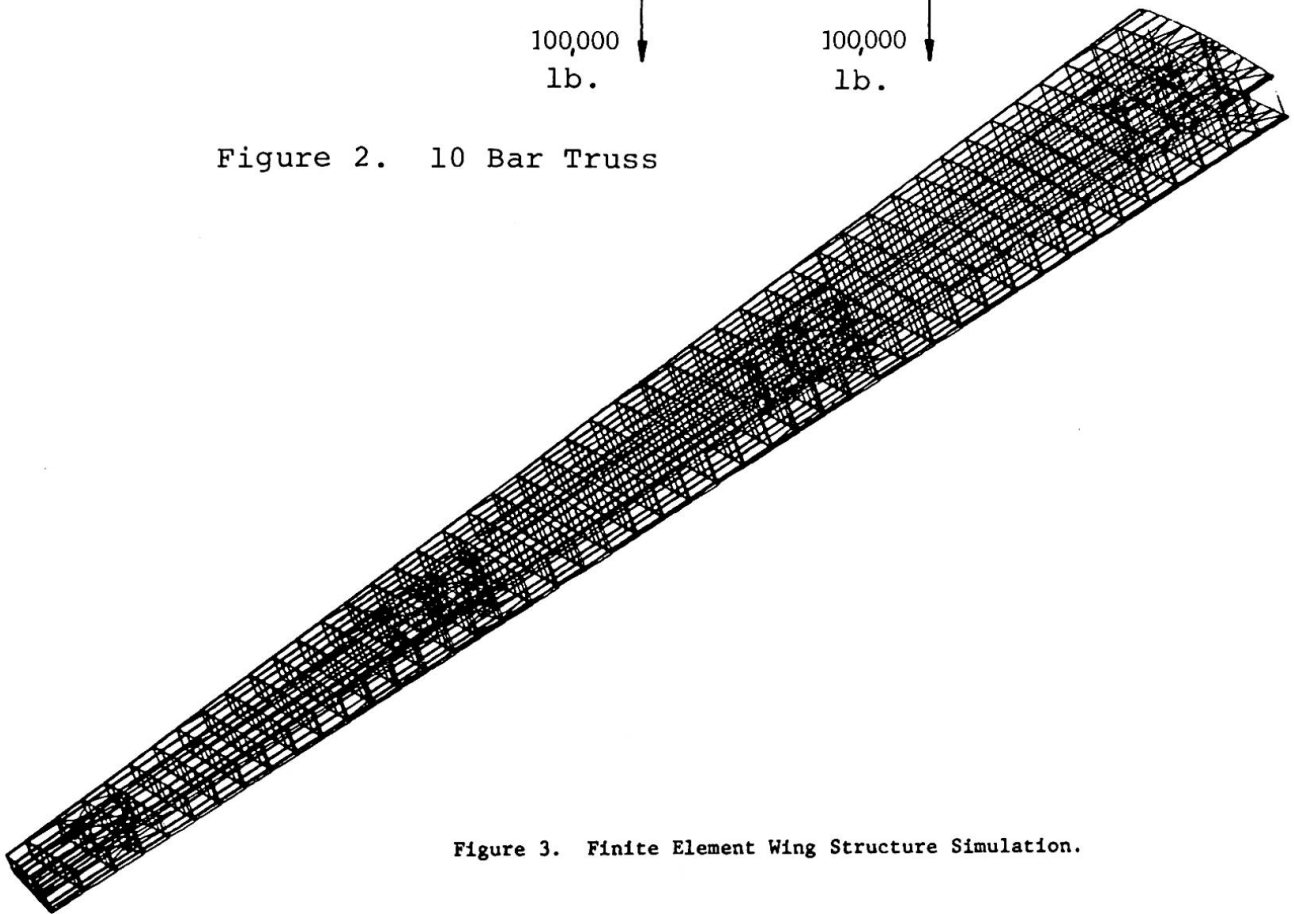


Figure 3. Finite Element Wing Structure Simulation.

is written and differentiated with respect to A_i to yield

$$\frac{\partial W^*}{\partial A_i} = \frac{\partial W}{\partial A_i} - \lambda \frac{\partial F}{\partial A_i} = 0 \quad (2)$$

Equation (2) is then the necessary condition for the optimum system, or the optimality criterion. For specific classes of problems, it can be proven that this condition is also sufficient for global or local optima.

Equation (2) can be rewritten in the more revealing form

$$\frac{\partial W / \partial A_i}{\partial F / \partial A_i} = \lambda = \text{constant, for all } i \quad (3)$$

Written in this form, there can be seen the valuable and relevant information that, in an optimal structure, the change in the measure of the behavior (displacement) for a unit change in the measure of merit is the same for every free variable. That is, the cost of improvement in the design is the same for every member in the optimal system. This statement is quite general and applies to the optimization of a structure for any type of merit function (weight, cost, etc.) and for any type of constraint which is characteristic of the structure as a whole. Thus, not only displacement constraints can be considered, but also overall buckling, dynamic response, flutter and any other phenomena which are indicative of total structural response.

By the same token, strength constraints do not satisfy the criterion of Equation (3), since they are, of necessity, individual characteristics of the constituent members and not of the structure in toto.

In order to translate Equation (3) into a working procedure for the stiffness optimization of a structure represented by an assemblage of finite elements, some particularization of the general definitions used previously is necessary. It is assumed that both merit and stiffness of the system are linear functions of the design variables A_i . These specializations are not necessary; they are made only to simplify the expressions for the cost and constraint function derivatives for a concise presentation. Other functional relationships are possible. One additional specification is crucial to the derivation of the final simple numerical procedure. This requirement, which is generally satisfied by most analytical methods, is that both the total cost and total stiffness be sums of individual members contributions. As a result, the simultaneous equations implied by Equation (3) uncouple for each value of i and can hence be solved in an extremely expedient manner using simple recursion formulae. The not very widely recognized importance of these key considerations, satisfied fortuitously by finite element analysis techniques, is that they remove obstacles which hitherto existed to the use of classical Lagrangian multiplier formulations for structural optimization. It is assumed in the following brief development that the complete

behavior of the structure is analyzed using the finite element displacement method. In accordance with the above definitions, a merit function (weight) is written

$$W(A_i) = \sum w_i = \sum \bar{w}_i A_i \quad (4)$$

Similarly the stiffness behavior is written

$$F(A_i) = \sum e_i = \sum \bar{e}_i / A_i \quad (5)$$

Equations (4) & (5) merely express the linear summations discussed previously. $\bar{w}_i$ & $\bar{e}_i$ are the contributions of individual unit-sized elements to the total weight and stiffness of the system. For a simple bar element with A_i as the cross-sectional area

$$\bar{w}_i = L_i \rho_i \quad (6)$$

where L_i is the bar length

and ρ_i is unit material cost (density).

For other types of elements A_i & L_i must be appropriately defined, but the general form of Eq. (6) still holds.

The stiffness of a structure under an actual loading system (P) is computed by imposing a virtual unit load system (Q) in the direction of displacement required, and computing the virtual work of system. The contribution of each element is given by

$$e_i = \delta_i^{Pt} K_i \delta_i^Q \quad (7)$$

where $\delta_i^{P,Q}$ are the vectors of the nodal displacement of i^{th} element due to the actual and virtual loading systems, $K_i = k_i A_i$ is the stiffness matrix of the element and k_i is the unitized element stiffness matrix.

For other types of stiffness related constraints, such as buckling, vibrational response, etc., corresponding relationships to Eq. (7) can be derived and used in the subsequent development of a suitable redesign algorithm. Examples of buckling and dynamic response constraint formulations can be found in References 23 and 24. Substituting the above relationships into Eq. (2) and after some algebraic manipulations the recursion relationship is obtained

$$A_i^{\nu+1} = \frac{A_i^\nu}{C^*} \sqrt{\frac{\delta_i^{Pt} k_i \delta_i^Q}{L_i \rho_i}} \sum_j A_j^\nu L_j \rho_j \sqrt{\frac{\delta_j^{Pt} k_j \delta_j^Q}{L_j \rho_j}} \quad (8)$$

where the superscripts ν , $\nu + 1$ indicate the values of A_i at successive iterations and C^* is the prescribed value of the stiffness.

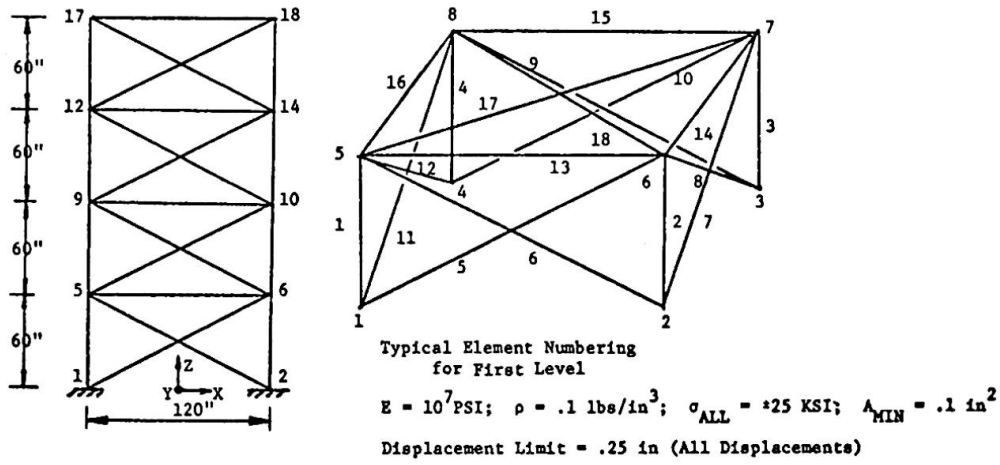
Eq. (8) is then the redesign algorithm for a single stiffness equality constraint. In order to generalize the algorithm for multiple inequality constraints, the recursion relationship is applied to each constraint in turn and then the dominant values of A_i are selected for each member. The redesign process is iterative at each stage and a procedure for partitioning design variables into active and passive groups is used to select which members are effectively design by which constraints. This algorithm, known as the envelope method, also permits the simultaneous consideration of strength and minimum member sizes. The envelope method is an obvious simplifying approximation and does not strictly satisfy the correct optimality criteria for multiple constraints. It basically disregards the sizing given by one constraint when satisfying another.

Thus, analogous to the case of f.s.d., a procedure has been obtained for stiffness redesign based on an approximate criterion which has the merit of great simplicity and good general behavior. Experience has shown that the solutions for stiffness constrained problems obtained using the envelope method usually compare very favorably with more rigorous solutions obtained otherwise at much greater computational cost. The convergence characteristics of the envelope method are similar to those of f.s.d. with usually rapid convergence in a very small number of iterations, apparently independent of problem size.

A number of computer programs using optimality criteria algorithms have been developed. The program OPTIM II⁽⁶⁾ is a large scale program which contains eight different finite elements in its basic library and is capable of application to a considerable variety of large scale problems. The elements include bars, beams and plates of various types. The program also contains a number of special features such as provision for linking elements, plate buckling computations and other capabilities intended to simplify the analyst's work.

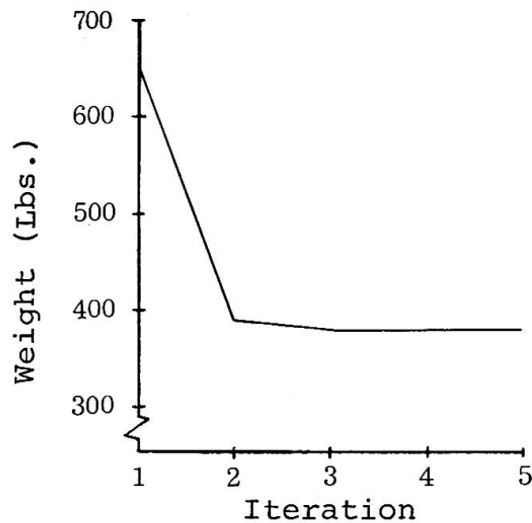
The capabilities of such optimization programs can be best illustrated by a few example problems. These problems are generally small scale, but are intended to demonstrate the potentialities of the programs rather than to overwhelm by sheer size of problem alone. The programs themselves are only really limited by available computer size and the price (in terms of numbers of analyses) which the designer is prepared to pay.

The first example (Fig. 3) is of a simple four-level tower structure, composed of 72 primary members. The tower is subjected to two loading conditions as indicated. For obvious reasons it is desired to maintain the double symmetry of the structure, although the loading itself is nonsymmetric. The automatic linking feature is used to tie together elements where necessary. There are stiffness constraints to ensure that the tower does not sway too much under load. Figure 4 indicates that only four analyses were required for convergence. The efficient redesign logic at each stage requires only 10-15% of the analysis time.



LOAD CONDITION	NODE	DIRECTION		
		X	Y	Z
1	17	5,000	5,000	-5,000
	17	0	0	-5,000
	18	0	0	-5,000
2	19	0	0	-5,000
	20	0	0	-5,000

(a) Geometry and Loading



(b) Iteration History

Figure 4. 72 Bar Four Level Tower

The second example is the geodesic dome of Figure 5, designed for both strength and stiffness constraints. In this problem involving 156 elements, the dome was subject to a uniform vertical load and the vertical displacement of the central point was limited. This problem was studied using various optimization programs available and full details of the results may be found in Reference 22.

Figure 6 represents the idealization of a wing carry-through structure on a large heavy swing-wing aircraft. The loading arises from operation with the wing in two different positions. The loadings on the pivot points were then principally flexural for the wing in a forward, unswept position and torsional with the wing fully swept back. In order to maintain the aerodynamic characteristics of the wing, the rigidity of this structure must be very high. Severe limits are therefore placed on the allowable displacements and rotations of the pivot points. Initially a strength only optimization was performed yielding a weight of 5035 lb in 50 iterations. This is a very slow convergence but it should be noted that a weight of 5049 lb (0.3% heavier) was reached by iteration 18. The structure was then reoptimized with both strength and displacement constraints. The least weight of 6159 lb was reached at 50 iterations, with the same slow convergence, but 6216 lb (1% heavier) was obtained at iteration 14.

If all members of the initially obtained strength-limited design had been directly scaled to reduce the displacements of that design to meet the specified stiffness constraints, the structure would have weighed 7961 lb, over 29% heavier than the actually optimized structure. This indicates the redistribution of material effected by the optimization algorithm.

In this example, a bar idealization has been used for simplicity, but in the actual structure, plates and shear webs would be used. This raises an important point in structural optimization regarding the influence of the idealization on the optimal system. All redesign logic, for both stress and stiffness constraints is eventually predicated upon the detailed internal stresses in the individual elements. Finite elements, or indeed any other numerical analysis techniques, by their very nature introduce a certain degree of approximation into a solution. Finite elements are a piecewise representation of a continuum and certain approximating assumptions are essential to their basic derivation. The actual errors introduced into a given analysis using finite elements is usually very small and hence the results obtained are perfectly satisfactory for an engineering analysis. The widespread use and acceptance of finite element methods is a testimony to their validity.

For optimization, where many analyses may be performed and each redesign is dependent upon an erroneous analysis, the effect of the inaccuracies may be cumulative. This does not imply that the final system will be unsafe, but merely that the optimization of a structure modelled by two slightly different idealizations could result in two radically different designs. Care must be exercised in the development of optimization programs to ensure that only the most accurate analysis techniques are used. In finite element analyses, bar elements are exact and involve no

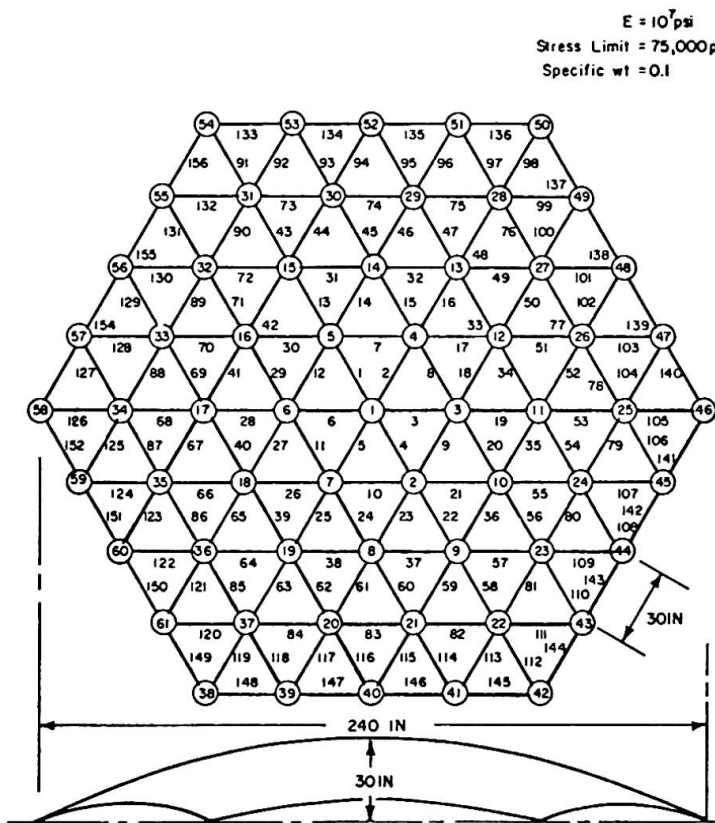


Figure 5. Geodesic Dome

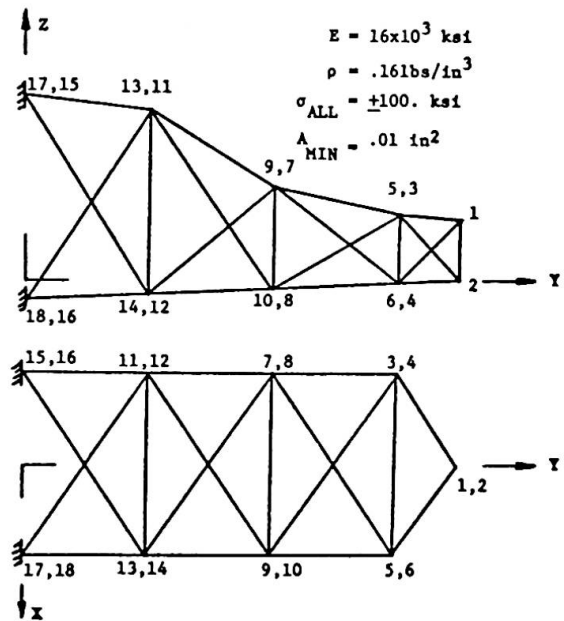


Figure 6. 63 Bar Wing Carry-Through Box

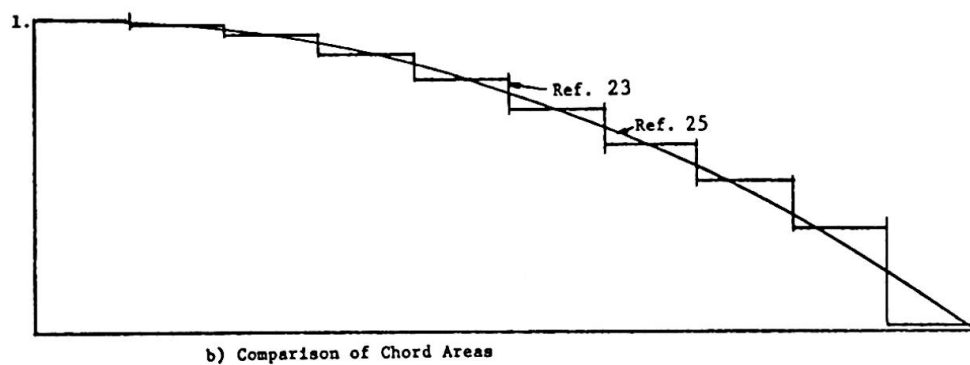
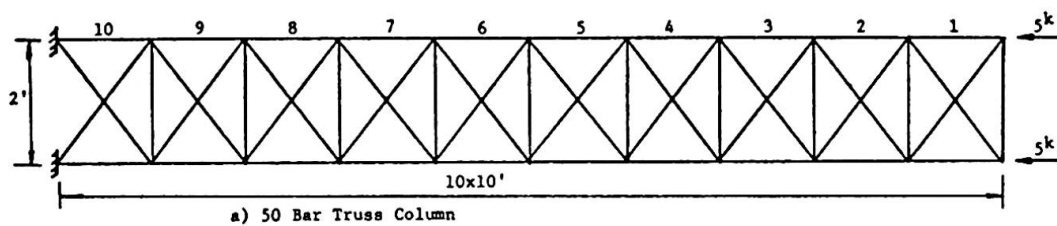


Figure 7 50 Bar Truss Column Problem

approximations. They are therefore frequently used for demonstration problems since they invite direct comparison of optimization solutions obtained by other methods by eliminating idealization errors.

The final example presented is that of the buckling of a simple laced column (Figure 7). The column has 50 bar elements and was optimized using a stiffness representation of the eigenvalue buckling problem.⁽²³⁾ The areas obtained for the chord (axial) members are shown plotted in Figure 7 in comparison with the exact solution obtained for the face sheets of a similar sandwich column.⁽²⁵⁾ The comparison is very encouraging.

III. OPTIMIZATION USING SIEVE-SEARCH

The selection of a truly optimal design to satisfy a particular set of engineering requirements is a complex process which strictly involves the consideration of all the classes of variables discussed in Section I. The approach presented in Section II deals with a more limited design problem in which geometry, material and construction are assumed to have been fully defined. A major question indeed must be on what basis will these governing design characteristics have been selected.

While it is true, that many psuedo-design parameters such as materials and construction cannot be treated as continuous variables and hence cannot be incorporated into any standard mathematical programming search technique, other considerations do enter into the picture. For the vast majority of engineering systems, only a limited number of materials really come into consideration. For civil engineering primary structures, titanium or boron-reinforced plastics, for example would have little or no applicability. Similarly reinforced concrete is seldom to be found in aerospace structures. Thus although there may be a potentially large number of possible materials and construction types, engineering practice and experience will indeed limit these to a finite set, which may be considered discretely. In a similar manner, although some aspects of the structural configuration, as defined by the arrangement and location of the principal structural members, are parameters to be selected by the designer, certain configurational characteristics will be absolutely defined by the service requirements of the structure. In addition, aesthetics and engineering codes will probably place some restrictions on other variables. The net result again is the specification of a finite set of configurational parameters. Finally the detailed design of individual structural components is governed by the critical loading which they experience locally. This critical loading may either arise from overall structural loading or may be a purely localized loading system which has little influence on the structure as a whole. Thus the optimum design can be generated for a given component under a specific loading system in isolation. Extending this concept, a range of optimal members can be pre-designed in some suitable manner for appropriate ranges of applied loadings and sizes. This then is a so-called data bank. An example of a data bank is a structural handbook, which specifies appropriate code sizes of beams, columns, etc., for given applied loadings. It is well recognized that internal loading distributions are not strongly influenced by small variations in member

properties. Hence, except for highly pathological problems of the type discussed in the previous section, it is assumed that only a very limited number of redesign iterations is required for a satisfactory degree of convergence. If the critical loading is purely local, convergence is achieved almost immediately.

With the above considerations as guides, an approximate optimization procedure for large structures was developed.⁽⁹⁾ The guiding philosophy in this sieve-search approach is that the optimum system is an optimal arrangement of pre-optimized components.

Individual components are optimized initially under local loading conditions and the potential designs stored in a data bank. A program is then set up which cycles sequentially through all the finite combinations of the major variables. For each configuration so defined, or segment thereof, an optimum design is generated using the data banks and compared with the best design available at that point. The best design is retained and the cycling is continued.

The efficiency of this process is then highly dependent upon the data banks available. These banks contain properties of optimized components generated either by classical methods of optimization or selected from standard structural codes. An additional, but nonetheless important facet of the preset technique is the use of simplified engineering analysis methods wherever possible during the iterative phases of the redesign cycles. Herein lies the efficiency of the sieve-search technique whereby literally hundreds of redesigns are rapidly made for selected configurational variables from which the optimum is obtained.

As a prime example of the sieve-search technique its application to the design of class of surface effect vehicles (SEV) is considered initially. The extension of the procedure to other structural systems is discussed later with particular emphasis on bridge structures.

Figure 8 is an actual photograph of a surface effect vehicle which is prototypical of an extensive class of high speed cargo vessels. Although operating in a marine environment, SEV are essentially aircraft-type structures which must be supported on a cushion of air. The development of least weight structures is therefore of prime importance in the design of such vehicles since the economic viability of SEV are dependent on low structural weight.

Before initiating the design process consideration must be given to the classes of parameters which would realistically be regarded as variables in performing the actual design. Thus external envelopes would be fixed by hydrodynamic and performance requirements - although some trade-off studies between configurations and performance might be desired. Figure 9 indicates the general form of the external craft envelope.

Constructional materials and modes may be fixed or may be selectable from a limited class of candidates. Environmental considerations will narrow the number of available materials and for each material only a very small number of constructional modes is technically feasible.

The internal arrangement of longitudinal and transverse beams and bulkheads will have been fixed in an overall sense, but the individual spacings and sizes will be treatable as free variables. The only possible restrictions being dictated by internal storage requirements. This then selects the classes of potential variables - material and construction modes, configurational variables and component sizes. In a sieve-search procedure, an attempt is made to consider all three classes.

In the particular case of SEV existing experience has indicated that a major portion of the structural design is governed by local hydrostatic and hydrodynamic pressure loadings. In addition, the requirement for internal cargo containers has a profound influence on the ranges of beam and bulkhead spacings which can be reasonably used in the ship design.

With these considerations, the design for minimum weight can be conducted on the basis of optimizing the structure for normal pressure loading and subsequently checking the resulting design for strength due to overall bending, shear and torsion loads. Plating (panel) thicknesses and beam cap areas are then increased to ensure the overall integrity of the structure. This approach led to two main procedural items - overall ship weight minimization and plating optimization. These led naturally to definition of the following variables:

- a) Construction module, including both material and constructional characteristics. Figure 10 presents sixteen combinations of materials and constructions which were considered feasible for this type of system.
- b) Configurational Variables (Figure 9)
 - 1) Longitudinal bulkhead spacing, l_{LB}
 - 2) Transverse bulkhead spacing, l_{TB}
- c) Dimensional Variables
 - 1) Plating - Panel Skin Thickness and Stiffener Dimensions

A finite number of longitudinal and transverse bulkheads and transverse frame spacings are specified and these configurational variables are optimized for minimum weight. Optimization of the dimensional variables results in generation of the data banks which store pre-optimized dimensional variables of structural components. In the present application, panels of the type shown in Figure 11 were optimized for minimum weight on the basis of normal pressure. A penalty function formulation with a Rosenbrock⁽²⁶⁾ search procedure was used. Geometric programming



Figure 8. Prototype SEV

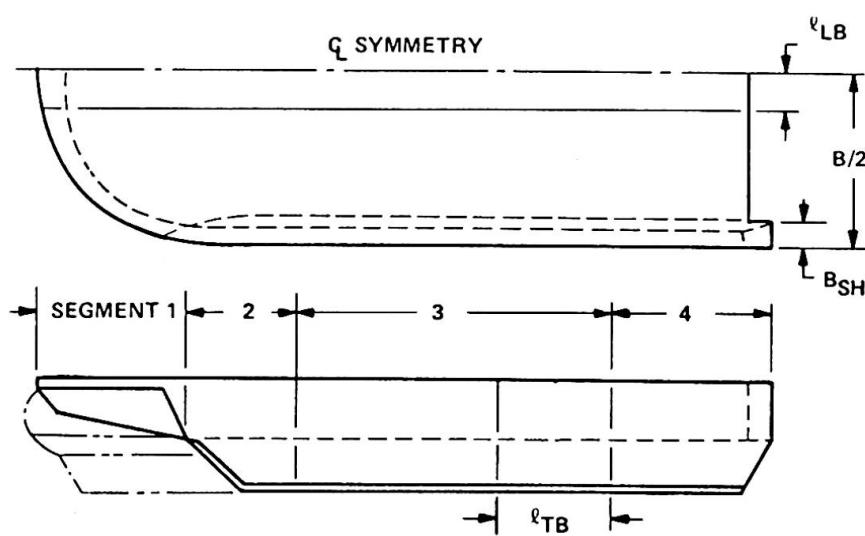


Figure 9. SEV Structural Envelope

methods for structural optimization⁽²⁷⁾ also appear extremely promising for use in component design where constraint and merit functions are expressible as nonlinear polynomials. The governing equations for the strength and stability of the panels under the action of uniaxial compression and in-plane shearing were also derived for use in the sieve-search method. For purposes of simplification, the panels were assumed infinitely wide and all critical conditions were expressed in terms of panel length, normal pressure, material characteristics and panel cross-sectional dimensions. The optimizations were then performed using the panel cross-sectional dimensions as variables. In addition to strength and stability constraints, consideration was also given to fabrication limitations for the various types of sections optimized.

Sixteen data banks consisting of eight basic geometric configurations with four materials namely, aluminum, steel, titanium, glass reinforced plastics were calculated and labeled construction modules. For these, all practical "failure" modes were derived analytically in five basic categories: material strength, overall buckling, local buckling, deformation limits and fabrication limits. Actually deformation and fabrication limits are not failure modes, but rather design specification modes which in many cases determined the optimum panel design.

When performing the optimization procedure, all of these critical conditions were expressed as inequality constraints. The fabrication constraints were based upon:

- 1) Considerations of practical sections, for example, no overlap of flanges, and
- 2) Data on the range of extruded sections which could be manufactured using existing dies and presses.

The deformation constraints were based upon the specific maximum allowable panel deflections.

The data banks are entered during the sieve-search process using the current spacing, L , and panel pressure, p , as shown by the dashed line on Figure 11. The resultant minimum weight, w , and cross-sectional geometry is stored for subsequent weight calculations.

A flow chart for the sieve-search program proper is shown in Figure 12. For application of the method, the vehicle was broken down into the four segments shown in Figure 9. These segments were defined in the present case by variations in the pressure loadings acting on the hull. Other forms of segmentation could have been selected to suit any arbitrary conditions. Within each segment certain configurational parameters were kept constant, although varying from segment to segment. The location of the longitudinal bulkheads was common to all segments. Each segment was further broken down into smaller zones such as deck, sidehull, etc. Each zone is then designed separately and combined to form the design of a segment.

	Aluminum		Steel		Titanium		GRP	
Alloys	5086	6061	HY-130	ALMAR-362	6Al-2Cb-1Ta-1Mo	6Al-4V	Polyester, Fiberglass Reinforced	Epoxy, Fiberglass Reinforced
Temper or Condition	H-111 Extrusion H-34 Sheet H-117 Plate	T6 { Sheet Plate Extrusion	Q & T { Plate Extrusion	Q & T { Plate Extrusion	Annealed { Sheet Plate Rolled - Forms Extrusion	Annealed { Sheet Plate Rolled - Forms Extrusion	Mechanical - Hand Lay-up	Mechanical - Hand Lay-up
Type of Construction	 Stringers and Plate Integral Extrusion Welded Assy.	 Stringers and Plate Integral Extrusion Riveted Assy.	 Plate and Extrusion Welded Welded Assy.	 Plate and Extrusion Welded Welded Assy.	 Plate and Extrusion Welded Welded Assy.	 Multiple Frames Plate and Rolled Forms Welded Assy.	 Box Core Bonded Assy.	 Box Core Bonded Assy.
Module	1(a)	1(b)	4(a)	4(b)	4(c)	5(b)	8(a)	8(b)
Type of Construction	 Formed Stringers and Frame - Sheet Welded Assy.	 Extruded Web-Core Riveted Assy.	 Formed Stringers and Frame - Sheet Welded Assy.	 Multiple Frames Plate and Rolled Forms Welded Assy.	 Formed Stringers and Frame - Sheet Welded Assy.	 Sheet and Rolled Forms Resistance Welded Assy.	 Stringers - Frames and Skin Bonded Assy.	 Stringers - Frames and Skin Bonded Assy.
Module	2(a)	3	2(b)	5(a)	2(c)	6	7(a)	7(b)

Figure 10. Construction Modules

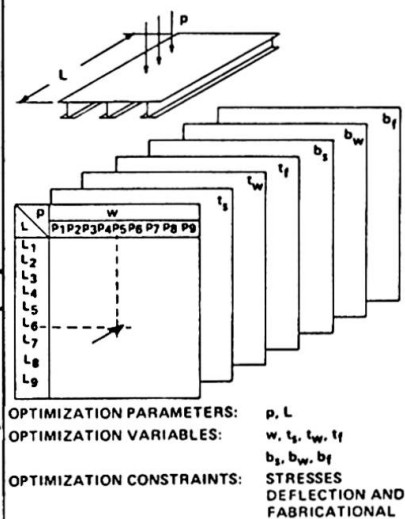


Figure 11. SEV Data Bank

The cyclic nature of the design process is apparent from the flow chart (Figure 12). It can be seen to be essentially a series of iterative looping operations which indeed permit the sequential consideration of all feasible possibilities.

The design process begins by selection of the appropriate SEV gross weight and construction module. The next choice is of the longitudinal bulkhead spacing from a list of allowable spacings. In SEV's cargo container size provides a lower bound on bulkhead spacing. For the specific longitudinal bulkhead spacing, allowable ranges of transverse bulkhead spacings are defined for each segment. In each segment the geometry is fully defined. Using the known local pressure loadings, the data banks are accessed for appropriate loads and geometry for each zone. The weight of a segment is computed and compared with that obtained for other transverse bulkhead spacings. This is repeated for each segment yielding the minimum weight design for the specified longitudinal bulkhead spacing. The entire looping is then carried out again for the next longitudinal bulkhead spacing and repeated to obtain the minimum weight craft.

Final checks on strength are performed using engineering analyses and where necessary incremental material is provided. For the ship system costing data is also computed.

The program then automatically cycles to the next construction module and SEV configuration, and repeats the entire process.

The above program was used extensively in the design of a range of SEV's varying from 500 to 10,000 tons gross weight.

Out of a total possible number of 232 ship designs, 173 were obtained. Designs for the remaining 59 configurations were not obtained due to the non-existence of minimum weight data for certain pressure/length combinations in the data banks. The availability of such data is directly dependent on the constraints placed on panel deflection, stress, and geometry in the process of generating the data banks. The constraints will yield, at times, nonfeasible panel designs and these appear as blanks in the data banks. If some of the constraints used in the component design are considered to be artificially severe, they may be modified. Using these less stringent criteria, additional ships designs would have been obtained.

Computational time was as low as 20 cpu seconds per ship design on an IBM 360/65 computer. The resulting output gave a very full description of the proposed structure including all scantlings, frame spacings and cost data.

As a second example of the use of the sieve-search procedure in a structural design process, its potential application to a bridge design problem is briefly considered.

For the purposes of a design study, a complete bridge structure may be broken down into the three major subdivisions,

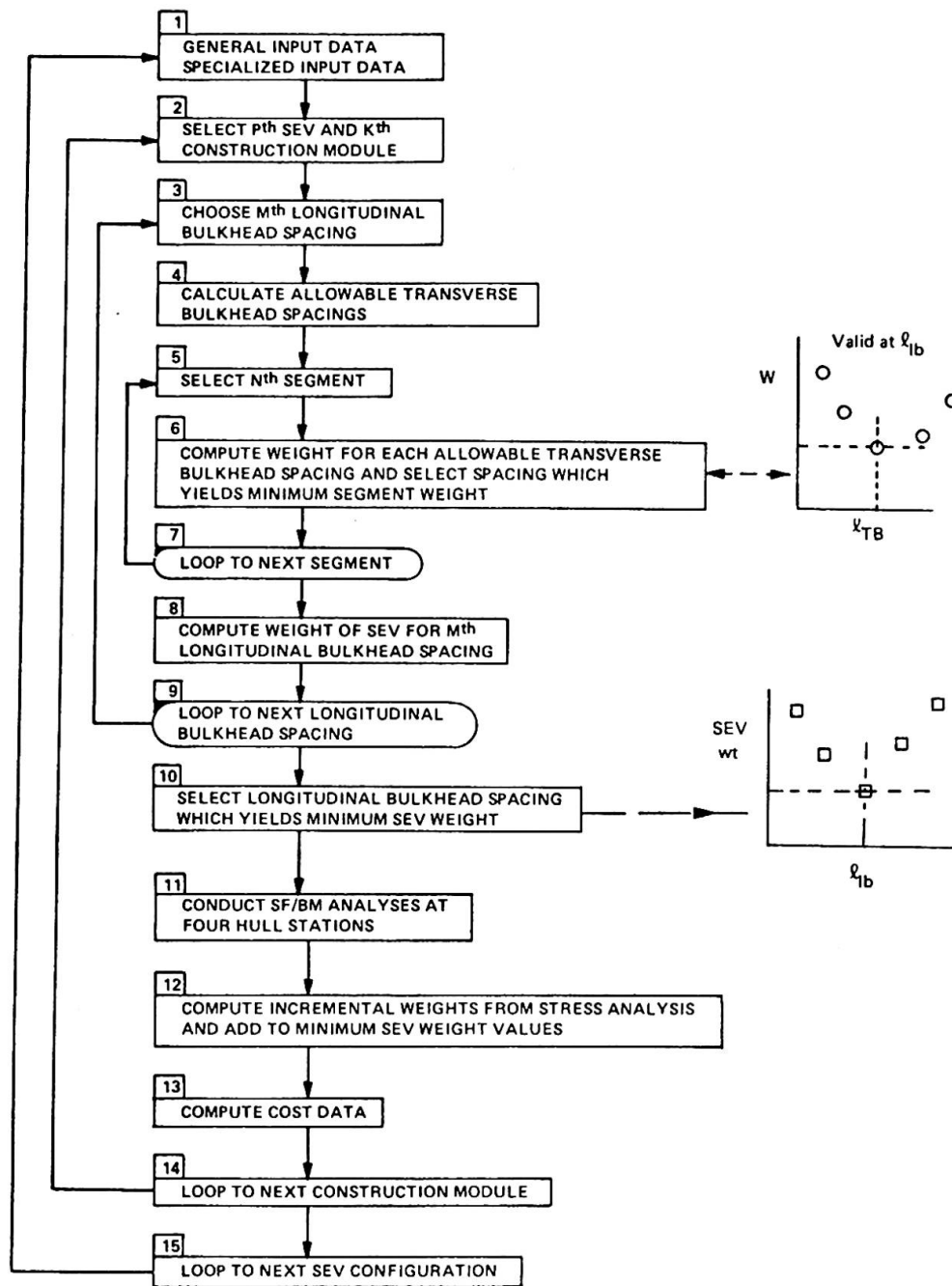


FIGURE 12. Sieve-Search for Minimum SEV Weight

- (i) Deck
- (ii) Primary structure spanning between piers and supporting the deck
- (iii) Substructure

The use of a sieve-search procedure for the optimal design of the deck and primary structure is outlined in a flow chart (Figure 13).

The essential characteristics of multiple levels of iterations with detailed design performed through the use of data banks is retained from the previous example, although the actual operations performed at each iterative stage may be totally different. For the bridge example the use of multiple data banks is deemed necessary.

The data banks for a bridge structure may contain a variety of different construction modules such as deck panels, plate or tubular girders, precast concrete beams, steel wide-flange beams with cover plates, cable arrangements or steel towers and concrete columns. All such potential bridge structural components may be pre-optimized on any suitable merit basis for suitable ranges of critical loadings and span lengths. The optimized data is then stored in banks readily accessible at the appropriate stage of the sieve-search program.

In selecting the bridge configuration a number of choices may exist and each may be programmed according to its intrinsic shape. Table I from Reference 28 indicates that for various spans alternate configurations may be possible, but engineering judgment and/or environmental conditions as well as other factors may narrow the choice of feasible designs.

For the deck construction, the most commonly used constructions are in-situ concrete, precast concrete and steel. Also experiencing growing popularity is the so-called orthotropic steel deck consisting of deck plate stiffened by parallel stringers. Some typical cross-sections may be found in Reference 28. In order to choose an appropriate deck, the following prime factors must be considered,

1. Strength, longitudinal and transverse
2. Dead weight
3. Cost

An efficient design includes the deck as part of the primary structure for load transferal and the true economic evaluation of the above three items may be successfully achieved when and only when the total bridge design is considered. For example, an orthotropic steel deck if viewed only as a slab will not compete in cost with reinforced concrete but the steel deck may be competitive if its axial force capacity and reduced dead load effects are considered through the complete superstructure and substructure designs.

The comparisons of all typical deck sections in context with the complete bridge structure are ideally suited for an automatic

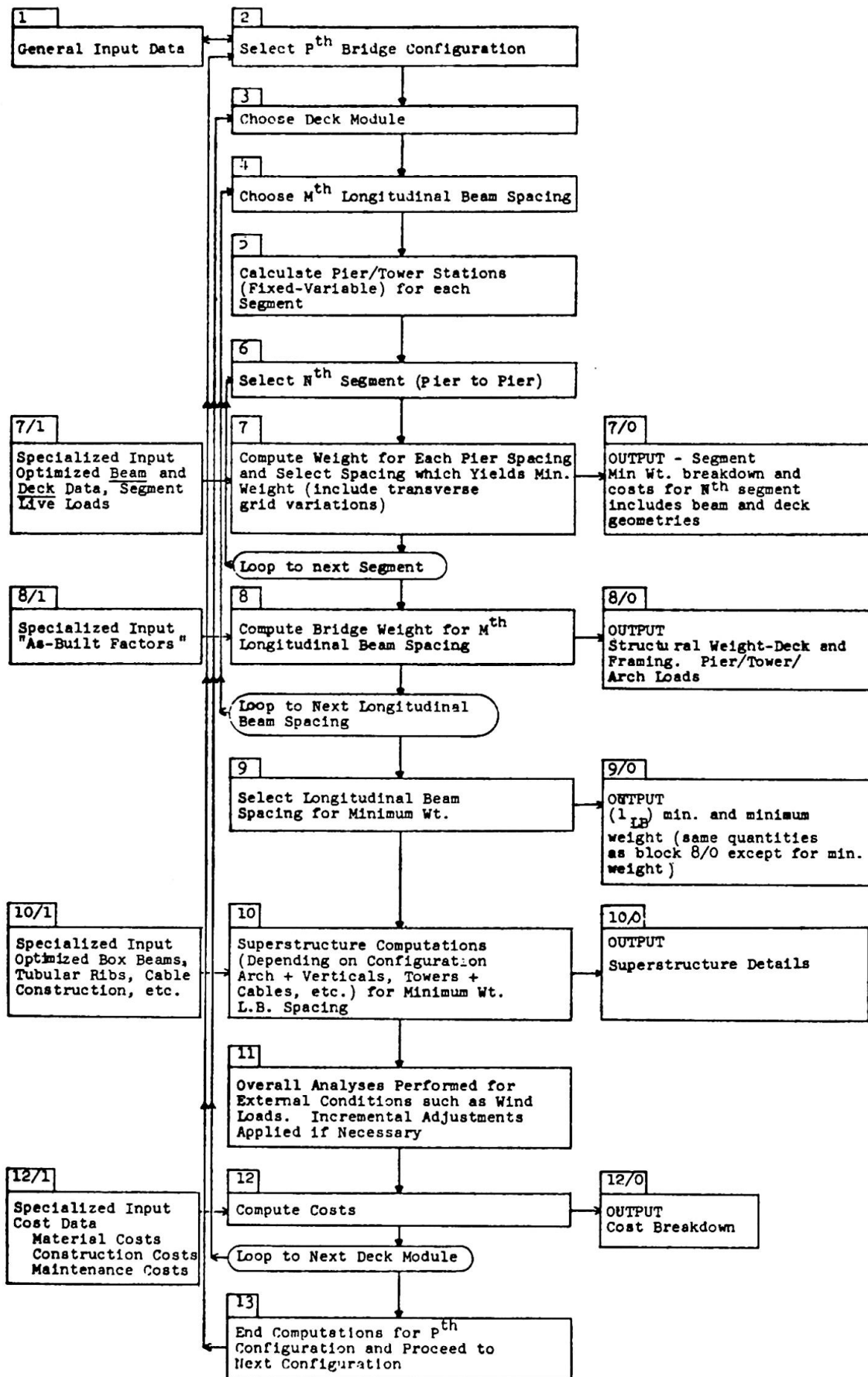


Figure 13. SIEVE-SEARCH FLOW CHART FOR BRIDGE DESIGN

sieve-search procedure. Data banks for each of the candidate deck cross-sections may be established. These files can be as sophisticated as desired wherein a range of span lengths together with a range of critical loads may be applied to each typical section. Figure 11 illustrates this and associated with each minimum weight (live + dead load), W_i , is a unit cost factor and optimal cross section geometries. The minimum weight and/or cost is evaluated under such constraints as deflection, strength, buckling, torsion, web crippling, etc. The definition here may be either working-load or ultimate. Fabricational limitations, code specifications and cost penalty factors may be included as well.

In the sieve-search, a predetermined table of acceptable longitudinal beam spacings may be specified, along with appropriate transverse spacings. The program will cycle through all the defined grids in its search for the optimum design. The configuration is also controlled by combination of fixed and variable lengths between abutments and piers. Each of the variable spans would be designated as a semi-independent segment for which a detailed design would be performed. For each segment, deck module and beam arrangement, the appropriate specialized data banks would be accessed to generate the local design which would then be compared with the previously stored optimal design. All segmental data is then assembled for the evaluation of the total design for a given longitudinal beam spacing. Specialized input, labeled "as-built" factors are provided to account for nonstructural items such as expansion joints, catwalks, railings, wearing surfaces, protective coatings, etc. After all potential longitudinal beam spacings have been considered, an interim optimal design is obtained. For this configuration, the superstructure is designed, again using appropriate specialized data banks. At this point a complete deck and superstructure have been designed and final check analyses should be performed. Some incremental adjustments on component sizing may be necessary. Consideration may even be given to the use of some suitable form of optimality-criteria optimization to refine a design, if this is felt to be appropriate.

Finally, the program would generate complete cost data for the selected design, including maintenance. The program is then repeated for other deck modules and configurations until the final design is rendered.

The preceding discussion has not been based upon an existing program but has been intended to indicate the possible extension of the sieve-search procedure to a civil engineering structure.

IV. CONCLUDING REMARKS

Two distinctly different approaches to the optimal design of structures have been presented. In both cases, the greatest possible emphasis has been placed on the practical aspects of the design problem in an attempt to produce a workable tool for the designer.

The optimality criteria approach is gaining acceptance by designers because of its fortuitous combination of simplicity and effectiveness. Computer programs based thereon are being used simply because no other method exists at this time that can cope with the very large number of variables encountered in finite element representations of real structures.

The use of the sieve-search procedure is a direct contrast in approach. The results obtained from the SEV design studies for an extremely modest expenditure of computer time, have indicated that this method is also an efficient cost-effective approach to automated optimal design. The ideal solution would possibly appear to be a combination of the two approaches, whereby the sieve-search defines configuration and noncontinuous variables and the optimality criteria method is used for refinement of the design. The extension of the procedures to other classes of design offers a considerable potential for overall system optimization.

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SUMMARY

Examples are presented of two approaches to the optimal design of complex structural systems. The first approach, based upon the use of optimality criteria is capable of optimizing finite element representations of large scale, complex structures with prescribed geometry. Both strength and stiffness constraints are considered. The second procedure is labeled sieve-search and is used for the overall optimization of structures. The method permits the full variation of construction method, materials and configuration as well as component sizing.

RESUME

Des exemples de calcul d'optimisation pour des systèmes de structures complexes sont présentées selon deux approches. La première, basée sur le critère d'optimisation, permet de résoudre des ensembles de grande dimension d'éléments finis, ou des structures complexes à géométrie donnée. Les contraintes de résistance et de raideur sont prises en considération. La seconde méthode, dite "sieve-search" (tamiser-chercher), sert à l'optimisation globale des structures. La méthode permet une complète variation de la méthode de construction, des matériaux, de la forme et des dimensions.

ZUSAMMENFASSUNG

Beispiele des Computer-Einsatzes bei der Optimierung von komplizierten Tragwerken sind nach zwei Methoden aufgeteilt. Die erste Methode wird das Optimierungskriterium benutzen, und erlaubt die Optimierung von komplexen Tragwerken mit einer bestimmten Geometrie, durch mächtigen Darstellungen finiten Elementen. Die zweite Methode, die "sieve-search" (sieben-suchen) heisst, wird für die globale Optimierung von Tragwerken benutzt. Sie erlaubt eine totale Bearbeitung der Baumethode, der Materialien, der Form und der Abmessungen.

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III

**Comportement des structures de bâtiments
sous l'effet des incendies**

**Tragverhalten von Bauwerken unter dem
Einfluss des Feuers**

Behaviour of Building Structures under Fire Effects

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IIIa

Thermal Effects of Fires in Buildings

Effets thermiques des incendies dans les bâtiments.

Thermische Auswirkungen bei Bauwerkbränden

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Fire load

The large building should be made as the fire resisting structure. The practical new design method for the fire resisting structures ought to be established from the engineering point of view - the conventional design has been based simply on the standard fire tests.

We should know severity of fire in compartment in the case of the fire resisting design.

The most important factor to control severity of fire is the amount of combustible materials in the compartment. Therefore, it is necessary to estimate the amount of combustible materials in the compartment to be designed.

Combustibles inside building can be divided into that of fixed combustibles such as beds of wall, ceiling, floor, partition equipment etc. and lining materials, fittings, fixed furnitures, and that of loaded combustibles such as furnitures, books, clothes and other stored materials.

Since combustible materials in the room consists of various kinds of materials of different calorific value at combustion, the amount of combustible materials expressed in terms of the weight of wood that would produce by combustion heat equal to the heat content of the materials. this is the older definition of fire load.

Now, the heat content of combustible materials per unit floor area of fire compartment is called the fire load density, which is a basic element for analysis of fire severity.

$$q = \frac{\sum(G_i \cdot H_i)}{A} = \frac{\sum Q_i}{A}$$

where, q : fire load density (Mcal/m²)

G_i : weight of combustible materials (Kg)

H_i : unit calorific value of combustible materials
(Mcal/Kg)

A : floor area of fire compartment (m²)

ΣQ_i : total heat content of combustible materials in fire compartment (Mcal)

Actual conditions of fire load have not yet been understood correctly. It is presumed that the amount of fire loads have greatly decreased by use of steel furnitures with a latest change in living mode. For this reason, several countries are trying to investigate into actual conditions of amount of combustible materials to promote rational design of fire resisting structures. Since the amount of fire loads, depends on living mode, it seems to be necessary to perform investigations on actual conditions of fire load in each countries independently.

The amount of loaded combustible materials in modern office buildings of Japan is shown in Fig. 1.

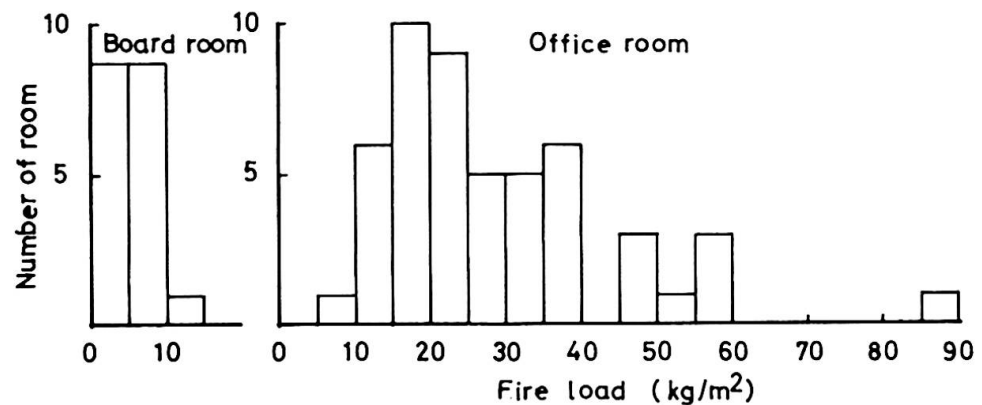


Figure 1. Distribution of fire load in room of modern office buildings.

Temperature History

The temperature history during a fire is illustrated in Fig. 2. The period A-B is the growth period which is quite long or short duration depended on a fire origin.

But a later stage of this period, development and spread of fire can be rapid as shown B-C, particularly when a room is lined with highly flammable materials or is filled with many furnitures made by plastics. When an enclosure becomes fully involved in fire and air temperatures have suddenly risen, flashover is said to have occurred. The period B-C is sometime said pre-flashover.

When flashover occurs smoke development and temperature rise in the compartment are so rapid that there is a life hazard inside building. We must consider the reduction of fire severity of flashover in designing buildings for life safety. Fire severity at flashover seems to depend on the size of fire origin, the area

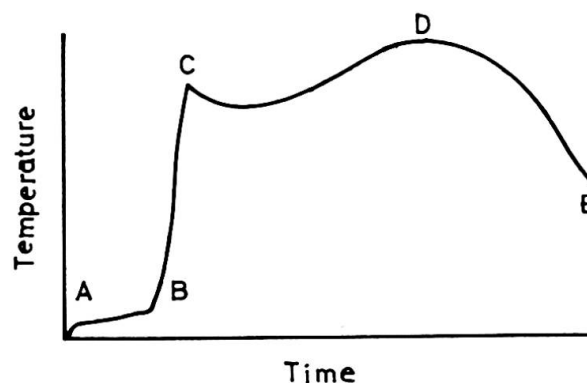


Figure 2. Air temperature history in a room.

of opening, and the quantity and quality of combustible materials including the internal linings, however, the theoretical approach to this phenomenon seems to be quite difficult. The second international co-operative study in working commission W-14 of CIB, which is to examine systematically the pre-flashover stage of fire, should yield very useful results.

As the mechanism of flashover is not clearly understood, the methods for testing combustibility of materials are quite different in many countries. Such lack of unity of testing method is rarely found in other fields. The working group of TC-92 of ISO, in which test method for fire are discussed, is expected to provide suitable fire test methods.

After flashover in a room the fully developed period C-D begins, in which period nearly steady state can be continued, although the air temperature in a room gradually rises. After the most combustible materials is burnt out the temperature starts to fall and the decay period D-E begins.

The problem of predicting the behaviour of fully developed fires in compartments is central to the structural design of fire resistance requirements for buildings. In spite of a large number of important investigations, our present state of knowledge on the detail characteristics of compartment fires is far from satisfactory. Somewhat simplified, we considered that fully developed compartment fires could be divided in the two types of behaviour[1].

The first regime of behaviour was well known ventilation controlled fire and the second was fuel surface controlled one.

Recently a new study has proposed the third regime which is controlled by fuel porosity[2].

Rate of Burning

The rate of burning after flashover in a room with normal openings is approximately proportional to the volume of inflow air through the openings. From the theory of ventilation based on buoyancy, and the basis of experiments, carried out in Denmark, Japan, Sweden, UK, USA and USSR, the following relation has been obtained[3,4]

$$R = (5.5 \sim 6.0) A \sqrt{H} \dots \dots \dots (1)$$

where R : rate of burning (kg/min)
A : opening area (m²)
H : opening height (m).

This relation yields results approximately in agreement with experiments of fire in compartment with normal openings as shown in Fig.3 [5].

Alternatively, Eq(1) can be given in the following dimensionless form

$$R/\phi = 0.0236 \dots \dots \dots (2)$$

where ϕ is a ventilation parameter of the same dimensions as the rate of burning R and defined by the ratio

$$\phi = \rho_a \sqrt{g} A \sqrt{H} \dots \dots \dots (3)$$

ρ_a is the density of air and g the acceleration due to gravity[5]. With $\rho_a = 1.29 \text{ kg/m}^3$ and $g = 9.8 \text{ m/sec}^2$, Equation(2) corresponds to the value $5.7 \text{ kg.min}^{-1} \text{ m}^{-5/2}$ of constant in Eq(1).

The experimental data, given in Fig. 3 are characterized by

a considerable scatter. In spite of this, different regimes are recognizable, viz. The ventilation controlled regime, marked by an inclined line, and the fuel bed controlled regime, marked by a horizontal line.

For ventilation controlled fires an accuracy is sufficient in most practical cases of a structural fire engineering design. For fuel bed controlled fires the present state of knowledge is too incomplete for enabling a satisfactory. It seems reasonable to base a structural fire engineering design on the assumption of the fire to be ventilation controlled which will be on the safety side in the most case.

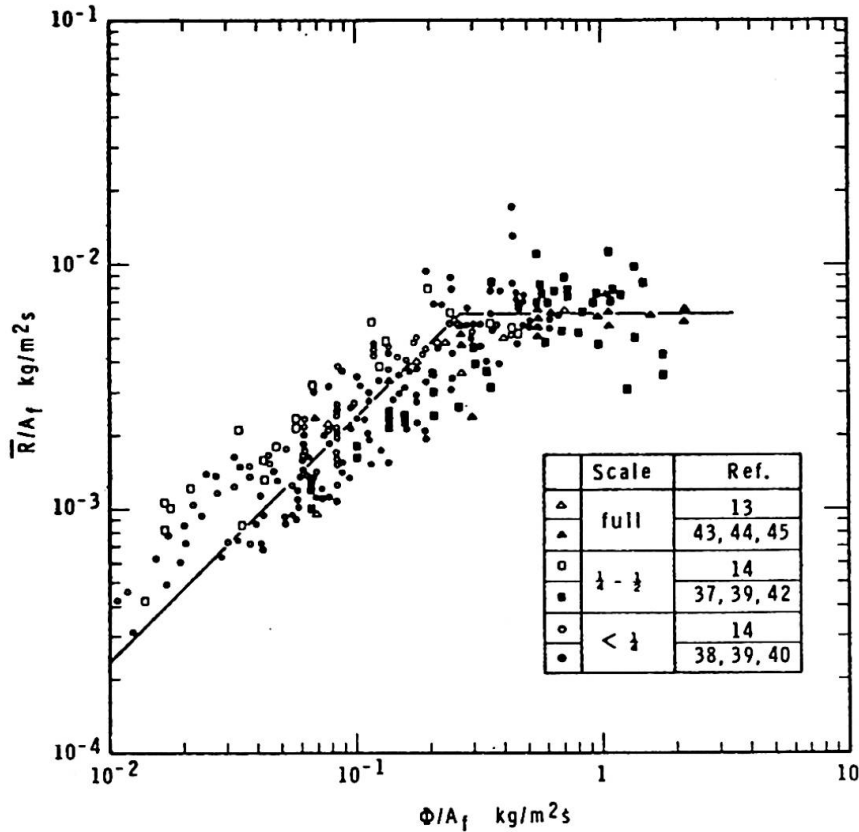


Figure 3. Correlation of experimental data concerning rate of burning in compartment fires.

Temperature-time Curve

When the rate of burning is known, the rate of heat release in a room can be estimated roughly. An equation of heat balance is as follows,

$$Q_H = Q_W + Q_B + Q_L + Q_R \dots\dots\dots (4)$$

where Q_H : the rate of heat release in the room

Q_W : the rate of heat loss by heat transfer to the interior surfaces

Q_B : the rate of heat loss by radiation through opening to the outer air

Q_L : the rate of heat loss carried away with the spouting flame from opening

Q_R : the rate of heat loss necessary for the air in the room warmed up to the fire temperature-ordinarily negligible.

By combining the heat conduction equations for the ceiling, walls and floor with the heat balance equation, it is possible to estimate a fire temperature vs. time curve in the room by numerical calculation[6,7].

The lower the thermal conductivity of the interior surface the higher the air temperature. If the thermal properties of all the interior surfaces and the height of the opening were assumed to be nearly the same for different rooms, the fire temperature would generally depend on the ratio between the area of the opening and the total surface area of the room.

After most of the fuel is burnt out, the rate of burning decreases but the remaining fuel on the floor continues to burn for a long time. For a full-scale fire test the temperature decreases at the rate of $5\sim 10^{\circ}\text{C}/\text{min}$. As the rate of burning is influenced by several factors in this decay period, it is difficult to determine the exact duration of fire. It is convenient in engineering point of view to determine the duration of the fire on the assumption that the rate of burning is always constant from beginning to end of a fire. This assumption errs on the side of safety.

If the height of the opening and the fire load per unit floor area were the same, the fire duration would depend on the ratio between the floor area and the opening area.

Relation With The Standard Fire Test

Although it is possible to estimate the temperature vs. time curve for a given room, the standard fire test is usually carried out by using a standard temperature-time curve. To find the relation between the standard and estimated curves, the area between the standard temperature-time curve and the line of the failure temperature for steel (which is around $400^{\circ}\sim 500^{\circ}\text{C}$) can be taken as the criterion, because when a structural member is heated by both temperature vs. time curves, maximum temperature rises of steel in side structural member are almost same as shown in Fig. 4[6]. Thus we could deduce the equivalent fire duration corresponding to failure according to the standard fire test.

For a long time, fire duration has been considered to be proportional to fire load. In some country still this relation is the basic philosophy of the structural design of fire resistance requirements although a lot of correction factors is introduced in the fire load.

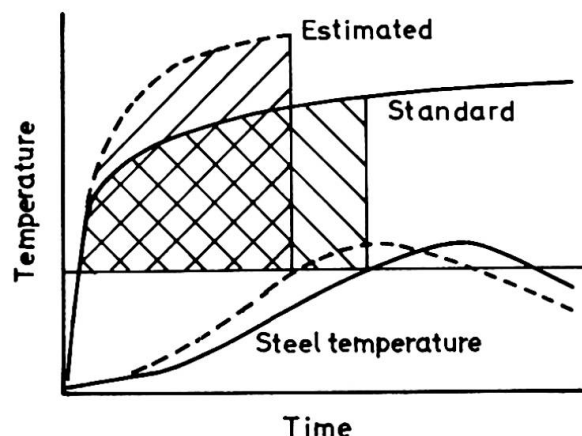


Figure 4. Equivalent standard fire test time.

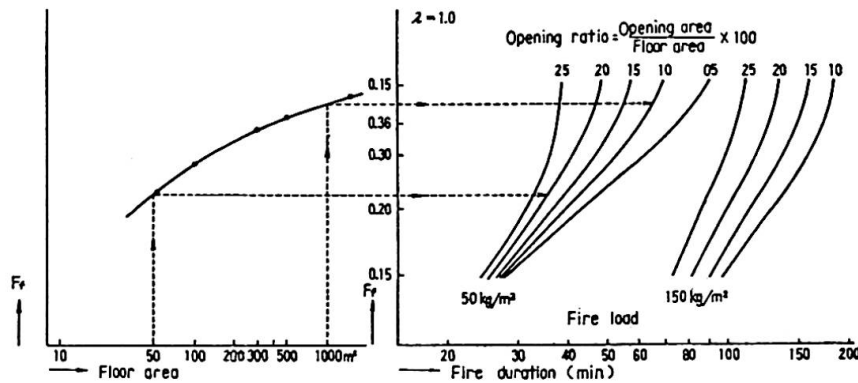


Figure 5. Relationship between fire duration, opening ratio and floor area (assumed that the ceiling height is 3.5 m)

Fig. 5[8] shows the relations between fire duration, opening ratio (area of opening/floor area) and floor area. We find that the fire duration is influenced by the floor area as well as the opening ratio. The larger the floor area of a compartment, the higher becomes the requirement for fire resistance under the same fire load.

Although at this time there are still many unknown variables it would be useful to establish a method for obtaining a temperature-time curve in a room for the structural design of fire-resistance even though it would be only a rough approximation.

Necessary Height of Spandrel

It is possible to predict flame height according to the following relation [9,10]

$$L/D \propto (R^2/D^5)^{1/3} \dots\dots\dots (5)$$

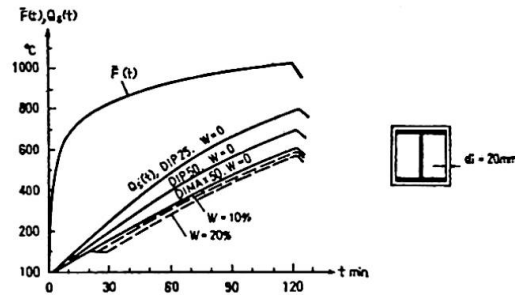
We can apply this relation to the estimation of the spandrel height in order to prevent vertical fire spread from window to window. The flame emerging from a wide window as commonly found in modern buildings, clings to the wall surface over a long distance[11]. It is therefore necessary to break up the flame by making projections from the wall such as a balcony or some other device.

Estimation of Temperature Rise in the Structural Member

As mentioned before, it is not a reasonable requirement that the fire-resisting ability of each structural member should always be determined only by a standard fire test, even though it is required by building regulations. This limitation restricts building design in sometimes uneconomical and even dangerous ways.

For example, if steel columns have different thicknesses, even though they have the same outside dimensions and are protected with the same materials, their rates of temperature rise would be different because of their different heat capacities [12,13] as shown in Fig. 6. It is, therefore, wrong to prescribe the thickness of fire protection material without consideration of the heat capacity of steel.

Figure 6. Temperature rise of steel column



It would be desirable to predict the temperature rise of the inside steel by means of heat flow calculations in each component of the structure.

The effect of big cracks or spalling of the cover should also be included in the heat flow calculations under some assumptions based on the fire tests. The thermal properties of building materials at high temperature are fairly well known, and heat flow calculations have been done in many countries. It must become more efficient and more reliable to calculate fire resistances for buildings rather than to determine them by fire tests. This would help to rationalize building design.

In line with the method of the estimation of temperature vs. time curve described previously, we can already calculate the heat conduction in structures surrounding on compartment.

Mechanical Properties of Structural Steel at High Temperature

Some research results are available concerning mechanical properties of structural steel at high temperature. Depression of yield point of hot rolled steel at high temperature is shown in Fig. 7 and 8[14].

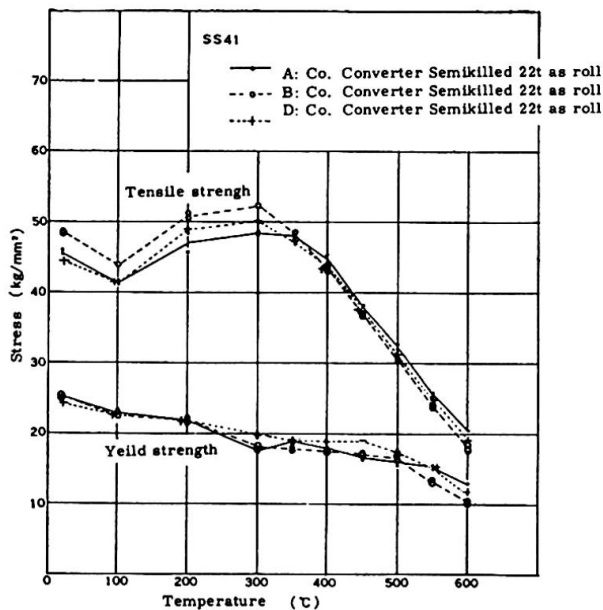


Fig. 7. High temperature properties of SS41

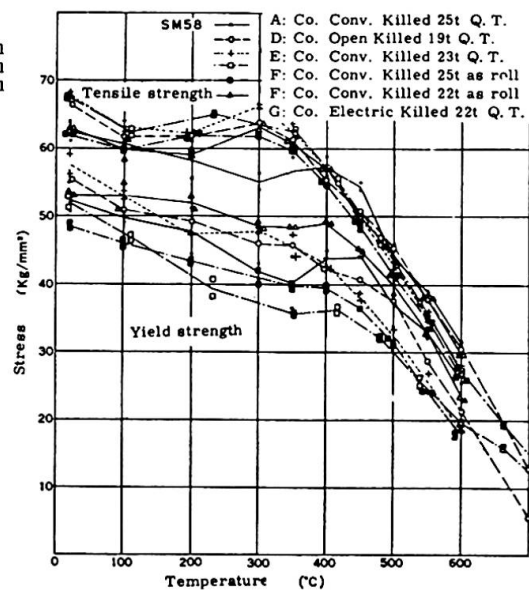


Fig. 8. High temperature properties of SM58

Rate of yield point depression shown here is the most important factor for fire resisting properties of steel structures. Supposing that a simple tensile member is heated when it has stress of 50% of yield point at a normal temperature by an action of tensile stress. Steel temperature naturally rises and yield point decreases. When yield point decreased as low as a half of that at a normal temperature, the member is destroyed by plastic deformation. The relation between this existing stress value and steel temperature at yield time is specified by yield point decrease ratios shown in these Figs. The yield point decrease ratio at high temperature depends on kind of steel.

Since mechanical properties at high temperatures and after heating depend on kinds of steel materials like this, full attention should be paid to mechanical properties of steel at high temperatures and after heating in evaluating fire resisting properties of steel structural members.

There has been almost no systematic research concerning high temperature strength of structural steel materials, however, a systematic experimental research on representative kinds of structural steel produced in Japan was carried out in recent years by the Society of Steel Construction of Japan, and systematic data were prepared.

Fire Resistance of Structures

The fire resistance of structural elements has, in the past, been determined only by the Standard Fire Test. Recently the study of structural behaviour at elevated temperatures has advanced remarkably. It is believed that the fire endurance of structural elements is influenced by the reduction of strength of materials and by the thermal stresses produced in the structure by restraint.

Steel Construction

When a simply supported flexural member of steel construction is heated from the lower side, its deflection increases in accordance with the differential thermal expansion between the upper and lower parts of the member, and with the reduction of the elasticity of steel. The collapse of the member occurs at the time when rise in temperature reduces the yield point to the working fibre stress of the member. The relation, shown in Fig. 9, is obtained from fire tests on beams [15]. A more precise analysis of the mechanism of failure has to be based on the creep behaviour of steel at elevated temperatures [16].

The allowable buckling stress of a column is reduced in accordance with the reduction in the values of the mechanical properties of steel caused by rise of temperature, as shown in Fig. 10[17].

In discussing fire resisting properties of structural members principally bending and buckling strengths of simple support members have been discussed. However, fire resisting properties of structural members of real building are strongly influenced by restraining condition of ends of those members and show conditions considerably different from cases of simple support members.

When structural members were heated during fire, elongation of members caused by temperature increase inside section and deflection caused by unequal temperature distribution inside section generally appear. Although it poses no problem for member

without restraint of its ends since deformation appears as it is, ordinary structural member produce internal stress since deformation is restricted by end restraint. If this internal stress, that is to say, thermal stress by heating during fire, increases, members are destroyed and structural strength sharply decreases. The influence of this thermal stress on building structure is a problem to be considered carefully. For steel structural members, thermal stress by deflection in general needs not to be considered since temperature distribution inside section is comparatively uniform. However, thermal stress of steel structural members is very large when axial elongation is restricted at ends because of large change in member length.

From results of theoretical and experimental researches, it was clarified to be almost impossible to insure steel structural members with end restraint without damage during fire.

Increase ratios of thermal unit stress and thermal deformation greatly depend on the value of end restraint factor. Although an origin is different depending on the value of initial existing unit stress, thermal unit stress increases almost linearly with an increase of temperature, and when it reaches buckling unit stress at high temperature determined by slenderness ratio of member, it causes buckling destruction. Fig. 11 shows an example of experimental results [18]. For ordinary building structures, end restraint factor K is determined by load-deformation property of members giving restraint. Therefore, in the case that heated member is destroyed at a comparatively low temperature when K is large, members giving restraint do not suffer damage, on the other hand, in the case that heated member is not destroyed until it reaches a considerably high temperature when K is small, members giving restraint suffer forced deformation to a considerable degree. This can be shown as in Fig. 12.

In general, it can be considered that beam is heated while pillar gives restraint, therefore, if forced deformation such like outside thrusting of pillar is not prevented by strengthening end

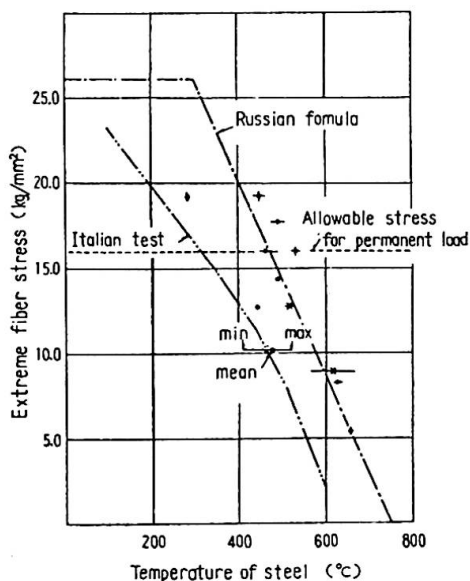


Figure 9. Relationship between working stress and temperature of steel at yielding failure

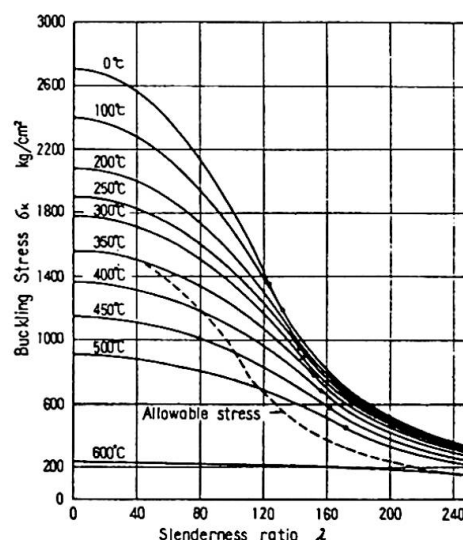


Figure 10. Buckling stress of steel column at elevated temperature

restraint degree against beam with enhancing rigidity of pillar, there is danger of collapse of the whole building. In other words, it is needed to try relaxation of thermal stress by partially destroying beams on purpose at an earlier time.

However, since multiple span structure is generally used and end restraint against central beams shows a very large value, central beam at first suffers partial buckling at a considerably low steel temperature during fire, and it is transferred to structure with a halved number of span. If partial destruction of beam develops by turns like this, forced deformation against pillar can be presumed to be not so large.

In the case that elongation restraint of end of member is acting on eccentrically, conditions are quite different from the case of central restraint. Test results of this case are shown in Fig. 13[18]. When bending moment by restraining force appeared with an increase of steel temperature reached a value of yield bending moment of member, increase of thermal unit stress stops. If steel temperature further increases, yield bending moment of member gradually decreases. In such a case, rapid destruction does not occur.

In ordinary steel structures, about 200°C in an average temperature of section of member is a critical temperature for partial destruction by thermal stress. Against this situation, it is necessary to take a measure to prevent partial destruction by designing sufficient fire covering or to loosen thermal stress of member relating with structural stability by partial destruction on purpose.

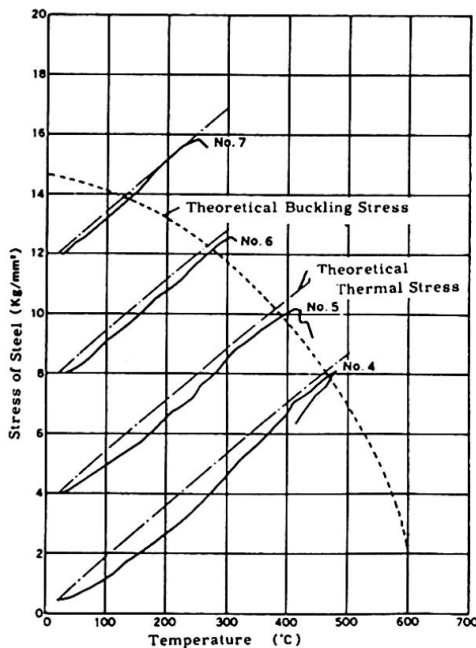


Fig. 11. Test results (central loading)

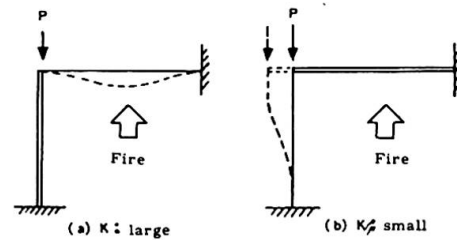


Fig. 12.

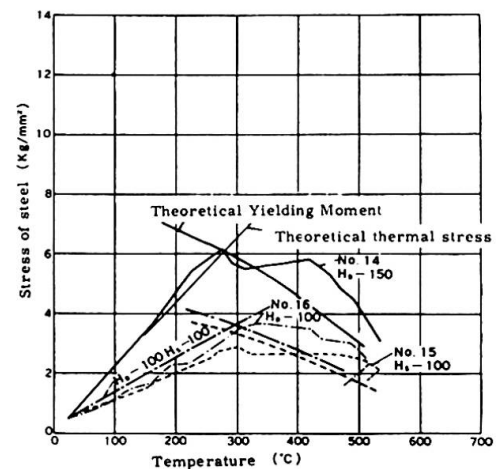


Fig. 13. Test result
(Eccentric loading)

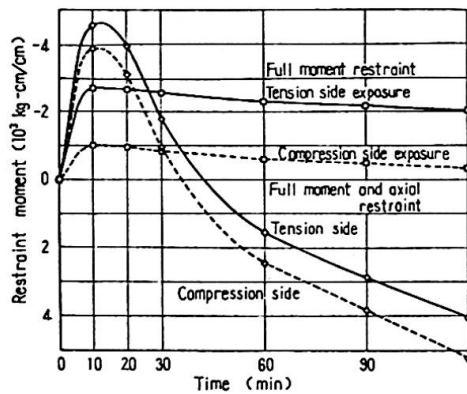


Figure 14. Restraint moment of reinforced concrete beam

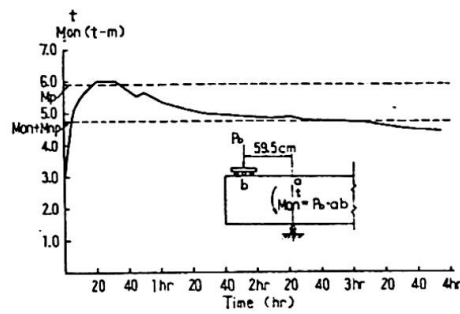


Figure 15. Restraint moment

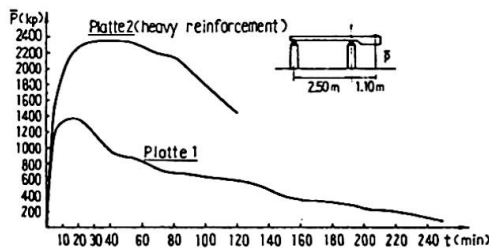


Figure 16. Continuous slab 1 and 2. Re-distribution of tension force P

Reinforced Concrete

The structural behaviour of reinforced concrete members in fire is affected by the mechanical properties of the concrete and of the reinforcement at elevated temperatures.

It is possible to estimate the internal thermal stresses and the deflection of each member theoretically, by using data on the thermal expansion of the concrete and of the steel bar and on the reduction of their elasticities with temperature [19,22]. The time of collapse is determined either by the ultimate strength of the concrete or by the effect of the elevated temperature on the steel bar.

The structural behaviour of a beam restrained at the both ends is shown in Fig. 14, 15 and 16 [19-21].

In the case of prestressed concrete structures, the strength of steel at the elevated temperature is different from the usual steel bar and the initial stress of concrete is also higher than for ordinary reinforced concretes, but the structural behaviour of a member exposed to fire is essentially the same as for ordinary reinforced concrete members.

Some mechanical properties of concrete and steel bars at elevated temperatures, especially the stress strain curves, have been investigated experimentally by a number of laboratories.

Spalling of Concrete

The explosive breaking off of pieces from concrete materials during fire exposure is known as spalling. Reinforced or prestressed concrete structure has explosively spalled in the early stage of fire. In general spalling reduces the fire resistance of a structure. Sometimes the concrete structure with thin thickness loses their function as the structural member by

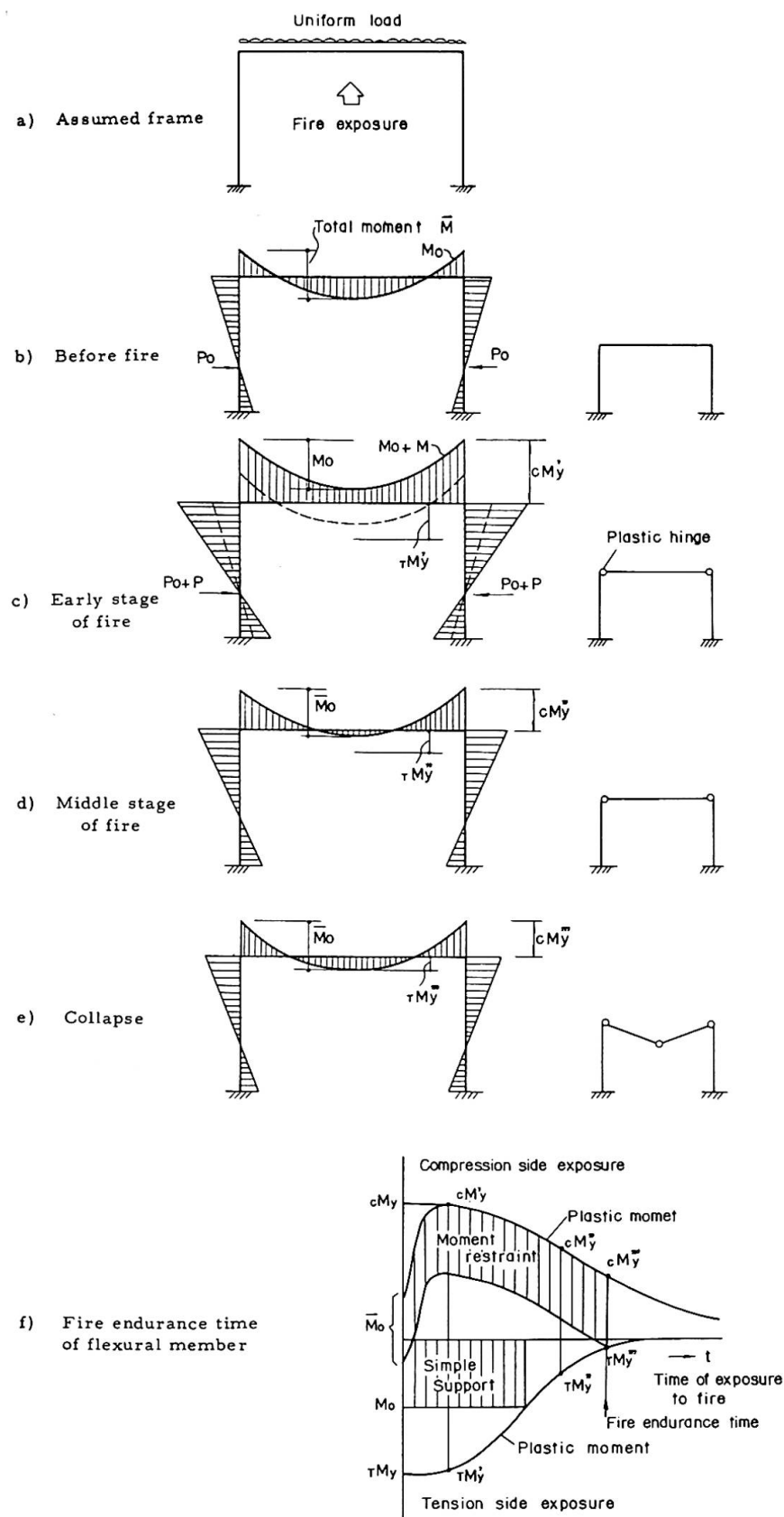


Fig. 17. Redistribution of moment and mechanism of collapse in building fire

the explosive spalling of concrete, while thick structural member does not cause structural failure except that part of the concrete surface falls to pieces by spalling. Therefore it becomes very difficult to protect a building from fire when concrete structures are used. Thus, the phenomenon of spalling of concrete structure under fire can be regarded considerably important among other happenings caused by fire.

At present, however, the cause for this explosive spalling of concrete when exposed to fire has not been thoroughly explained. There are numerous factors which influence spalling. From experimental and theoretical data it can be derived that the most common types of spalling are spalling mainly due to thermal stress of a concrete near the surface [23-25], and spalling mainly due to formation of steam at high pressure in the concrete [26].

Effect of Restraint

In the case of general, structural member is fixed to other members at the end. Therefore, it can be considered as having an elastic restraint against expansion and rotation. Suppose the beam in Fig. 17-a, which receives uniform load, is heated at the bottom. Bending moment of each member distributes as shown in Fig. 17-b. When the beam is heated, distribution of bending moment moves upwards in parallel because of axial and moment restraint as shown in Fig. 17-c. If stiffness of restraint member is large, this change grows large.

Moreover, magnitude of end moment ($M+M_0$) differs by reinforcement at the end of beam and it grows up to the value of bending moment cM_y at which end section of beam being heated on the compression side causes failure. Since value of cM_y decreases with accordance of the passage of time, ($M+M_0$) at the end, lowers along with the passage of heating time after it reaches cM_y and then becomes cM_y'' and cM_y''' . Because total moment by dead load and external load of beam is definite, moment in the center of span increases by gradually. When moment in the center of span grows as large as the value of bending moment $\dagger M_y'''$ at which section heated on the tension side causes failure, beam as a whole makes sudden transformation to result in collapse, as shown in Fig. 17-e.

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SUMMARY

The inadequacy of traditional design philosophy for fire protection in accordance only with a building code is discussed.

The characteristics of compartment fires are reviewed. And the fire endurance of structural elements is influenced by the reduction of strength of materials and by the thermal stresses produced in the structure by restraint. Therefore, the influence of the thermal stress is a problem to be considered carefully.

A new fire engineering design approach is considered.

RESUME

Les dispositions constructives traditionnelles pour la protection contre l'incendie ne sont pas satisfaisantes lorsqu'elles sont basées uniquement sur un règlement de construction.

Les caractéristiques des incendies sont passées en revue. La résistance à l'incendie des éléments d'une structure dépend de la résistance des matériaux et des efforts thermiques produits dans la structure. L'influence des efforts thermiques doit être étudiée sérieusement.

Une nouvelle méthode de dimensionnement au feu est proposée.

ZUSAMMENFASSUNG

Bezüglich des Brandschutzes unterstreichen die Autoren die Unzulänglichkeit einer traditionellen Entwurfskonzeption, welche nur auf Baureglementen basiert.

Die charakteristischen Eigenschaften von Feuereinwirkungen in Brandabschnitten werden untersucht. Der Feuerwiderstand von Tragelementen wird durch die Abnahme der Materialfestigkeit und durch die thermischen Zwängungsspannungen beeinflusst. Diese Zwängungsspannungen verdienen daher besondere Aufmerksamkeit.

Es wird ein neues ingenieurmässiges Entwurfsverfahren für den Brandschutz dargestellt.

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IIIb

Calcul et conception des structures métalliques ou mixtes en vue de leur résistance à l'incendie

Bemessung von Stahl- und Verbundbauwerken gegen Brandeinwirkungen

Design of Steel and Composite Structures for Fire Resistance

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Comportement au feu des structures en acier ou mixtes

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1. CONSIDERATIONS PRELIMINAIRES

Toute structure de bâtiment comporte certains risques de dommages en cas d'incendie. L'importance de ces risques dépend de nombreux facteurs ayant trait aussi bien aux caractères du bâtiment qu'à son utilisation et à son contenu, ainsi qu'aux éventuelles mesures de prévention et de protection.

Les dangers encourus peuvent être classés en trois grandes catégories, selon qu'ils concernent les personnes, les biens matériels ou le bâtiment même. Le présent rapport ne se réfère qu'à cette dernière catégorie de risques et traite uniquement des bâtiments à structure portante en acier ou mixte (acier-béton).

L'étude du comportement des structures de bâtiments sous l'effet des actions thermiques d'un incendie a progressé considérablement au cours de la dernière décennie, les résultats les plus significatifs étant enregistrés dans le domaine des structures en acier.

Une meilleure connaissance des phénomènes, en fonction des conditions et circonstances précises dans lesquelles ils se produisent et se développent, permet de serrer la réalité de plus près et de mieux doser les mesures destinées à limiter les risques et leurs conséquences. Or, la résistance des structures n'est qu'un aspect partiel dans l'ensemble des problèmes de la sécurité vis-à-vis de l'incendie. Il convient donc, bien sûr, d'assurer cette résistance dans toute la mesure nécessaire, mais sans exagération inutile et anti-économique, qui serait éventuellement réalisée aux dépens des mesures de sécurité prioritaires, concernant par exemple directement les personnes.

La conscience de la nécessité d'une connaissance aussi précise que possible du comportement au feu des structures a largement contribué à susciter et à encourager les recherches dans ce domaine, dont les résultats sont sommairement reflétés dans le présent rapport.

2. ASPECTS FONDAMENTAUX DU COMPORTEMENT AU FEU DES STRUCTURES METALLIQUES

2.1. Effets du réchauffement sur les propriétés des aciers de construction

Le processus de réchauffement d'un corps, sous l'effet d'un flux thermique donné, dépend notamment de la chaleur spécifique et de la conductivité thermique du matériau. Ces propriétés caractéristiques sont assez bien connues pour les aciers de construction, en fonction de leur composition chimique. En particulier en ce qui concerne la conductivité thermique -propriété qui détermine le gradient de température dans la phase de réchauffement- les études expérimentales ont montré qu'elle varie en fonction du niveau de température et, plus précisément, qu'elle subit une légère diminution au fur et à mesure que la température monte.

La complexité des données fondamentales et les dispersions inhérentes aux études expérimentales dont elles résultent, conduisent cependant à l'adoption de valeurs considérées moyennes dans le domaine d'application pratique envisagé. Le degré de précision de ces valeurs semble acceptable en ce qui nous concerne.

L'élévation de température due aux actions thermiques d'un incendie exerce une influence directe sur les principales caractéristiques mécaniques de l'acier, qui déterminent la tenue des éléments de construction : limite d'élasticité σ_y , résistance à la rupture σ_R et module de déformation E .

La variation de ces caractéristiques en fonction de la température a été étudiée expérimentalement par divers chercheurs. On connaît ainsi avec assez de précision l'abaissement progressif de la limite d'élasticité et du module de déformation E , dû à l'élévation de température. On a également constaté, pour la résistance à la rupture en traction, un léger accroissement à partir de l'ambiante, avec un maximum vers 200-250° C, suivi d'un abaissement progressif de résistance au-delà de cette température. Enfin, il est bien connu que les aciers de construction présentent certaines caractéristiques de fluage pour des températures supérieures à 400-450° C (figure 1).

Les diagrammes de traction obtenus expérimentalement sur des aciers courants de construction sous diverses températures montrent, par exemple, l'abaissement progressif et finalement la disparition de la limite d'élasticité avec l'élévation de la température. Implicitement, ils font ressortir la variation du module de déformation E , ainsi que le fluage prématuré aux températures élevées (figure 2).

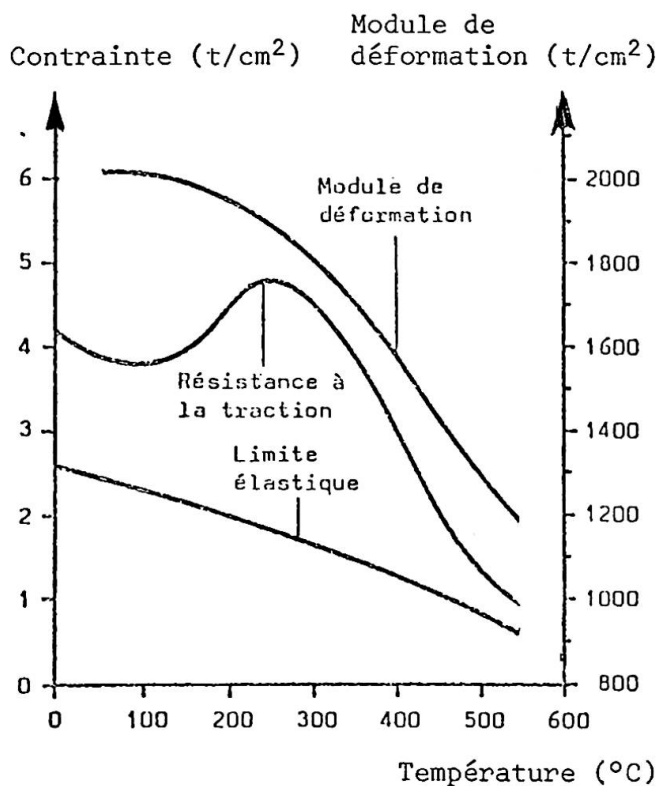
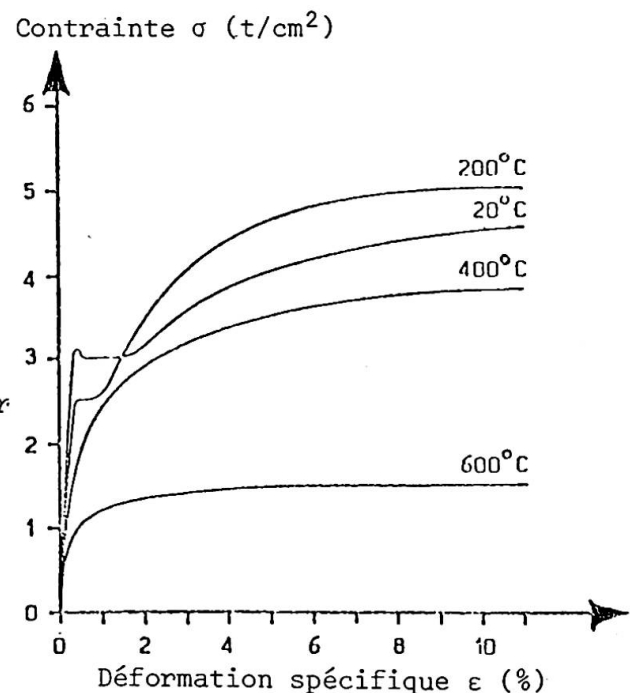


Figure 1

Caractéristiques de l'acier Ac 24/37 à différentes températures.

Figure 2

Diagramme contrainte-déformation de l'acier doux ASTM A.36 à différentes températures



Les résultats de ces recherches accusent une certaine dispersion due sans doute aux conditions d'essai respectives, mais les conclusions qui s'en dégagent sont suffisamment concordantes pour être acceptées comme bases des études de la tenue au feu des structures métalliques. Or, la connaissance du comportement de l'acier sous l'effet des températures élevées est une condition préalable pour le dimensionnement des éléments de construction exposés à de telles températures.

2.2. Critères de référence pour la tenue d'un élément de construction

De même que pour l'étude de résistance et de stabilité des structures et de leurs éléments constitutifs sous l'effet des charges statiques ou dynamiques, l'étude de la tenue au feu doit se rapporter à une certaine limite conventionnellement ou réglementairement adoptée comme critère de référence.

Sur un plan général, une telle limite correspond à un seuil accepté comme représentatif de la cessation d'une fonction essentielle de l'élément ou de la structure. Pour un élément porteur, cette limite peut traduire soit la rupture, soit l'affaissement ou une déformation excessive, annulant ou compromettant sa capacité portante : il s'agit alors d'un critère de ruine. On peut cependant aussi envisager la référence à d'autres critères, liés à des aspects fonctionnels de "serviceabilité".

Pour ce qui est de la tenue au feu, le critère de référence peut être défini de différentes manières. En effet, il peut se traduire soit par une limite de déformation, soit par l'affaissement de l'élément, soit enfin par une température critique.

Ainsi, par exemple, les règlements belge et français prescrivent, pour les éléments sujets à flexion, un critère de ruine très simple représenté par une limite de flèche à 1/30 de la portée.

Ce critère présente l'avantage de la simplicité, mais il se prête mal à la définition d'une température critique, qui dépend de nombreux paramètres qui n'apparaissent pas dans cette approche trop sommaire et globale.

Un autre critère, largement appliqué dans la pratique expérimentale, est défini par rapport à une "flèche spécifique", ou vitesse de déformation, en application de la formule préconisée par Ryan et Robertson :

$$\frac{\Delta_f}{\Delta_t} = \frac{l^2}{9000 h} \quad \text{cm/min}$$

où Δ_f , l et h sont respectivement l'accroissement de la flèche, la portée de l'élément et la hauteur de la section (en cm)

et Δ_t l'intervalle de temps considéré (en min).

Un tel critère fondé uniquement sur la déformation peut servir pour définir conventionnellement le point de ruine et se traduire par une température critique lorsqu'il s'agit d'un élément isostatique dont les caractéristiques géométriques sont bien définies par ailleurs.

Il est cependant possible de définir la température critique indépendamment du système statique, comme étant celle pour laquelle la vitesse de déformation devient infinie :

$$\frac{\Delta_f}{\Delta_t} = \infty$$

Ce critère est facile à établir. De plus, il permet la comparaison directe entre les essais sur éléments de construction et les essais de fluage sur éprouvettes de traction. Il présente donc un certain intérêt pour la recherche.

L'étude plus en plus poussée du comportement des structures et de l'incidence d'ensemble des divers paramètres, a conduit au concept plus élaboré de la température critique en tant que critère de référence dépendant de toutes les influences entrant en ligne de compte.

Ainsi, il est maintenant reconnu que la température critique dépend largement de la distribution des températures dans l'élément considéré, aussi bien en section que sur la longueur. Selon que l'élément est également frappé par l'action thermique sur toutes ses faces ou qu'il se trouve partiellement protégé, par exemple au contact d'une dalle (collaborante ou non), on peut enregistrer une distribution différente de la température critique dans la section : elle est sensiblement uniforme dans le premier cas, mais peut présenter un gradient non négligeable dans le second.

De même, des différences de réchauffement sont souvent constatées entre les zones centrales des travées et celles proches des appuis. Elles ont une influence notable sur la température critique.

L'influence des distributions non uniformes des températures a fait l'objet d'études expérimentales et théoriques concordantes, qui permettent déjà d'en tenir compte dans le dimensionnement des éléments. Ces différences de réchauffement sont d'autant plus marquées que l'élévation de température est plus rapide. Donc, influence de la vitesse de réchauffement.

Un autre aspect majeur de la détermination de la température critique résulte du fait que, dans les structures réelles de bâtiments, les éléments frappés par l'action thermique d'un incendie ont toujours à supporter les charges statiques qui leur sont normalement imposées. Il y a donc simultanéité d'action des charges statiques et des actions thermiques.

Les charges statiques dont il s'agit dans ce cas ne sont pas les charges de calcul, mais celles qui correspondent à l'utilisation probable du bâtiment à un moment quelconque. Elles correspondent donc, sauf cas exceptionnels, à un niveau de contrainte inférieur à celui résultant de l'utilisation maximale admissible de la résistance à l'ambiante.

Il est donc important de reconnaître que la température critique d'un élément de construction varie avec le degré d'utilisation de sa capacité portante. Ce degré s'exprime par le rapport M/M_u ou F/F_u entre le moment ou l'effort supposé réel et le moment ou l'effort maximum correspondant à la capacité de résistance de l'élément à l'ambiante (20° C).

Enfin, il y a lieu de remarquer la similitude entre le comportement plastique des structures sous l'effet des charges statiques et leur comportement en présence des actions thermiques. Logiquement cette remarque vaut aussi pour l'intervention combinée de ces deux types d'actions.

En particulier, la ruine du système porteur se produit, dans un cas comme dans l'autre, de façon semblable par la formation de rotules plastiques et la constitution de mécanismes. Le fait que le réchauffement agisse sur les résistances plutôt que sur les sollicitations n'altère pas le caractère du processus.

La conséquence directe en est que, tout comme pour les charges statiques, l'hyperstaticité du système apporte un surplus considérable de résistance des structures vis-à-vis de l'incendie.

On constate, en conclusion, que la température critique qui caractérise la tenue d'un élément sous l'effet des actions thermiques dues à un incendie, dépend d'un grand nombre de facteurs dont aucun ne doit être négligé si l'on veut atteindre une approche correcte du problème.

2.3. Influence de la massivité des sections

Le processus de réchauffement des éléments exposés au flux thermique dépend évidemment de l'importance de celui-ci par rapport à la masse métallique à réchauffer.

Or, le flux thermique se présente, par convection et par radiation sur les faces exposées de l'élément, à l'exclusion donc de celles accolées à une dalle ou à tout autre élément massif protecteur.

De ce fait, on fait intervenir dans les calculs un facteur U/F , dit "de massivité" dans lequel U est le contour de la section transversale ou la partie de ce contour exposée au flux thermique et F est l'aire de la section transversale de l'élément étudié.

Ce facteur représente, en fait, l'inverse de la massivité, car plus la pièce est massive, plus ce facteur est faible.

L'incidence du flux thermique sur les faces de l'élément métallique et, par conséquent, le processus de réchauffement qui en résulte, diffèrent considérablement selon que cet élément est protégé ou non et, dans le premier cas, selon le type, la nature et l'épaisseur de la protection. Il est donc logique que la protection des éléments métalliques entre, elle aussi, pleinement en ligne de compte dans la détermination des effets d'un programme thermique donné.

Pour la commodité des applications pratiques, il est indiqué de considérer conjointement l'influence du rapport géométrique proprement dit U/F et celle de la protection. Les valeurs ainsi déterminées représentent donc, en fait, une massivité fictive puisqu'elles englobent les deux influences.

Ce procédé simplifié s'impose en vue de la confrontation des études théoriques avec les résultats d'essais, puisque ces deux influences sont indissociables expérimentalement.

En considérant séparément le cas des éléments métalliques dont une face se trouve en contact avec une dalle de béton -cas fréquent des planchers mixtes- ou celui des éléments partiellement enrobés, on peut déterminer les valeurs de ce facteur en tenant compte de ces influences additionnelles.

Enfin, on peut considérer à part le cas des éléments pour lesquels il y a lieu d'admettre comme température critique celle qui intervient dans une partie de la section, déterminante pour la tenue de l'élément, par exemple dans la semelle tendue d'une poutre en flexion.

Les valeurs ainsi déterminées du facteur de massivité (fictive) sont données, pour les profils laminés courants, sous forme de tableaux ou abaques pour l'usage pratique, permettant le dimensionnement de chaque élément par simple choix direct du profil.

2.4. Effets des dilatations thermiques

L'élévation de température d'un élément métallique provoque une dilatation de celui-ci, donc une variation de sa longueur. Cette variation reste sans effets sur la tenue au feu tant qu'il s'agit d'un élément isostatique, dont la dilatation est libre.

Cependant la dilatation des éléments constitutifs d'une structure est souvent entravée, et parfois même empêchée, par la résistance que les éléments adjacents opposent au déplacement des noeuds.

Il en est ainsi par exemple, des poteaux de bâtiments à étages ou des traverses de portiques multiples.

Les majorations de contraintes pouvant en résulter dans ces éléments doivent être déterminées avec discernement, compte tenu, d'une part, de la distribution des températures sur la longueur de l'élément considéré (effet modérateur d'une distribution non uniforme, par rapport à celui d'une distribution supposée uniforme de la température du point le plus sollicité thermiquement) et, d'autre part, de la résistance effective rencontrée par la dilatation ainsi déterminée, en fonction des caractéristiques du système de structure.

Dans la détermination de ces majorations de contraintes, il y a lieu de tenir compte du fait que dans toute pièce comprimée il y a une certaine interdépendance entre la flèche et la contrainte.

2.5. Effet de l'empêchement du déplacement des extrémités d'une poutre

La flèche que prend sous l'effet du réchauffement une poutre isostatique sur appuis libres, a pour conséquence un rapprochement de ses extrémités. Cela implique des précautions pour empêcher la chute de la poutre entre ses appuis.

Mais lorsque les extrémités d'une poutre sont tenues par des attaches qui empêchent leur déplacement axial, la flèche s'en trouve réduite par rapport à celle qui se produirait dans le cas des appuis libres. C'est le cas, par exemple, d'une travée intermédiaire d'un portique multiple, où la rigidité des files de poteaux situées de part et d'autre s'oppose à ce déplacement.

De cette entrave au déplacement des extrémités, il résulte un relèvement considérable de la température critique définie en fonction de la flèche.

Cet effet a été prouvé expérimentalement.

2.6. Effets du gradient de température

Un autre aspect essentiel est le gradient de température qui se produit entre une face exposée et une face non exposée au feu d'un élément. La dilatation différentielle qui en résulte donne lieu à une courbure qui se superpose à celle due au réchauffement général de la poutre.

C'est notamment le cas des solives et poutres de planchers mixtes, supportant une dalle de béton (associée ou non), ainsi que celui des poteaux exposés au feu sur un seul côté.

2.7. Effets des encastrements

Lorsqu'un élément isostatique sur deux appuis prend une flèche, ses extrémités subissent une rotation dans le plan de flexion. Cette rotation se produit alors librement.

Par contre, dans un élément faisant partie d'une structure et notamment d'un système hyperstatique, les attaches aux extrémités de la barre et les éléments adjacents s'opposent à la rotation des noeuds. Il s'agit alors d'une rotation entravée, dont on peut déterminer la valeur selon les méthodes classiques du calcul des structures.

Le cas limite de cet effet est celui d'un encastrement parfait (ou quasi parfait) dans un milieu rigide, tel qu'un massif en béton.

Le degré d'encastrement exerce une influence considérable sur le comportement de l'élément et donc sur sa résistance sous l'effet des actions thermiques.

L'effet d'encastrement joue pleinement dans le cas des travées intermédiaires des poutres continues.

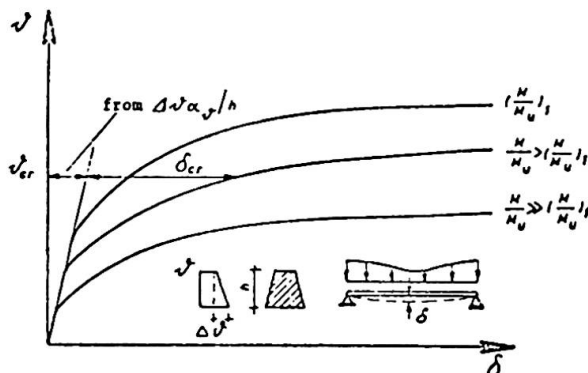
2.8. Effets du taux d'utilisation de la capacité portante

La température critique d'un élément porteur dépend fortement de son niveau de sollicitation statique en service, celui-ci étant caractérisé -suivant le mode de sollicitation- par le rapport entre la charge ou le moment du à ce niveau de sollicitation et la charge ou le moment ultime à l'ambiante (F/F_u ou M/M_u).

Des essais systématiques sur éléments de construction différemment chargés ont confirmé les résultats des considérations théoriques à ce sujet et ont permis d'établir avec une précision suffisante cette influence.

Le diagramme de la fig. 3, résultant de tels essais sur une poutre en flexion fait apparaître cette influence. On y remarque, au départ, une zone linéaire correspondant au gradient de température en section, suivie par une plastification progressive accompagnée de fluage à chaud.

Figure 3



Il est important de noter qu'il ne s'agit pas des sollicitations nominales de calcul, mais bien de celles qui avec une probabilité suffisante, seraient rencontrées à un moment quelconque en service normal.

Cette influence peut être exploitée dans certains cas dans lesquels il est possible d'atteindre une durée de résistance exigée, avec une protection réduite ou même sans protection, en choisissant un profil plus fort. Le critère de l'opportunité d'une telle mesure est alors de nature économique.

3. TENUE AU FEU DES ELEMENTS PORTEURS ET DES STRUCTURES EN ACIER

3.1. Éléments sollicités en traction

La ruine (critère de référence) d'un élément tendu se rapporte soit à une perte de résistance, soit à un allongement excessif entraînant des désordres inadmissibles dans le système porteur.

Les éléments tendus qui assurent la stabilité de la structure ou de sous-ensembles importants appellent une attention particulière et une marge de sécurité accrue.

C'est le cas notamment des barres de contreventement et de certains tirants dont la mise hors service compromettrait la stabilité du bâtiment.

3.2. Poutres sollicitées en flexion

Quoiqu'il soit toujours possible de recourir à un calcul en élasticité, pour lequel l'état limite serait en principe le même que dans le cas des charges statiques, le caractère du processus de ruine dû au réchauffement s'apparente, en fait, au comportement plastique sous charges statiques, avec la formation de rotules plastiques conduisant à la constitution d'un mécanisme. C'est donc par un calcul en plasticité qu'il convient de traiter le problème.

Schématiquement et en se fondant sur le concept d'analyse plastique du premier ordre, Vinnakota propose un diagramme très suggestif qui fait apparaître les niveaux des sollicitations limites selon le mode de calcul (figure 4).

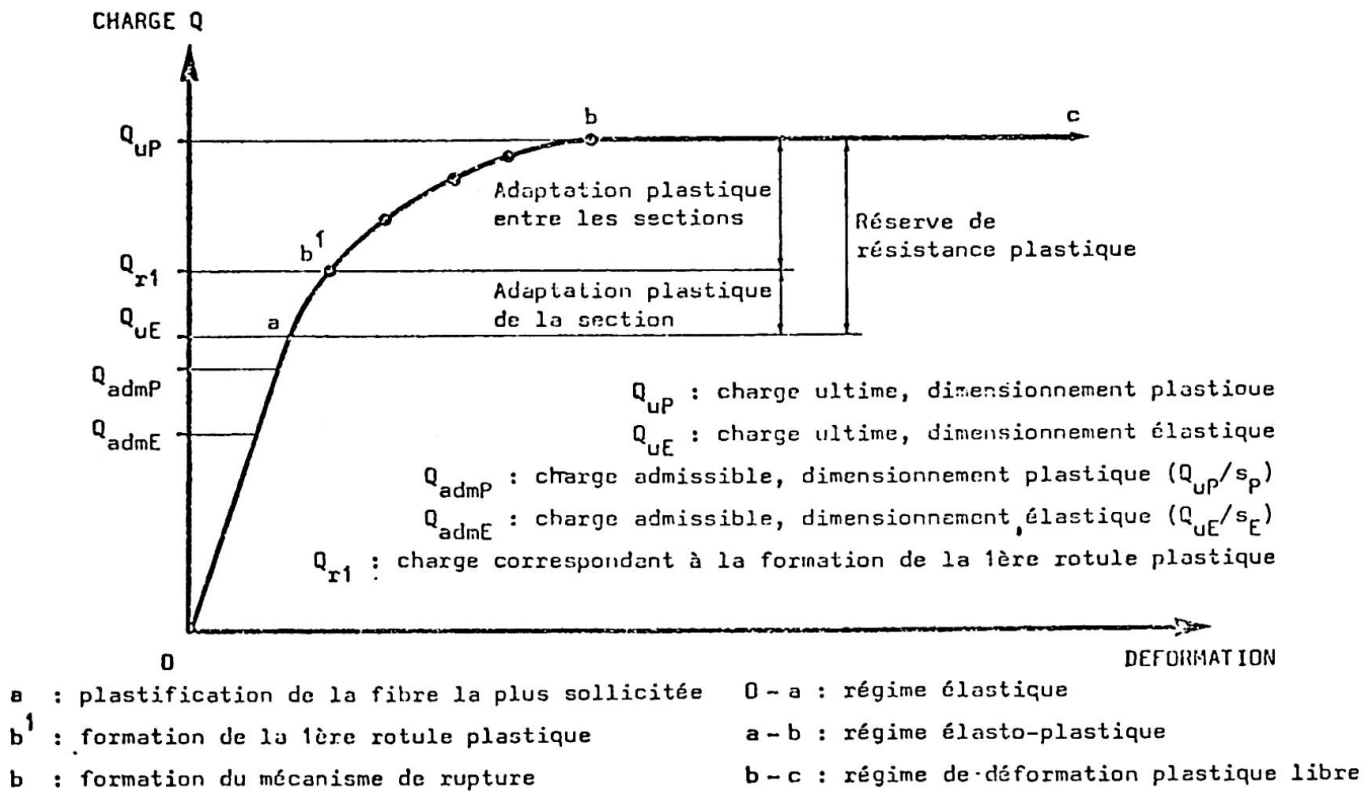
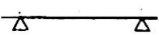
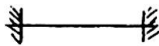


Figure 4

Courbe charge-déformation d'une structure métallique

Nous nous bornons ici à reproduire un tableau récapitulatif des résultats obtenus en Suède, par Thor, dans une étude systématique relative à une poutre sur deux appuis, car elle met très clairement en évidence les effets des facteurs mentionnés ci-après.

Distribution des températures	Système	Degré d'utilisation %	Température critique °C
a) uniforme, avec élévation lente		50	530
b) non uniforme dans la section, avec élévation rapide et avec un gradient donnant dans la semelle supérieure une température supérieure de 100° C à celle de la semelle inférieure		50	590
a)		25	620
b)		25	720
a) uniforme, avec élévation lente		50	560
b) non uniforme sur la longueur, avec élévation rapide et une température aux extrémités inférieure de 100° C à celle de la zone centrale		50	640
a)		25	640
b)		25	800

Résultats des recherches de Thor sur l'influence
- de l'hyperstaticité du système
- de la distribution des températures
- du degré d'utilisation de l'élément
sur le niveau de la température critique

Ainsi, par exemple, pour la poutre isostatique sur deux appuis, la température critique est celle qui correspond à la formation d'une rotule plastique, tandis qu'il faut trois rotules pour créer un mécanisme à un degré de liberté dans la poutre encastree aux deux extrémités.

D'une manière générale, on peut constater que de tous les types d'éléments porteurs, les poutres en flexion représentent la catégorie dont la tenue au feu a été le mieux explorée expérimentalement, permettant ainsi d'étayer les conclusions des études théoriques par des résultats d'essais suffisants.

Il a donc été possible d'établir, pour les types de profilés courants, des moyens pratiques et simples -sous forme d'abaques ou de tableaux- pour le choix des sections et la détermination des protections nécessaires en fonction des résistances exigées.

3.3. Éléments comprimés

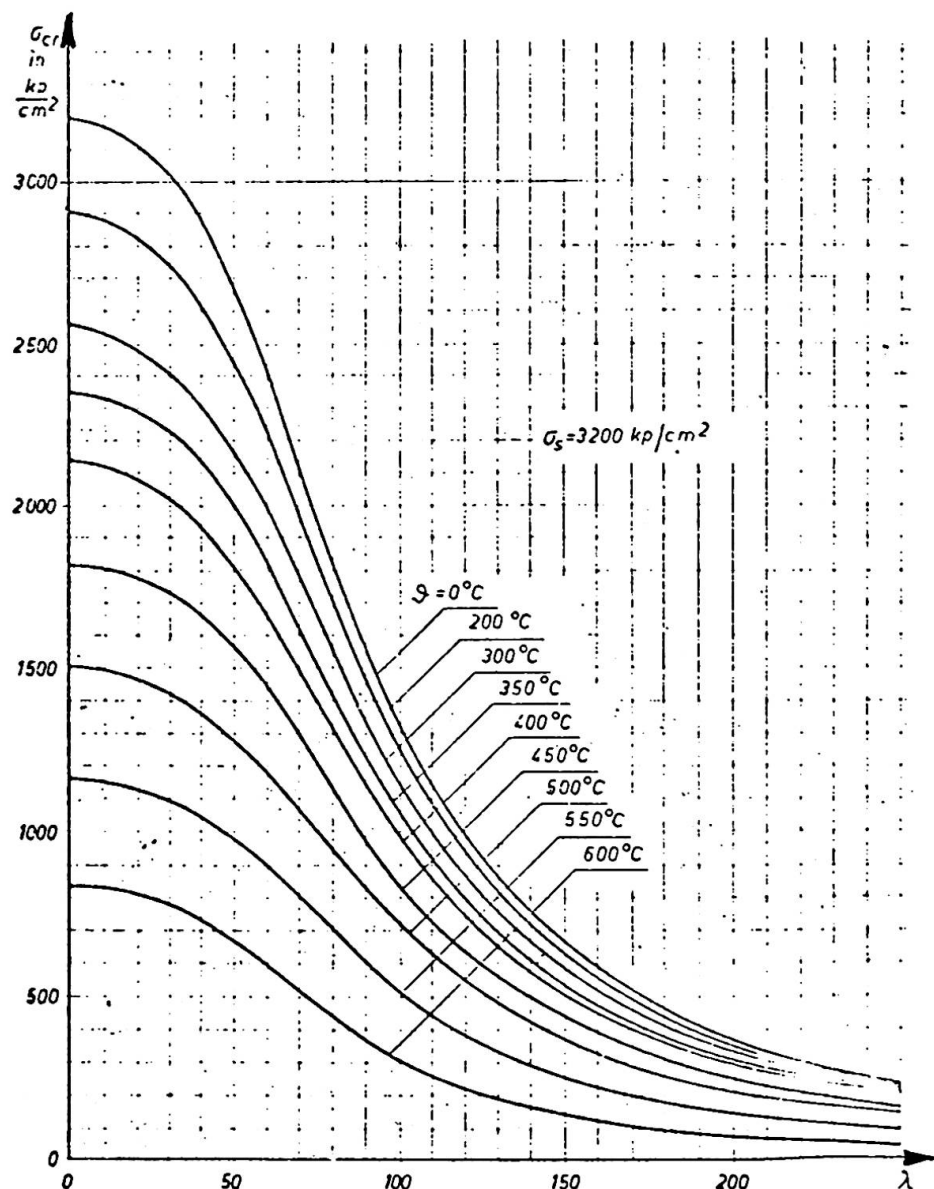
Le comportement des poteaux et autres éléments comprimés n'a pas encore fait l'objet d'une expérimentation aussi complète que celui des poutres fléchies. Toutefois, certaines séries d'essais récents ou en cours constituent déjà une base d'appréciation non négligeable.

L'action thermique influe de diverses manières sur la capacité portante d'un élément comprimé par :

- la réduction des caractéristiques mécaniques du métal
- l'excentricité supplémentaire pouvant résulter des déformations d'origine thermique
- les contraintes supplémentaires résultant d'un empêchement de dilatation
- l'effet de torsion dû à un réchauffement non uniforme ou les moments pouvant résulter d'un empêchement de cette torsion
- les phénomènes de fluage qui interviennent aux températures élevées.

En se fondant sur les théories générales du flambement et en y appliquant les influences des effets thermiques, il a été possible d'établir des courbes de flambement pour un élément bi-articulé, en compression axiale et en fonction de la température.

Figure 5



Des essais effectués sur poteaux protégés ou non, semblent révéler des résistances légèrement supérieures aux indications théoriques, mais il serait prématuré de conclure dans ce sens avant de disposer des résultats d'une expérimentation plus complète.

Ces essais se réfèrent à une température critique définie comme étant la température moyenne en section, à partir de laquelle l'élément métallique cesse de remplir sa fonction portante.

La pratique expérimentale courante dans la plupart des essais de poteaux effectués à ce jour a été fondée sur la prédiction des conditions de ruine à partir des températures mesurées de l'élément métallique (protégé ou non). Cette méthode semble conduire à des résultats acceptables, mais il n'en reste pas moins que l'essai en charge donnera toujours les indications les plus fiables.

3.4 Remarques générales relatives aux structures

Les aspects fondamentaux qui concourent à la tenue au feu des éléments structuraux en acier sont maintenant assez bien connus, grâce aux études théoriques et aux expériences qui en ont fourni les bases ou en ont confirmé les résultats.

De même, les processus de comportement des structures en acier et de leurs éléments constitutifs sous l'effet du feu apparaissent nettement en superposant les influences d'origine thermique à l'étude classique de la stabilité et de la résistance des systèmes porteurs.

Par ailleurs, l'étude expérimentale correspondante a pris récemment une tournure très réaliste et significative, par l'orientation vers des essais d'éléments et de sous-ensembles en grandeur nature, avec application simultanée de charges statiques et des programmes thermiques reflétant les conditions d'un incendie réel. La CECM a joué un rôle décisif dans ce sens, par la création d'une station d'essais conçue à cet effet.

Quelques lacunes subsistent cependant encore dans cet enchaînement, au niveau de la corrélation entre les résultats observés dans un four et ceux qu'un même programme thermique provoque dans l'enceinte d'un bâtiment, où les parois -suivant leur nature- ont des caractéristiques d'émissivité et de réflexion différentes.

Par ailleurs, il faut reconnaître que s'il reste encore beaucoup à faire dans le domaine de l'analyse du comportement de certaines catégories d'éléments porteurs, l'extrapolation des connaissances acquises à l'étude des structures n'en est qu'à ses tout premiers débuts.

Il n'est certes pas envisageable pratiquement de procéder à une large expérimentation dans ce domaine, mais la complexité de l'étude fondée sur les effets concurrents d'un grand nombre de paramètres exige au moins une confirmation ponctuelle des résultats par des essais sur des sous-ensembles en grandeur nature, corroborés avec des essais plus systématiques sur modèles réduits.

Ces perspectives de recherches qui s'ouvrent encore dans le domaine de la résistance au feu des structures en acier, doivent conduire à mieux définir les types de structures par rapport à leur tenue à l'incendie et à guider le choix des systèmes porteurs et des dispositions constructives en fonction de leur tenue au feu.

3.5. Structures non protégées

La détermination des conditions et des limites dans lesquelles la protection d'un élément ou d'une structure en acier n'est pas nécessaire, est une question de première importance, tant du point de vue de la sécurité que de l'économie et de la réalisation de l'ouvrage.

L'analyse de cette question s'inscrit dans l'étude d'ensemble de la sécurité à l'incendie, qui se situe en dehors du sujet précis du présent rapport. Notons toutefois que dans de nombreux cas il est inutile de protéger la structure en acier, soit parce qu'un incendie possible en fonction de la charge calorifique et des conditions géométriques et de ventilation du local ne pourrait pas donner lieu à une élévation de la température au-dessus du niveau critique dans les éléments affectés, soit en raison des exigences minimales de résistance au feu qui découlent de l'utilisation des locaux et de l'ensemble des mesures de protection existantes.

Prévoir alors néanmoins une protection serait une mesure anti-économique injustifiée.

Les critères généraux concernant la résistance des structures mentionnés en 2.1. à 2.8. et les indications données en 3.1. à 3.3. sont valables sans restriction dans le cas particulier des structures non protégées. Leur application permet donc d'établir des moyens pratiques de dimensionnement en fonction des mêmes paramètres que pour les éléments protégés.

Ainsi l'abaque de la figure 6, tracé à partir de considérations théoriques, fait apparaître l'influence de la température critique sur la résistance des poutres en acier non protégées, exprimée en minutes, par référence aux essais en four conduits sous le programme thermique normalisé.

Il est également important de ne pas confondre la résistance à un incendie naturel avec celle rapportée au programme thermique normalisé. Or, la durée de résistance d'un profil donné varie fortement en fonction de l'évolution et de l'intensité de l'incendie. Cela apparaît, par exemple, dans la figure 7 qui représente des courbes de résistance des éléments en acier non protégés, pour différents programmes thermiques. Ces courbes sont tracées sur la base de résultats d'essais, ce qui les rend incontestables.

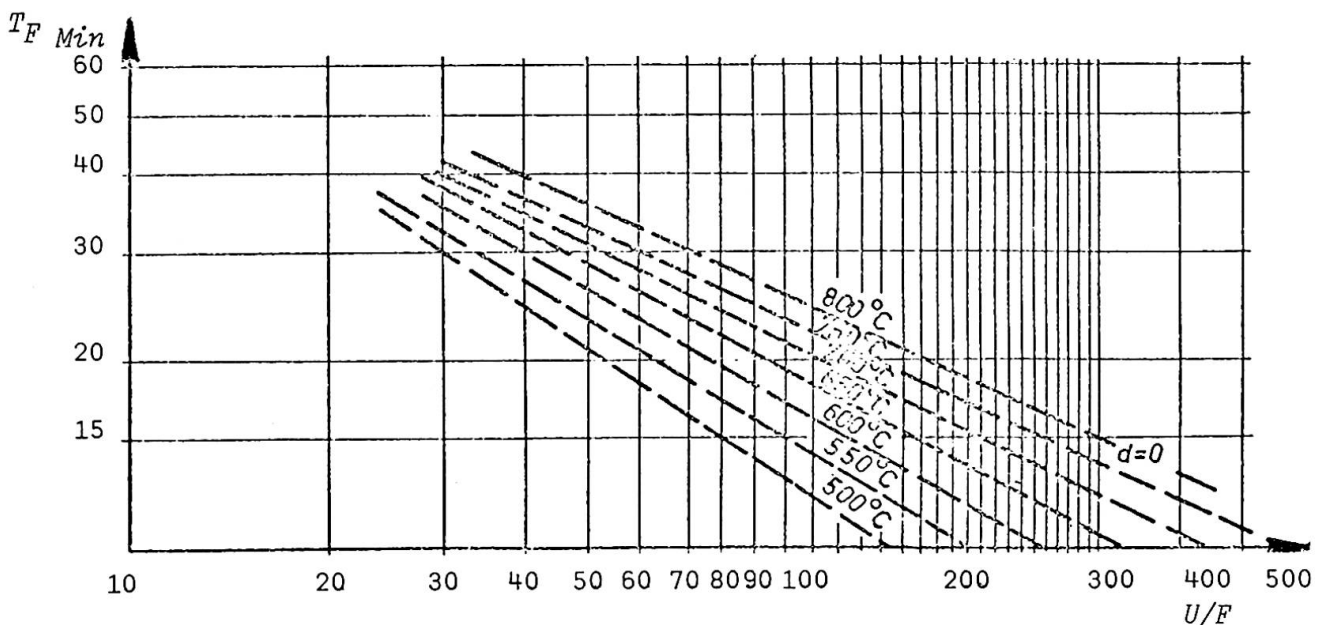
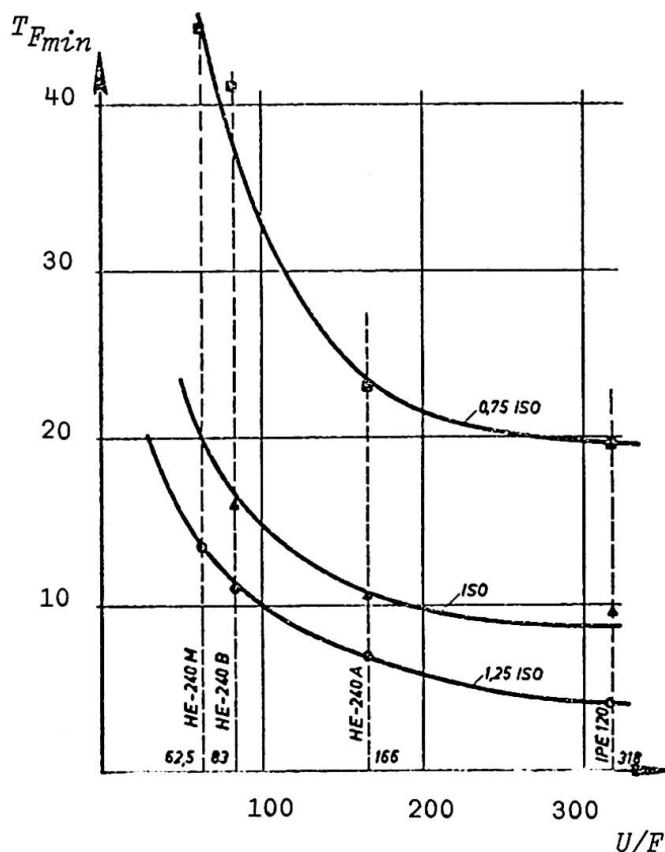


Figure 6

Figure 7



Dans le cas particulier des solives ou poutres de planchers supportant une dalle de béton, la protection partielle offerte par celle-ci n'est pas négligeable, car elle entraîne une augmentation sensible de la résistance au feu du profilé nu, exprimé par une diminution de la température critique.

Les aspects à considérer sont donc multiples. Leur étude permet de bien définir les limites d'acceptabilité de telles structures.

4 PROTECTION DES ELEMENTS PORTEURS ET DES STRUCTURES EN ACIER

4.1. Protection par couche isolante

Lorsque la résistance de la structure nue est insuffisante, elle doit être protégée. Le procédé classique consiste à interposer une couche protectrice, qui peut se présenter soit sous forme de plaques rigides disposées en caisson, soit d'une couche épousant le profil de l'élément à protéger.

L'effet protecteur de ces couches -en fonction de leur nature et de leur épaisseur- se détermine expérimentalement pour chaque type de profil. Cette méthode est sûre, car elle fait apparaître directement la résistance réelle au feu de l'élément protégé. Elle a toutefois l'inconvénient d'être globale et de ne pas permettre de dégager clairement les propriétés de la couche protectrice.

Cependant, le transfert de chaleur à travers cette couche a fait l'objet d'études qui ont conduit à des méthodes plus ou moins élaborées, tenant compte des propriétés thermiques du matériau isolant (conductivité et capacité thermique), du gradient de température dans la couche protectrice, de la transmission de chaleur par radiation, etc. Il est donc possible de connaître avec précision le pouvoir isolant d'une couche de protection.

Les peintures intumescentes récemment développées représentent une catégorie à part, dont l'intérêt est certain dans le domaine limité des résistances qu'elles peuvent couvrir.

4.2. Protection par transfert de chaleur

Des systèmes depuis longtemps préconisés, mais qui n'ont connu que des applications pratiques récentes et peu nombreuses, sont fondés sur le transfert de chaleur par irrigation de la structure.

Ce mode de protection est parfaitement sûr et peut être rigoureusement contrôlé par le calcul. Son développement est conditionné par les particularités structurales qu'il impose (usage exclusif d'éléments creux, dispositifs de circulation et équipements annexes) et surtout par ses incidences économiques.

5. STRUCTURES MIXTES ACIER-BETON

La tenue au feu des structures mixtes résulte, d'une part, du comportement spécifique de chacun des matériaux associés et, d'autre part, de l'incidence de la présence d'un matériau sur le comportement de l'autre.

Les aspects relatifs à l'acier sont brièvement exposés dans le présent rapport, ceux qui concernent le béton armé ou précontraint font l'objet d'un rapport distinct.

Les effets de protection partielle des poutres métalliques dans les planchers mixtes ont été également mentionnés dans le présent rapport.

Un domaine important est celui des poteaux mixtes : poteaux en acier enrobés de béton ou tubes en acier remplis de béton. Les essais effectués sur ces deux catégories d'éléments permettent déjà d'en évaluer la tenue au feu, mais des investigations plus poussées et des études théoriques plus complètes sont encore nécessaires.

La place importante occupée par les systèmes mixtes dans la construction justifierait un effort accru de recherche dans ce domaine.

6. SECURITE DES STRUCTURES EN CAS D'INCENDIE

L'incendie est un événement qui doit être classé dans la catégorie correspondant aux charges exceptionnelles.

La résistance des éléments de construction et des systèmes porteurs vis-à-vis des actions thermiques du feu doit par conséquent être traitée comme telle, par application des coefficients de sécurité affectés à cette catégorie des charges.

En ce qui concerne les combinaisons de charges à prendre en compte dans les calculs, il faut également appliquer les mêmes principes, en associant les sollicitations thermiques aux charges les plus probables en régime normal de service et en écartant la simultanéité d'intervention de charges exceptionnelles à leur niveau maximal.

C'est ainsi seulement qu'une sécurité effective et suffisante, mais raisonnable, peut être garantie.

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RESUME

Principes et aspects fondamentaux de la tenue au feu des éléments et structures en acier.

Effets des principaux paramètres.

Brèves remarques sur les structures mixtes et la sécurité au feu des structures.

ZUSAMMENFASSUNG

Grundsätze und grundlegende Aspekte des Brandverhaltens der Bauteile und Tragwerke aus Stahl.

Einwirkung der wichtigsten Parameter.

Kurze Bemerkungen über Verbundkonstruktionen sowie über die Sicherheit der Tragwerke im Brandfall.

SUMMARY

Principles and basic aspects of the behavior of steel members and structures in fire.

Effects of the main parameters.

Brief remarks on composite structures and on the fire safety of structures.

IIIc

Design of reinforced and prestressed concrete Structures for Fire Resistance

Calcul et conception des structures en béton armé ou précontraint en vue de leur résistance à l'incendie

Bemessung von Stahlbeton- und Spannbetonbauwerken gegen Brandeinwirkungen

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How to design prestressed Concrete for a specific Fire Endurance

INTRODUCTION

Building elements such as beams, slabs, columns, and walls, react to fire in accordance with certain laws of nature. By the use of accepted engineering principles, it is possible to predict through calculations the behavior of prestressed concrete elements which are exposed to fire. The calculation procedures involve the use of physical properties of concrete and steel at high temperatures together with the temperature history and temperature distribution within the element.

STRUCTURAL BEHAVIOR

Simple Support: To design a simply supported slab of prestressed concrete for a specific fire endurance, one must know the span length, the loading, the types of steel and concrete, the slab thickness, and the position of the prestressing steel.^{(1)**} Because fire endurance of a slab is determined experimentally by exposing the underside of the slab to a standard fire,⁽²⁾ it should be assumed for design purposes that such conditions exist. Under such conditions, the temperature distribution within the slab can be calculated or otherwise determined (e.g., from published fire test data⁽³⁾) for the fire endurance time. For a given location of the reinforcement, the steel temperature is thus known. The strength of the steel can be determined for that temperature from experimental data relating temperature and strength.^(4,5) Similarly, the strength of the concrete in the compressive zone can be determined.⁽⁶⁾ By using the strengths of the steel and concrete thus determined, calculations for ultimate moment capacity can then be made utilizing formulas similar to those for normal temperature conditions.⁽¹⁾ By equating the applied moment to the ultimate moment capacity, the required amount of prestressing steel can be calculated. If the amount of steel results in an over-reinforced section, the design must be modified, e.g., by making the slab thicker.

Continuity: For continuous slabs or beams subjected to fire, thermal deformations occur that cause a redistribution of applied bending moments, i.e., the moments over the supports increase until the top reinforcing steel yields and the moments near midspan decrease.^(7,8) The top reinforcement must be sufficiently ductile and long enough to permit moment redistribution without causing a shear failure.

Restraint to Thermal Expansion: When a fire occurs beneath the floor slab of an interior bay of a multi-bay building, the heated portion tends to expand and exert forces on the surrounding slab. The surrounding slab, in turn, pushes against the heated portion and in effect, externally pre-stresses the slab. When the fire starts, the line of action of the force (called restraint force or thermal thrust) acts near the bottom of the slab.⁽⁹⁾ The effect of this force on the capacity of the slab is similar to that of "fictitious reinforcement" located at the line of thrust.⁽¹⁰⁾ In general, such restraint to thermal expansion greatly increases the slab's fire endurance.⁽¹¹⁾

Fig. 1 shows diagrammatically the effect of thermal restraint on the behavior of a uniformly loaded member. The thrust, T , is treated as fictitious reinforcement.

CALCULATION PROCEDURES

Procedures for calculating fire endurance of simply supported and continuous beams and slabs follow accepted engineering principles and have been explained elsewhere.^(1,12) A search of the literature shows a lack of examples of calculations dealing with restraint of thermal expansion of beams and slabs exposed to fire. Such an example is included in this paper, following a discussion of some basic concepts.

Selvaggio and Carlson⁽¹³⁾ showed that the magnitude of the thermal thrust force, T , for a given expansion, Δl , varies with the "heated perimeter," s , and the modulus of elasticity of the concrete:

$$\frac{T_1}{T_0} = \frac{s_1 E_1}{s_0 E_0} \quad \dots(1)$$

where T = thrust in kips

s = heated perimeter, i.e., the portion of the perimeter of the cross section, normal to the direction of thrust, that is exposed to fire, in inches

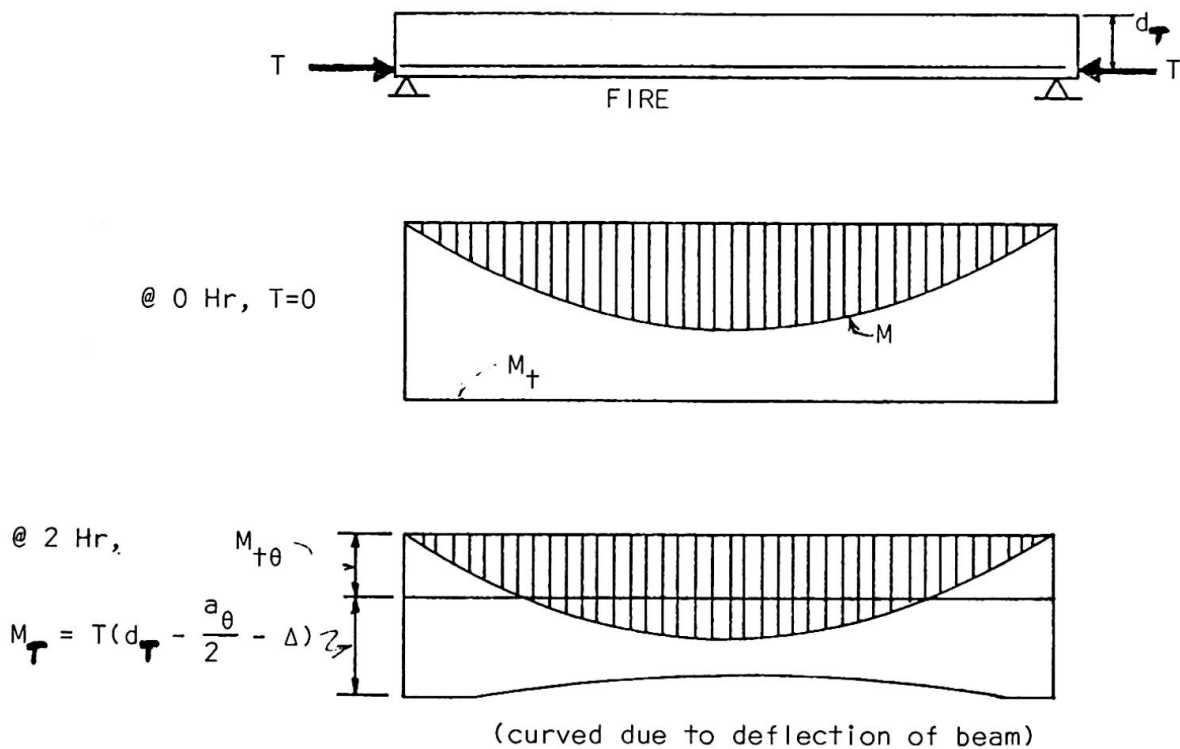


Fig. 1 -- Moment diagrams for axially restrained beam during fire exposure. Note that at 2 hr $M_{T\theta}$ is less than M and effects axial restraint permit beam to continue to support load.

E = concrete modulus of elasticity in ksi and subscripts 0 and 1 refer to the reference specimens, i.e., those described in references 9 and 13, and the member in question, respectively. Issen et al.⁽⁹⁾ refined Eq. 1 by introducing a term $z = A/s$, so that

$$\frac{T_1}{A_1 E_1} = \frac{T_0}{A_0 E_0} \left(\frac{z_0}{z_1} \right) \quad \dots (2)$$

To facilitate calculation of T_1 , Issen et al.⁽⁹⁾ introduced two dimensionless terms, T/AE , the thrust parameter, and $\Delta l/l$, the strain parameter, in which l is the heated length in inches. Reference 9 also gives nomograms, shown in Fig. 2, relating the thrust parameter, the strain parameter and z . Thus for a given member it is possible to estimate T_1 for any particular value of Δl , without use of the specific values of T_0 , E_0 , or A_0 . Note that the nomograms can be used for reinforced as well as pre-stressed concrete of either normal weight or lightweight concrete.

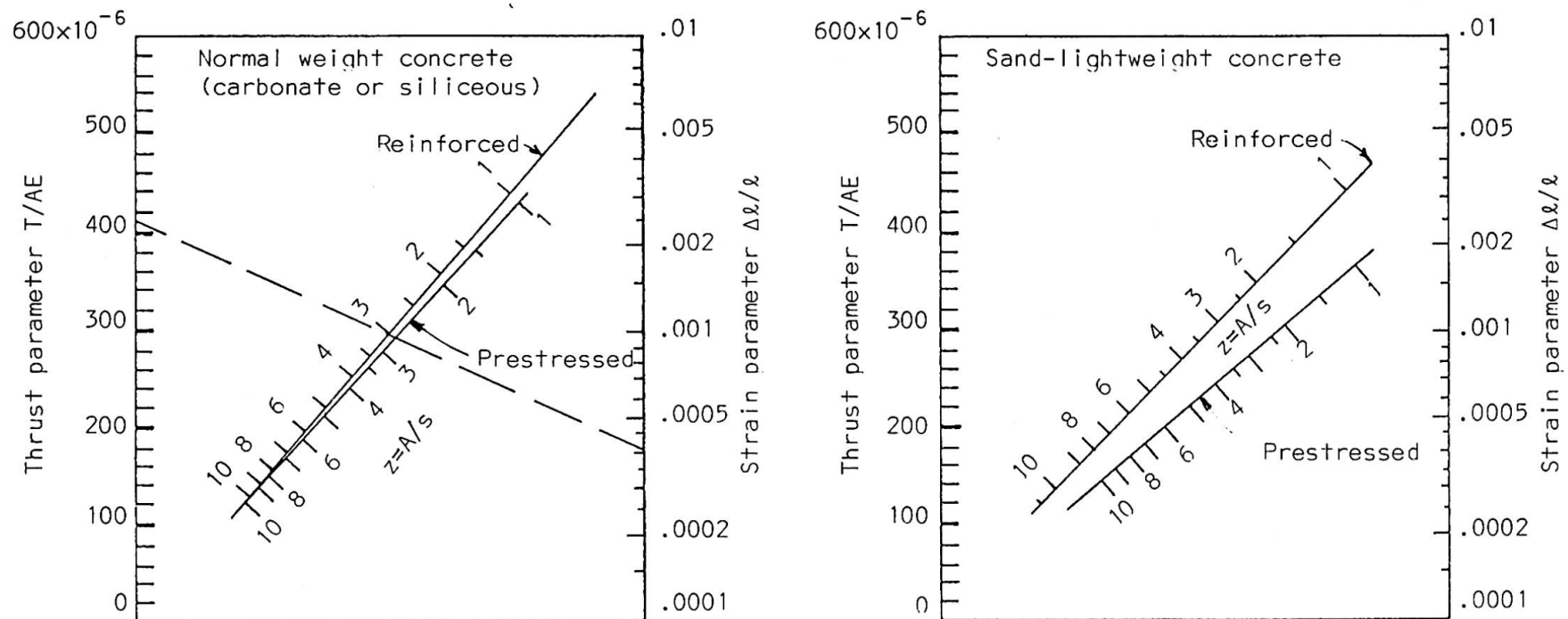


Fig. 2 Nomograms relating thrust parameter, strain parameter, and ratio of cross-sectional area to heated perimeter.⁽⁹⁾

In design problems involving restraint forces, it is necessary to estimate the deflection of the fire-exposed member. This can be done by making use of data developed during fire tests of the reference specimens. Fig. 3 closely represents the deflection of the reference specimens in which minimal restraint occurred.

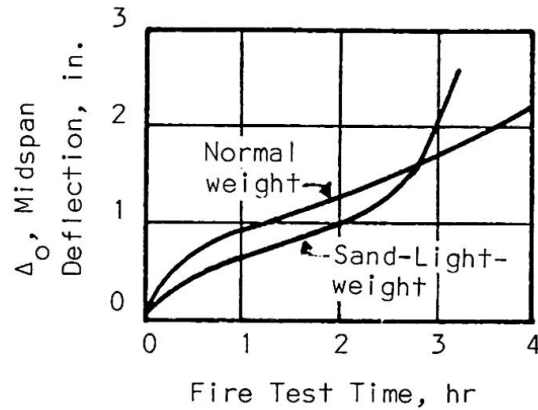


Fig. 3 -- Midspan deflection, Δ_0 , of reference specimens

The deflection of members can be estimated:

$$\Delta_1 = \frac{l_1^2 \Delta_0}{3500 y_{b1}} \quad \dots(3)$$

in which Δ_0 is obtained from Fig. 3 in in.

l = heated length of member in in.

y_b = distance between the centroidal axis of the member and the extreme bottom fiber in in.

A procedure for estimating the thrust requirements for a given fire endurance for simply supported slabs or beams follows:

1. Determine the retained moment capacity, $M_{t\theta}$, for the required fire endurance. The units for $M_{t\theta}$ are in.-kips.
2. If the applied moment, M , is greater than $M_{t\theta}$, estimate the deflection, Δ_1 , assuming that minimal restraint occurs. Use Fig. 3 and Eq. 3. (If $M_{t\theta} > M$, no thrust is needed.)
3. Estimate the location of the thrust line at the supports. For minimal restraint, the thrust line is near the bottom of the

member at the support and can be assumed to be $0.1 h$ above the support where h is the overall depth of member.

4. Calculate the magnitude of the required thrust T_1 using the formula:

$$T_1 = (M - M_{t\theta}) / (d_T - \frac{a_\theta}{2} - \Delta_1) \quad \dots(4)$$

where d_T is the distance between the thrust line and the top of the member at the supports.

5. Calculate the thrust parameter, T_1/A_1E_1 and $z_1 = A_1/s_1$.
6. Enter Fig. 2 with T_1/A_1E_1 and z_1 and determine the strain parameter, $\Delta\ell/\ell$.
7. Calculate $\Delta\ell$ by multiplying the strain parameter by the heated length ℓ .
8. Determine if the surrounding structure can withstand the thrust, T_1 , with a displacement no greater than $\Delta\ell$. If the structure cannot withstand T_1 with a displacement no greater than $\Delta\ell$, either $M_{t\theta}$ must be increased or the surrounding structure must be made stiffer.

Example Problem: A parking structure consists of multi-story reinforced concrete columns, L-shaped spandrel beams, and 8-ft wide, 57-ft span double tees with a cast-in-place topping. In much of the structure continuity can be achieved with reinforcement in the topping, but ramp areas consist of a single span. Determine if adequate restraint can be achieved in ramp areas to achieve a 2-hr fire endurance.

For double-tee floor (4-ft wide section):

$$A_{ps} = 7(0.153) = 1.071 \text{ in.}^2$$

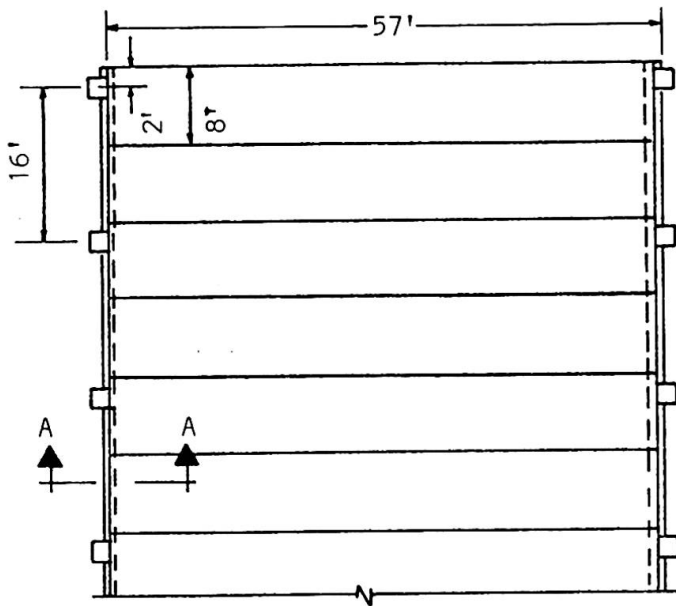
$$A_c = 5(48) + 22(7 + 5)/2 = 372 \text{ in.}^2$$

$$w_d = 372(115)/144 = 297 \text{ lb/ft}$$

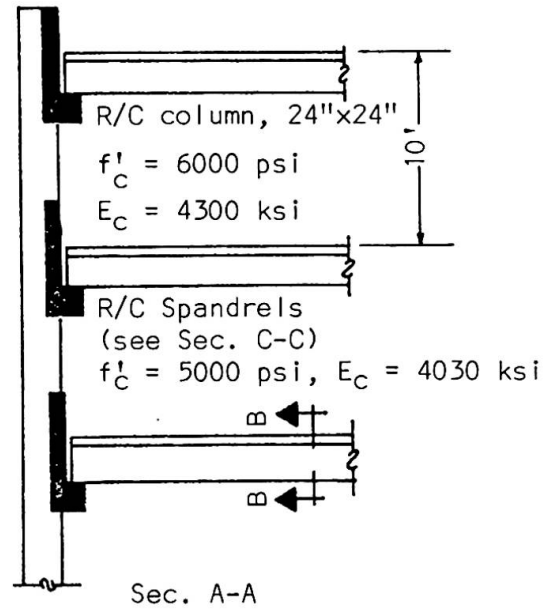
$$w_\ell = 75 \text{ psf} = 300 \text{ lb/ft}$$

$$M = (0.297 + 0.300)(57)^2/8 = 242.4 \text{ ft-k} = 2909 \text{ in.-k}$$

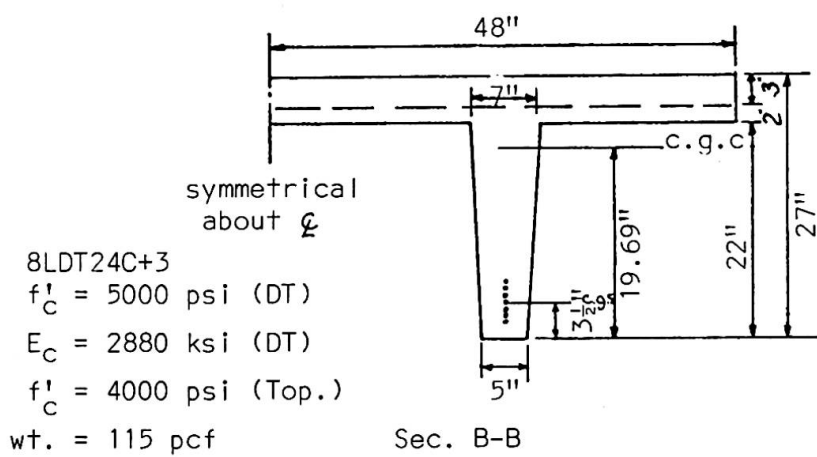
$$d = 27 - 3.5 = 23.5 \text{ in.}$$



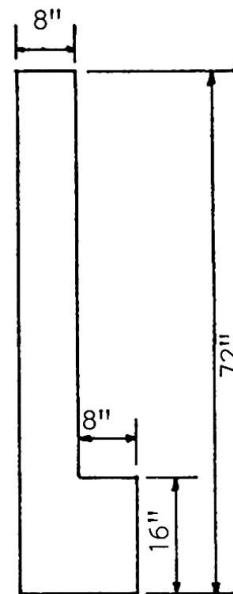
Plan of Ramp



Sec. A-A



Sec. B-B



Sec. C-C

Solution:

(1) Estimate $M_{t\theta}$ for double tee at 2 hr

(1.a) Determine temperature of prestressing steel, θ_s , from Fig. 4.

To enter Fig. 4, the beam width at the steel centroid, b_o , and the distance between the steel centroid and the bottom of the beam, u , must be determined. [Note that Fig. 4 applies only to the temperatures along the vertical centerline of beams, thus if the steel is located away from the centerline, the beam isotherms would have to be plotted to determine the steel temperature more accurately.]

$$u = 3.5 \text{ in.}$$

$$b_o = 5.0 + \frac{3.5}{22} (7 - 5) = 5.3 \text{ in.}$$

From Fig. 4, $\theta_s = 895^\circ\text{F}$

(1.b) The strength of prestressing steel at 895°F is 35.5% [From Fig. 5] of its strength at normal temperatures. Thus

$$f_{pu\theta} = 0.355 f_{pu} = 0.355(270) = 95.8 \text{ ksi}$$

(1.c) The stress in the steel at ultimate is

$$f_{ps\theta} = f_{pu\theta} \left(1 - \frac{0.5 A_{ps} f_{pu\theta}}{b d f'_c} \right) = 94.7 \text{ ksi}$$

$$\text{and } a_\theta = \frac{A_{ps} f_{ps\theta}}{0.85 f'_c b} = 0.62 \text{ in.}$$

$$(1.d) \quad M_{t\theta} = A_{ps} f_{ps\theta} \left(d - \frac{a_\theta}{2} \right) = 2352 \text{ in.-k}$$

$M_{t\theta} < M$, so added capacity is needed.

(2) Estimate midspan deflection of double tee assuming minimal restraint at 2 hr for sand-lightweight concrete.

From Fig. 3, $\Delta_o = 1.0 \text{ in.}$

$$\text{From Eq. 3, } \Delta_1 = \frac{l_1^2 \Delta_o}{3500 y_{b1}}$$

$$\Delta_1 = \frac{(57 \times 12)^2 (1.0)}{3500 (19.69)} = 6.79 \text{ in.}$$

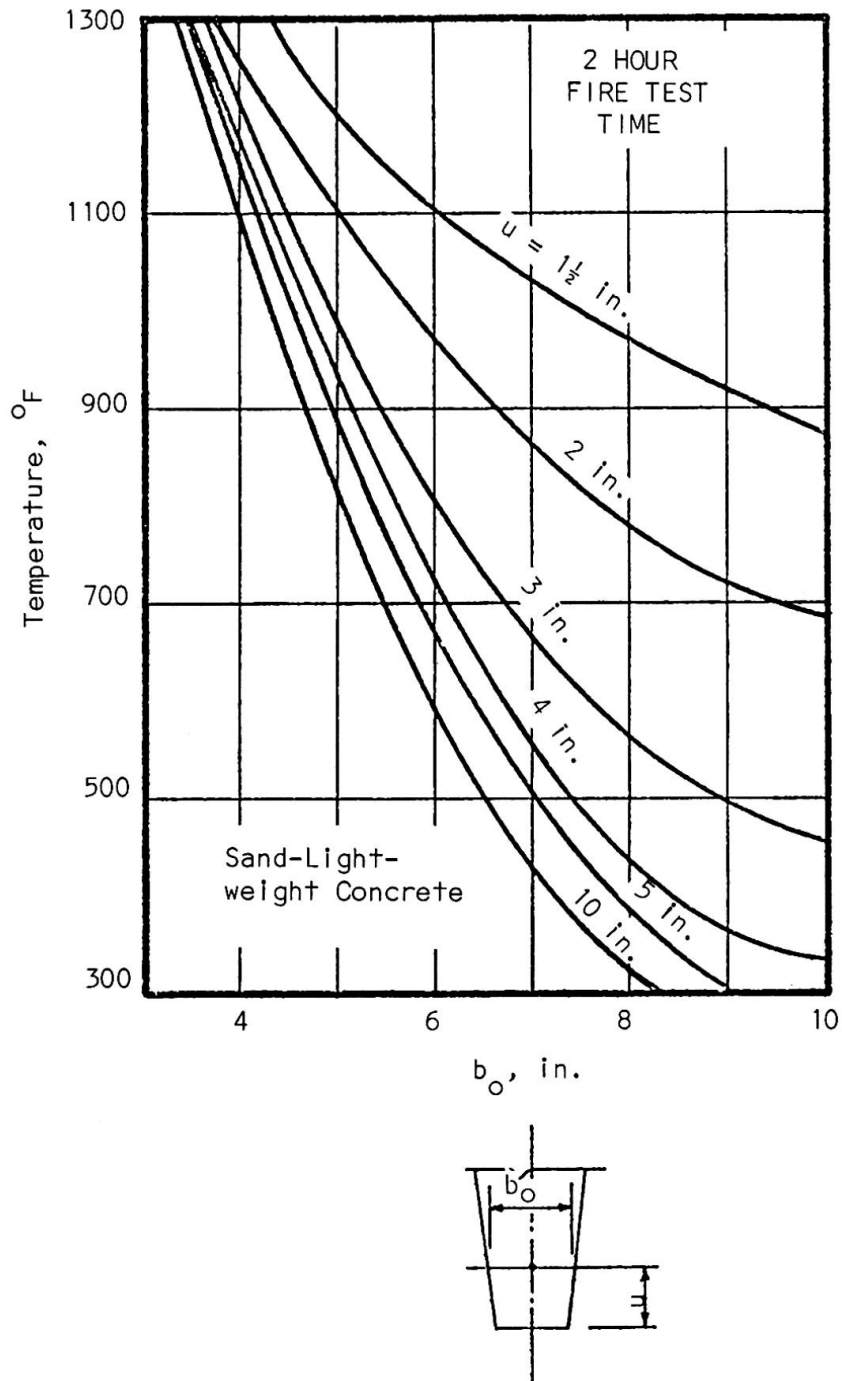


Fig. 4 -- Temperatures at 2 hours along the vertical centerline of beams made of sand-lightweight concrete.

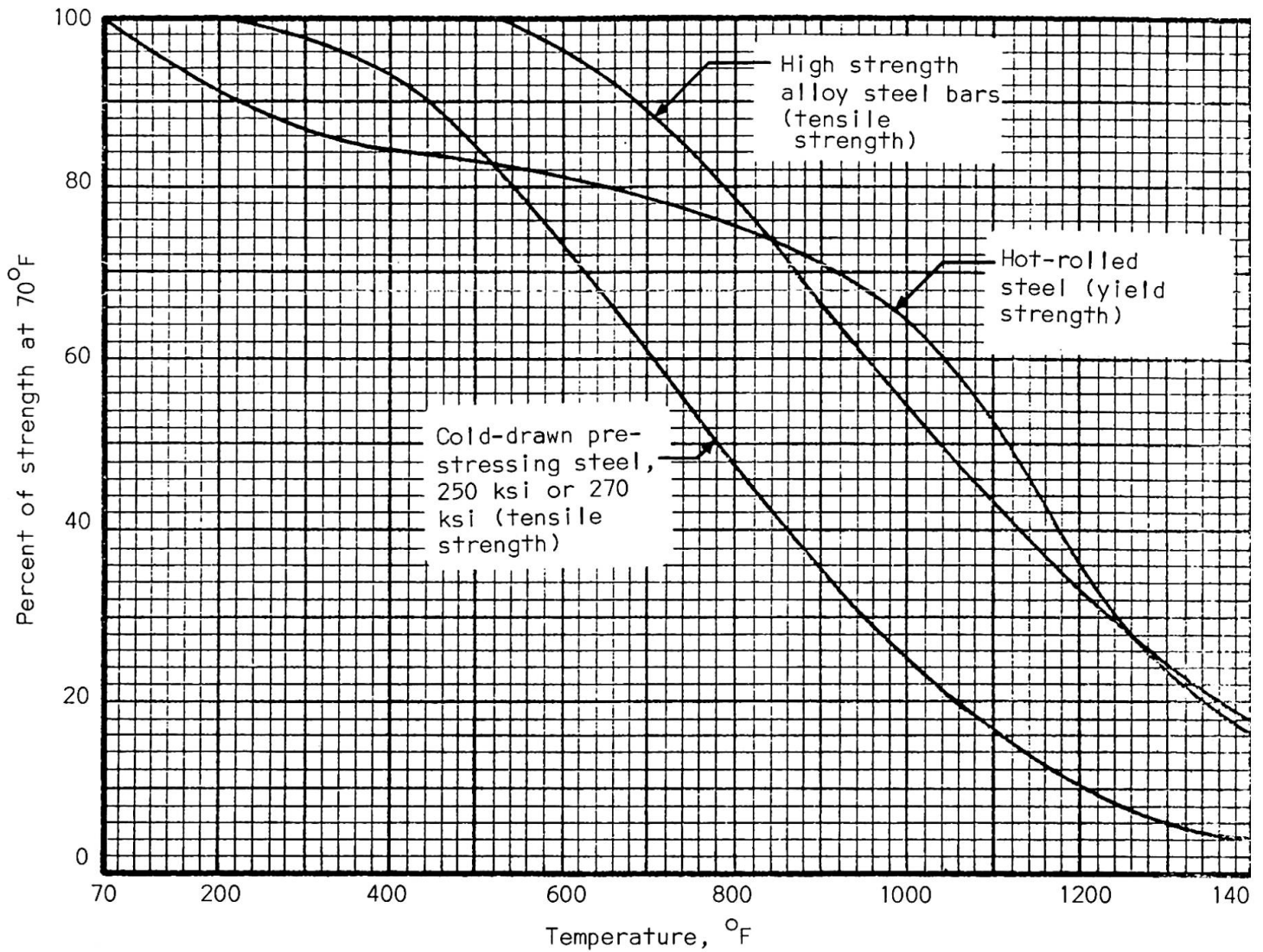


Fig. 5 -- Temperature-strength relationships for hot-rolled and cold-drawn steels.

- (3) Determine the position of the thrust line at the supports.

Assume that the thrust line is 0.1 h above support

$$d_T = 27 - 0.1(27) = 24.3 \text{ in.}$$

- (4) Calculate required thrust T_1

$$T_1 = \frac{(M - M_{+0})}{(d_T - 0.5a_\theta - \Delta_1)}$$

$$a_\theta = \frac{2909}{2352}(0.62) = 0.77 \text{ in.}$$

$$T_1 = \frac{(2909 - 2352)}{(24.3 - 0.39 - 6.79)} = 32.5 \text{ k/beam}$$

- (5) Calculate the thrust parameter T_1/A_1E_1 and $z_1 = A_1/s_1$

$$T_1/A_1E_1 = 32.5/372(2880) = 30 \times 10^{-6}$$

$$z_1 = 372/[48 + 2(22)] = 4.0 \text{ in.}$$

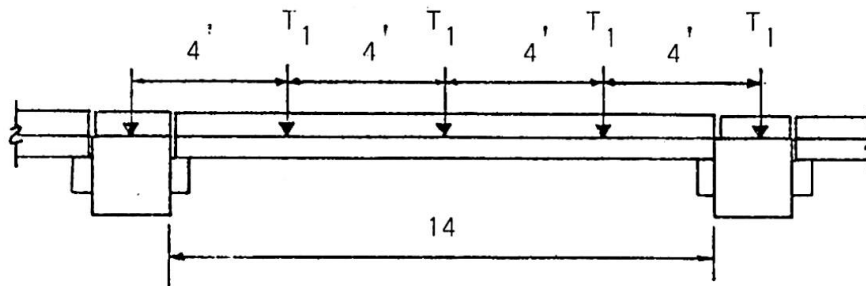
- (6) Determine the strain parameter, $\Delta l/l$, from Fig. 2:

$$\Delta l/l = 0.0026$$

- (7) Determine Δl :

$$\Delta l = 0.0026(57 \times 12) = 1.78 \text{ in.}$$

- (8) Determine if spandrel can withstand thrust from the double tees:



PLAN OF SPANDREL

- (8.a) M at spandrel midspan: (assume spandrel span = 13 ft 8 in. = 164 in. center to center of supports)

$$M = 1.5T_1(82) - T_1(48) = 75T_1 = 75(32.5) = 2438 \text{ in.-k}$$

Spandrel must withstand $M = 2438 \text{ in.-k}$ with a load factor of 1.4 because the thrust occurs early and remains essentially constant. Determine the amount of reinforcement needed. Assume $d = 14 \text{ in.}$, $a = 4 \text{ in.}$, and $f_y = 60 \text{ ksi}$:

$$A_s = \frac{1.4M}{f_y(d - a/2)} = \frac{1.4(2438)}{60(14 - 2)} = 4.74 \text{ in.}^2$$

Check reinforcement index, ω :

$$\omega = \frac{A_s f_y}{bd f'_c} = \frac{4.74(60)}{16(14)(5)} = 0.25 < 0.30 \text{ O.K.}$$

Check a :

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.74(60)}{0.85(5)(16)} = 4.17 \text{ in.} \approx 4 \text{ in. O.K.}$$

- (8.b) Estimate lateral deflection of spandrel due to thrust. Assume cracked section, $I_{cr} = 3300 \text{ in.}^4$, $E_c = 4030 \text{ ksi}$.

$$\Delta = \frac{T_1}{48EI} [(164)^3 + 6(34)(164)^2 - 4(34)^3] = 0.50 \text{ in.}$$

(8.c) Estimate column deflection assuming fire beneath top level.

For column, $I = 27,600 \text{ in.}^4$, $E_c = 4300 \text{ ksi}$.

For top level,

$$\Delta = \frac{Ph^3}{3EI} = \frac{4T_1 h^3}{3EI}$$

where h is the unsupported story height =

$$10 \text{ ft} - (27 \text{ in.}) = 120 - 27 = 93 \text{ in.}$$

$$\Delta = \frac{4(32.5)(93)^3}{3(4300)(27,600)} = 0.29 \text{ in.}$$

(8.d) For top level, displacement at midspan of spandrel =

$$2(0.50 + 0.29) = 1.58 \text{ in.} < 1.78 \text{ in.}$$

Thus adequate restraint will occur to achieve 2-hr fire endurance provided that the space (total) between the ends of the double tees and the vertical face of the spandrel beams is less than $1.78 - 1.58 = 0.20 \text{ in.}$

NOTATION

- a = depth of equivalent rectangular stress block at ultimate load, and is equal to $A_{ps} f_{ps} / 0.85 f'_c b$ or $A_s f_y / 0.85 f'_c b$ (in.)
- A, A_c = cross sectional area of a member subjected to thrust (in.²)
- A_s = area of reinforcing steel (in.²)
- A_{ps} = area of prestressing steel (in.²)
- b = width of compression zone (for use in flexural calculations) (in.)
- b_o = width of a beam or joist at centroid of reinforcement (for use in estimating temperature during fire exposure) (in.)
- d = distance between centroid of reinforcement and extreme compression fiber (in.)
- d_T = distance between line of action of thrust at the supports and extreme compression fiber (in.)
- E, E_c = modulus of elasticity of concrete (ksi)
- f'_c = compressive strength of concrete (ksi)
- f_{ps} = stress in prestressing steel in flexural member at ultimate load (ksi)
- f_{pu} = ultimate strength of prestressing steel (ksi)
- f_y = yield strength of hot-rolled steel (ksi)
- h = overall depth of flexural member (in.)

- h = unbraced height of column (in.)
 I = moment of inertia of cross section (in.⁴)
 I_{cr} = moment of inertia of cracked cross section of flexural member (in.⁴)
 l = heated length of a flexural member (in.)
 Δl = increase in length due to thermal expansion (in.)
 M = service load bending moment (in.-k)
 M_t = theoretical moment strength (in.-k)
 M_T = ultimate moment due to thrust resulting from restraint of thermal expansion (in.-k)
 s = heated perimeter of a member, i.e., that portion of the perimeter of a section of a member, normal to the direction of the thermal thrust, which is exposed to fire (in.)
 T = thermal thrust (k)
 u = distance from bottom of slab or beam to a point within the member, e.g., the distance from the underside of a slab to the center of a prestressing strand (in.)
 w = uniformly distributed load on a flexural member, in general $w = w_d + w_l$ in which the subscripts d and l indicate dead and live loads (k/in. or k/ft)
 y_b = distance between centroidal axis of flexural member to extreme bottom fiber (in.)
 $z = A/s$ (in.)
 Δ = deflection (in.)
 Δl = increase in length due to thermal expansion (in.)
 θ = temperature (°F)
 θ_s = temperature of steel (°F)
 $\omega = A_s f_y / b d f'_c$

Subscripts

- c = of concrete
 p = of prestressing steel
 s = of reinforcing steel
 $o, 1$ = of reference specimens and member in question
 θ = as affected by temperature
 b = with reference to the bottom fiber

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SUMMARY

Building elements such as beams, slabs, columns, and walls, react to fire in accordance with certain laws of nature. By the use of accepted engineering principles, it is possible to predict through calculations the behavior of prestressed concrete elements which are exposed to fire. The calculation procedures involve the use of physical properties of concrete and steel at high temperatures together with the temperature history and temperature distribution within the element.

RESUME

Les éléments de bâtiments tels que poutres, dalles, colonnes et parois, réagissent au feu selon certaines lois naturelles. Par application de principes de calcul reconnus, il est possible de prédire- et de chiffrer- le comportement d'éléments en béton précontraints, exposés au feu. Les procédés de calcul font appel aux propriétés physiques à haute température des aciers et bétons, ainsi qu'à l'évolution de la température dans le temps et dans les éléments.

ZUSAMMENFASSUNG

Bauelemente, wie Balken, Decken, Stützen und Wände reagieren auf Feuer entsprechend bestimmten Naturgesetzen. Unter Verwendung angenommener Prinzipien ist es möglich mittels Berechnungen das Verhalten vorgespannter, dem Feuer ausgesetzter Stahlbetonelemente vorauszusagen. Die Berechnungsverfahren enthalten die Anwendung physikalischer Eigenschaften von Beton und Stahl bei hohen Temperaturen sowie die Temperaturentwicklung und -verteilung innerhalb des Elementes.

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IV

**Constructions spéciales
(acier, béton, mixtes; études comparatives)**

**Spezielle Bauwerke
(Stahl, Beton, Verbund; vergleichende Studien)**

**Special Structures
(Steel, Concrete, Composite; comparative Studies)**

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IVa

Offshore Structures

Constructions en mer

Bauwerke im Meer

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INTRODUCTION

With an ever-expanding world population the demands for space, food and energy are increasing annually. This development has caused world-wide concern since the world's land area only covers about three tenths of the earth. Therefore, with the on-shore resources gradually being depleted, growing attention has been focused on the part of the world covered by the oceans. Although studies have been carried out regarding the use of the sea for habitation, so far the main efforts have been directed to exploring the mineral resources beneath the sea. In man's quest to explore these riches, the Continental shelf - the portion of the sea floor less than 200 m below sea level - has become the initial proving ground of the structural engineer, thereby moving gradually from shallow to deeper waters. However, the development of these regions has been rather slow because of the high cost of exploration and production, and the complexity of the associated engineering problems. By early 1974, of the 27,876,000 square kilometers of ocean with depths of 30 m or less, about 60% had shown sedimentary basins potentially holding oil and gas deposits. Of this portion only 25% had been leased for exploration and actually only 15% of that has been explored.

During the sixties the offshore industry has been rapidly expanding to meet world energy demands. In 1960 there were only three of four countries and about five companies with offshore petroleum interests. Fourteen years later several hundred companies are exploring the Continental shelves of 80 countries and already 30 nations are producing, or about to produce, subsea oil and gas. Over 10,000 wells have been drilled offshore and oil and gas are being piped from as far as 400 kilometers offshore and in water depths of up to 150 m. Figure 1 shows the potential offshore oil areas around the world and the locations with current production. With the 1973 increases in the posted price of oil, the economic feasibility of offshore oil and gas exploration and production has improved drastically, unfortunately thereby contributing to an almost rampant world-wide inflation. As a result the expected production rates for the next decade will increase exponentially, as shown in Table I.

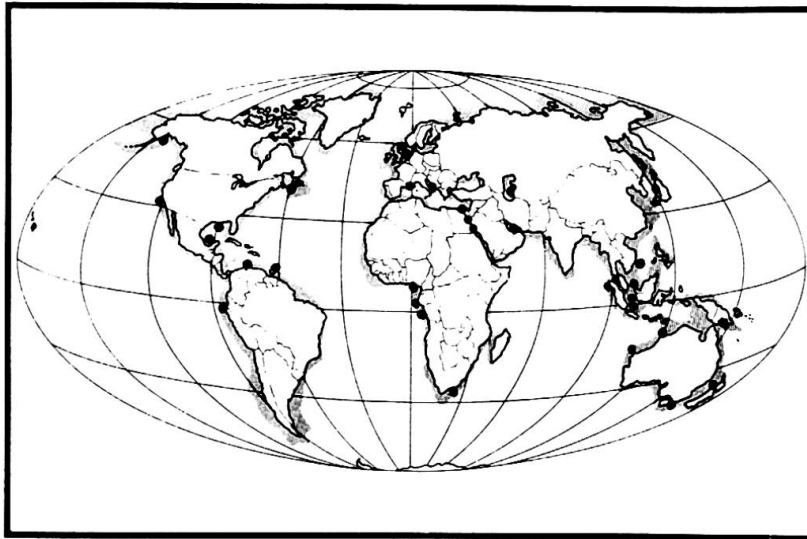


Fig. 1. Regions with offshore oil and gas

In the search for hydrocarbons the objectives of oil companies are the location of promising geological structures, the testing of these structures and the development of successful finds of oil and natural gas. It is realized that deepwater terminals and other near-shore structures may qualify as offshore structures. However, this paper will deal specifically with a discussion of structures which are used in the search and development, or in the exploration and production of hydrocarbons. Associated structures, like offshore storage facilities and underwater pipelines will also be reviewed briefly. Because of the nature of the environment the role of the naval architect in the conceptual design of certain structures is significant. However, because of his particular training, the final formulation of the design configuration is almost exclusively the task of the structural engineer.

Since this paper is a review paper the author decided first to acquaint the reader who is unfamiliar with this field of engineering with the basic types of structures operating in the offshore environment. Subsequently, a discussion of the problems associated with the analysis and design of these structures will be presented.

TABLE I - WORLD OFFSHORE PRODUCTION

YEAR	OIL million bbl/day	GAS billion cu ft/day
1974	8	28
1979	15	59
1984	30	114

OFFSHORE STRUCTURES

Based on the sequence of offshore developments, offshore structures can be grouped in two major categories, namely mobile drilling rigs and production platforms. A third group involving structural engineering could be identified as oil transportation equipment like offshore oil storage facilities, pipe-laying barges and pipelines.

MOBILE DRILLING RIGS

The offshore industry recognizes three types of mobile rigs, namely drillships, jack-ups and semi-submersibles. These units are normally used in the exploration of specific regions of the continental shelf. The jack-up platforms have an operational depth limit of maximum 90 m. For exploration at greater depths the industry uses drillships and semi-submersible drilling rigs. While the advanced units of these latter types have been designed to operate in water depths of 300 m, some of the latest designs have even specified operating depths as large as 600 m. These units are equally suited for operation in calm seas. However, in heavy seas the semi-submersible rigs, because of the partial submersion, provide a more stable drilling platform than the drillships. The roll and pitch of the drillship in heavy weather may force halting the drilling operation.

The cost of these mobile drilling units has lately increased considerably, due to both inflation and more stringent design criteria. Jack-up rigs are now costing between \$14 and \$18 million and drillships may vary in cost between \$20 and \$30 million. Finally, the advanced semi-submersibles may run as high as \$35 to \$40 million. The world-wide inflationary trend has also affected the day rates for these units. A couple of years ago a mobile rig could be contracted at a daily rate of about one thousandth of the initial cost of the equipment. Recently this rate has increased to \$1,500 per one million dollar initial cost.

The design of the drillship is of course entirely the domain of the naval architect. However, the design of the jack-ups and semi-submersibles is a joint effort. While the naval architect is responsible for developing the seagoing characteristics of these units, the structural engineer is normally called upon to carry out the structural design of these rigs. Therefore, in the following sections only the jack-up and semi-submersible drilling units will be discussed.

Jack-up Drilling Platforms

The jack-up, or self-elevating platform, typically consists of a floating cellular hull with retractile legs. While enroute the legs are raised as high as safely possible to limit the drag. On site, the legs are lowered onto the seabed and allowed to penetrate while the hull is being raised out of the water. The major structural feature of these rigs are the legs. These units are mostly three or four legged and are presently designed to operate in water depths of up to about 110 m. Conceptual designs have been developed whereby a two-level jack-up and hull system would allow operation in water depths of about 175 m.

One of the larger typical jack-up rigs is shown in Fig. 2. This unit with a hull measuring approximately 70 m x 60 m x 8 m has three legs with a maximum length of 135 m. The maximum operating depth is 90 m with a 9 m leg penetration and an air gap between the water and the bottom of the hull or platform of 15 m. The legs of this rig are oriented vertically and are constructed as welded steel tubular trusses with a square cross-section. While most platforms have vertical legs, some rigs have slanted legs. Others again have tubular trussed legs triangular in cross-section. In some instances the legs are single thick-walled steel columns, either square or circular in shape.

While most of the jack-ups have to be towed, Fig. 3 shows a self-propelled jack-up, the first of its kind. The vessel-shaped hull measures 85 m in length and 40 m in beam. The four tubular trussed legs of triangular cross-section have a length of 108 m and allow operation in water depth of up to 76 m.

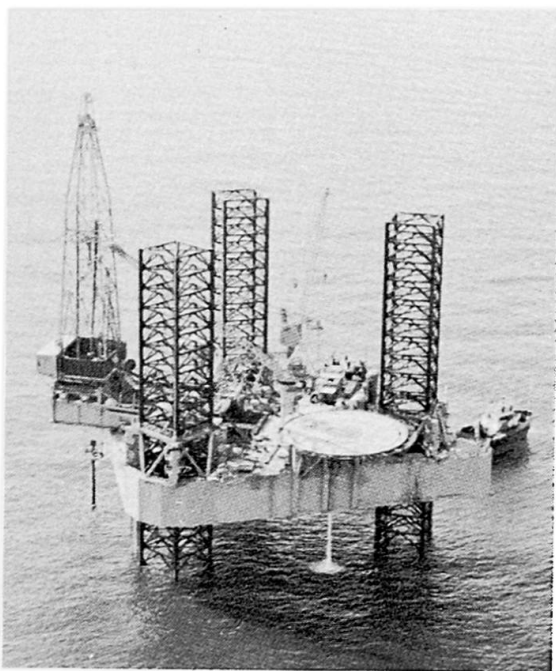


Fig. 2. Jackup Drilling Platform

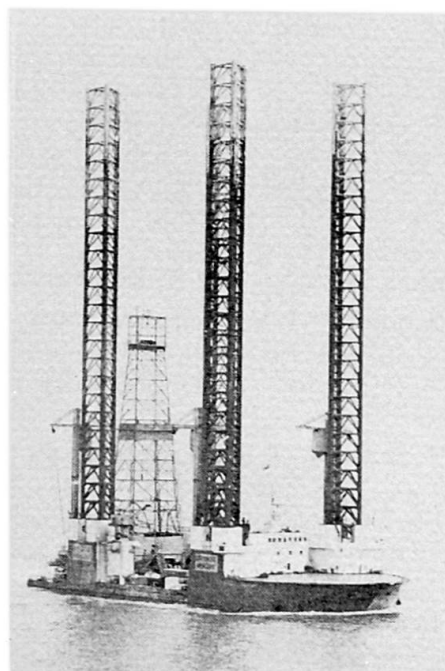


Fig. 3. Self-propelled Jackup

In general, the legs of these jack-ups reach their full length by adding on leg extension sections. These truss extensions are not used when the rig operates in shallower waters. The operating depth for these units, as listed by the owners, entirely depends on the prevailing sea conditions used in the design (maximum design wave, wave spectrum and current). Operating these rigs in a more hostile sea environment requires a reduction of the effective operating depth, not only to achieve an adequate air space but also to maintain the original life-expectancy of the unit. For instance, the deepest rated jack-up rig built to date was specifically designed to meet the stringent requirements of the Norwegian North Sea and can drill in 91.5 m of water during the summer and 84 m during the winter. Operating in other areas, with less severe environmental conditions, additional leg sections can be added to increase the operational water depth to 108 m. The three almost 135 m long slanted legs supporting this unit are square in overall cross-section and have pointed spud cans designed to obtain sufficient penetration and reduce scouring effects on the North Sea floor.

Semi-Submersible Drilling Rigs

The development of semi-submersibles during the last 10 years has shown an evolution from the pontoon-supported, multiple-column stabilizing units to the present-day twin hull rigs. The semi-submersible platform gains its main source of buoyancy from the pontoons or hulls which are submersed below the surface where wave action is less severe. Stability is provided by the vertical columns which pierce the water plane.

Some of the earlier units (1966) were pontoon supported and had three stabilizing columns as shown in Fig. 4. These units were designed to operate in water depths of up to 180 m. More advanced units having the same basic geometry have been designed to operate in depths of 245 m. These rigs have hull dimensions of about 100 m x 100 m. In addition to these units with triangular



Fig. 4. Pontoon-supported
Semi-submersible

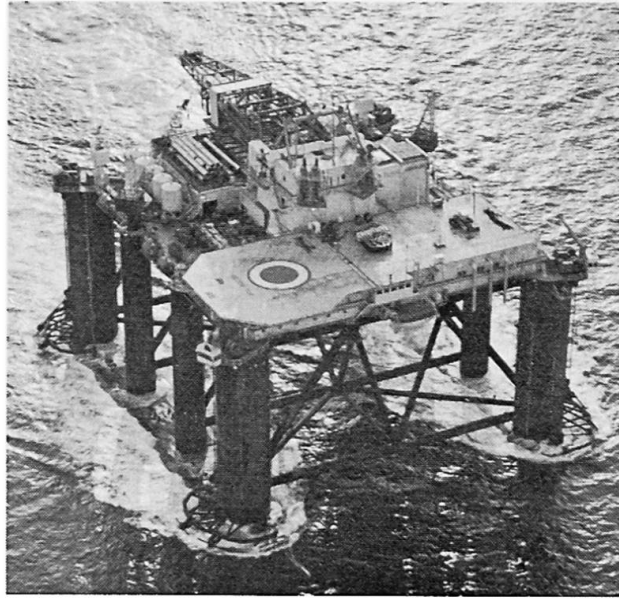


Fig. 5. Twin-hull supported
Semi-submersible

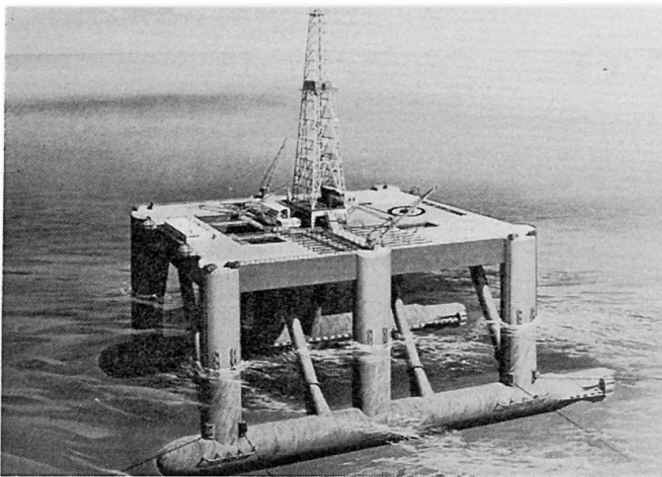


Fig. 6. Twin-hull Self-propelled
Semi-submersible

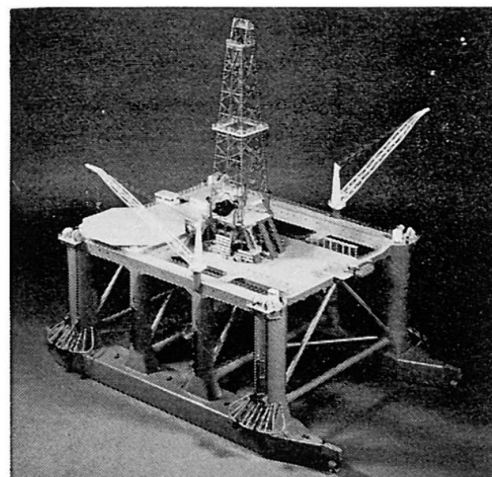


Fig. 7. Twin-hull Self-propelled
Semi-submersible

column layouts, other recently built platforms have pentagonal pontoon-supported column arrangements, capable of operating in water depths of 200 m.

Most of the semi-submersibles recently delivered or presently under construction fashion a twin-hull, column stabilized platform as illustrated in Figures 5 through 8. The deck areas are virtually square, with the overall width of the twin hulls about double the height of the structure. The hull length may vary depending on the self-propulsion system. These propulsion assisted units allow a reduction of the towing time and thus become more effective. Figure 9 shows one of these units in operation in the North Sea.

The operating depth of these newer twin-hulled rigs is almost invariably 300 m, thus allowing exploration of the Continental slopes. Actually, one of the most advanced twin-hull units to be placed in service in 1975 is designed to drill in a maximum water depth of about 600 m. Like all other semi-submersible

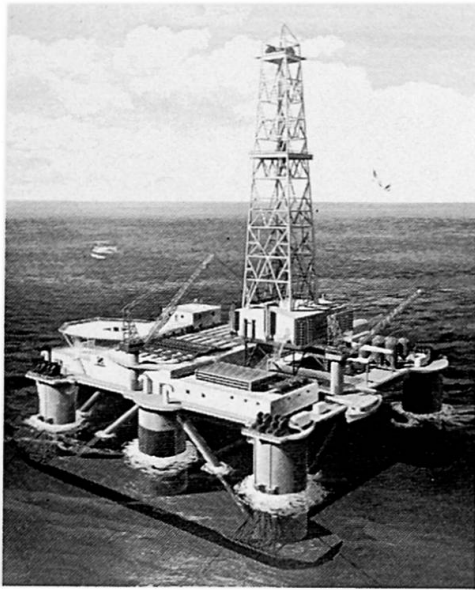


Fig. 8. Twin-hull supported
Semi-submersible



Fig. 9. Semi-submersible in operation



Fig. 10. Self-propelled multiple-hull
Semi-submersible

rigs this unit also uses a standard anchoring system to maintain location. However, because of the tremendous anchor forces of these deepwater units, future designs will probably use the principle of dynamic positioning.

A basically different rig design with an octagonal column layout develops its buoyancy through an orthogonal arrangement of multiple hulls as shown in Figure 10. This unit is self-propelled and can drill in water depths of 200 m. While en route only the longitudinal hulls are submerged, thus limiting the drag forces.

The steel framed semi-submersible units require that special attention be paid to the design of the truss system and the welded tubular connections. Hence rigorous analyses and model tests to determine the sea-going characteristics of these units and the associated member forces are of utmost importance. Great care should be exercised in developing appropriate joint design details, thus limiting stress concentrations wherever possible.

While semi-submersible drilling rigs have been built so far exclusively in steel, recently a design for a concrete drilling rig has been developed. This Condrill platform - as shown in Figures 11 and 12 - can be used for both exploratory drilling and as floating storage and production platform. The caisson type of structure consists of fourteen vertical cylindrical shells with external diameters of 8.25 and 15 m. These cells are poured in a single operation to form a monolithic unit. Six of the cells are capped, while the remaining eight extend above the waterline to support the double-level deck. The total concrete

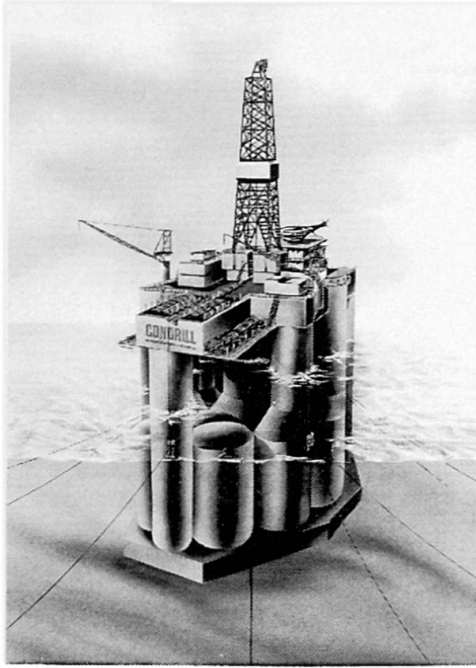


Fig. 11. Concrete Semi-submersible

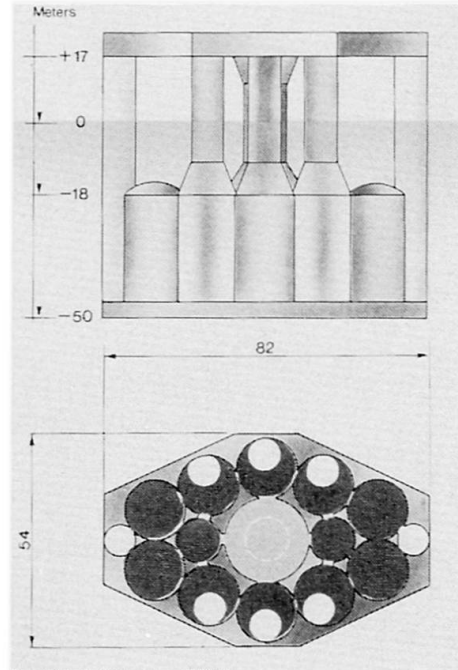


Fig. 12. Dimensions unit shown Fig.11

weight of the structure is about 50,000 ton, sixty percent of which is concentrated in the bottom slab. The storage capacity of this unit is listed as 2,000,000 barrels of oil. Drilling will take place through a 21 m diameter cell extending through the center of the platform. At the deck level this cell reduces to about 10 m in diameter. The unit is designed to operate in a water depth of 300 m and has a drilling draft of 50 m. Under tow this draft is reduced to 30 m.

DRILLING AND PRODUCTION PLATFORMS

Following the successful completion of the exploratory drilling phase, it is necessary to install a platform for the drilling of the production wells and subsequent production. These platforms have typically been designed as steel welded tubular space frames, called jackets. The vertical jacket legs, or columns, support the deck sections, while the diagonal or K-braces together with the horizontal web members provide the primary resistance against the lateral loads due to waves, currents, ice flow, wind and possibly earthquakes.

The smaller jackets - for water depths of up to 100 m - are invariably brought to the site on a barge and either lifted in position or launched off a barge. These platforms are subsequently anchored to the sea floor by driving steel piles through the inside of the jacket legs. The space between the pile and the inside of the leg is subsequently cement grouted to create an integral, well anchored truss structure. Next the deck units and operating equipment will be installed. The jackets are typically fabricated while in a horizontal position as illustrated by the North Sea Ekofisk jacket designed for a water depth of 75 m, (Fig. 13). A completed multiple platform unit located at the Leman Bank field (North Sea) is shown in Fig. 14. For most jackets the wells are normally placed outside the column legs, thus requiring conductor guide frames as shown in Fig. 13.

In case a platform is to be installed in waters with ice field movements, it is necessary to protect the well pipes by placing them inside the column legs. The absence of outside conductor pipes is illustrated by the three-legged platform located in Cook Inlet, Alaska (see Fig. 15). Furthermore, since it is necessary under those conditions that the web members do not pierce the waterline they should be restricted to the under-water portion of the tower. Because of

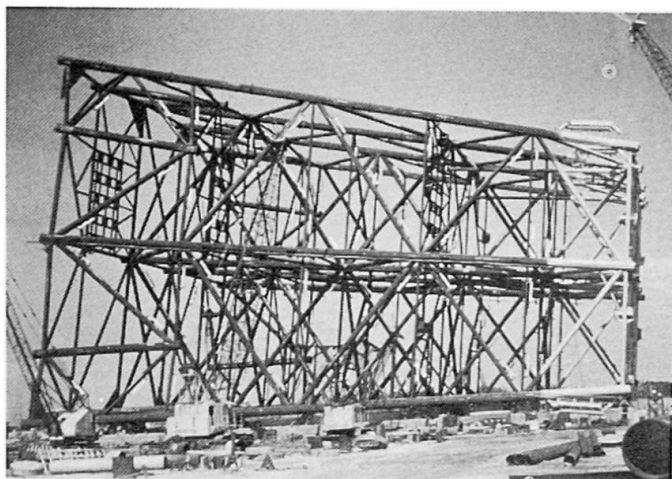


Fig. 13. Jacket at fabricating yard

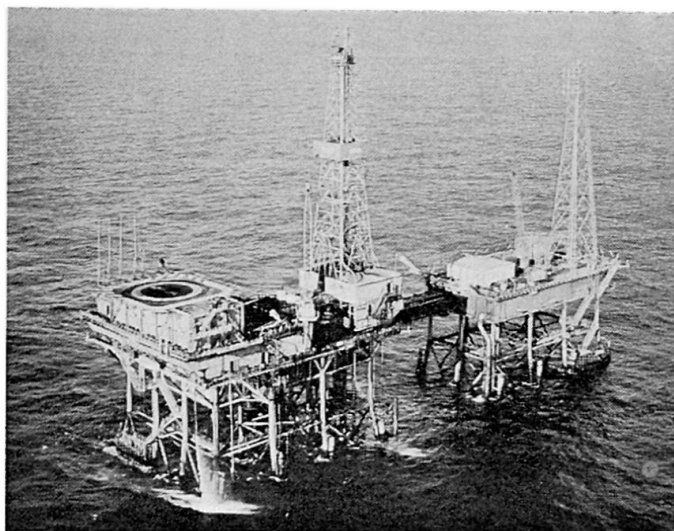


Fig. 14. Offshore Tower

a dry dock and floated out on its side while supported by a specially designed re-usable steel flotation unit (see Fig. 17). The jacket structure has a weight of 21,000 metric tons, while the flotation unit weighed 9700 tons. After the structure was tipped and sunk into place, the flotation structure was retrieved. Forty four 137-cm diameter steel piles, 73 m long and placed on the outside of the corner legs - note pile guides in Fig. 17 - secure the jacket to the sea floor. After placing the two-level deck modules together with the drill towers on top of the jacket, the total height of the structure will be about 220 m. The deck sections and auxiliary equipment, including piles and well conductors will weigh about 13700 tons. Hence, the total weight of the structure will be about

the larger space requirements to locate piles and well pipes inside the jacket legs these sections are substantially larger in diameter as compared to the more standard units (4 to 5 m versus 1.00 to 2.00 m). The increased column sizes and large column surface loads require a substantial internal stiffening by either radial and longitudinal stiffeners or by cement grouting the void spaces after installation, or both. These larger column dimensions provide sufficient buoyancy to float the jacket structure on its side. At location the jacket is upended by flooding the column legs as shown sequentially in Fig. 16 for a 32 conductor, four-legged Cook Inlet platform. Piles driven inside the legs secured the structure to the sea floor.

The largest steel jacket installed to date is the Highland One, a 145 m high structure standing in 127 m of water. This structure, which is one of the four jacket-type towers to be installed for production of the North Sea Forties field, was fabricated in



Fig. 15. Offshore Tower

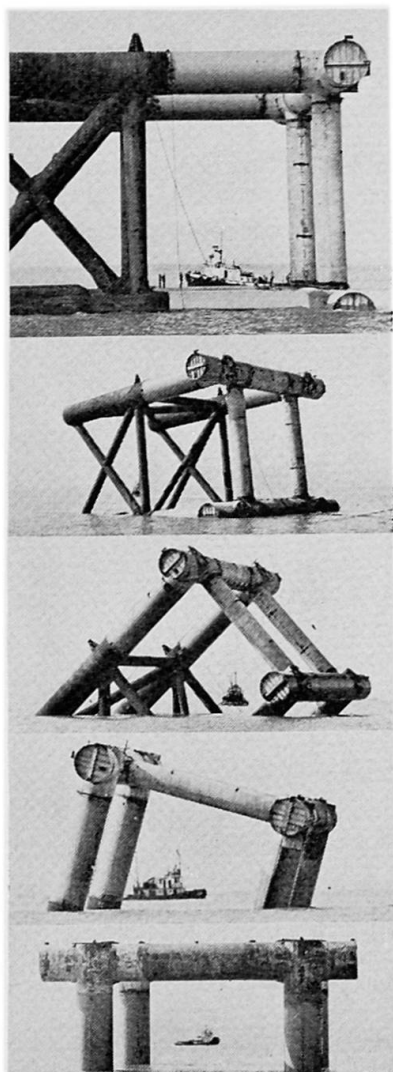


Fig. 16 Launching sequence

about fifteen times the weight of a comparable steel platform. Hence, the installation costs of concrete gravity structures - about 10% of the total cost - is considerably smaller than for the very large steel jacket platforms. The overturning movement under the most extreme sea conditions is completely counteracted by the structure's gravity. Under those circumstances it is imperative that the surface and near-surface soil conditions of the sea floor should assure the stability of the structure and soil. Hence, the soil layers should be horizontal in order to prevent sliding and to assure uniform consolidation.

35,000 tons. The cost of this unit when completed is expected to be about \$165 million.

The largest steel jacket platform presently under design will stand in the 162 m deep water of the North Sea Thistle field. The total height of this unit to the top of the flare stack is to be 280 m and the weight about 29,000 tons. This jacket will derive its flotation capability from two 9 m diameter legs and two additional cylindrical tanks, 82 m long and about 9 m in diameter, attached permanently to the two flotation legs. These supplemental flotation units provide a 70,000-barrel oil storage capacity when in operation. The daily production from the 60 wells to be drilled from this platform is estimated at 200,000 bbl.

The basic concept of oversized column legs on one side, in order to float the jacket out on its own buoyancy, is not new and has been used successfully before in platform designs offshore California. One of the critical aspects of the steel jacket-type platforms in a North Sea environment is the risk of upending the structure and the time and costs involved to drive the piles in order to tie the platform down to the sea bed. The latter time element reflects the risk that the jacket might be subjected to heavy weather before being properly anchored down. To reduce this risk, concrete gravity structures, serving as drilling, production and storage facilities, have been introduced in the offshore industry for the first time last year.

The concrete gravity units have the advantage that they do not need to be anchored to the sea floor because of their enormous dead weight -

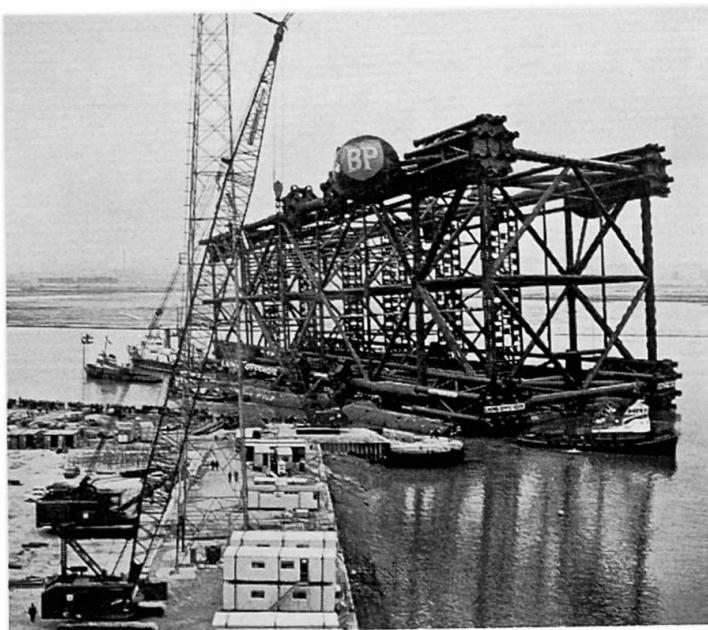


Fig. 17 Jacket on flotation unit

Since weaker layers may underlie stronger but shallower surface layers, deep skirts which are to penetrate the stronger upper layers and to develop the strength of the lower soils are commonly proposed.

The first concrete gravity structure is the 1,000,000 bbl storage tank which was installed at the North Sea Ekofisk field in 1973. Fig. 18 shows the structure while under construction and Fig. 19 gives a view of the tank as installed at the 75 m deep Ekofisk site. The tank is used for both production and storage. The inner tank complex, as illustrated in Fig. 18, is protected from the direct wave impact by an almost circular perforated breakwater wall.

The concrete gravity structures under construction at this time combine storage facilities, housed in a multi cellular system at the base of the structure, with the typical drilling and production facilities. The 1,000,000 bbl storage capacity of the Condeep design is provided by the nineteen vertical cylindrical tanks each with a 20 m outer diameter and a wall thickness of about 75 cm. The tanks are arranged in a pentagonal array as shown in Fig. 20. Sixteen of the tanks are capped at a height of 50 m, while the remaining cylinders form the base of the three post-tensioned concrete columns which will rise to about 20 m above still water and will carry the steel deck structure. (See Fig. 21). Of the five Condeep platforms presently under construction around the North Sea the first one ordered will be installed in 1975 in the Beryl field at a water depth of 110 m. Two other units are destined for the Brent field, where they

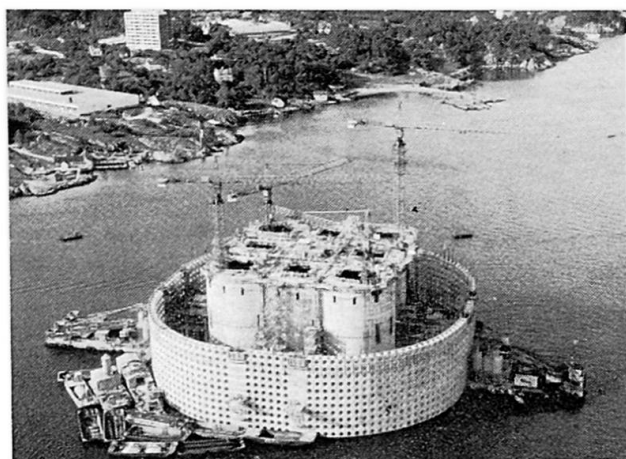


Fig. 18 Ekofisk tank under construction

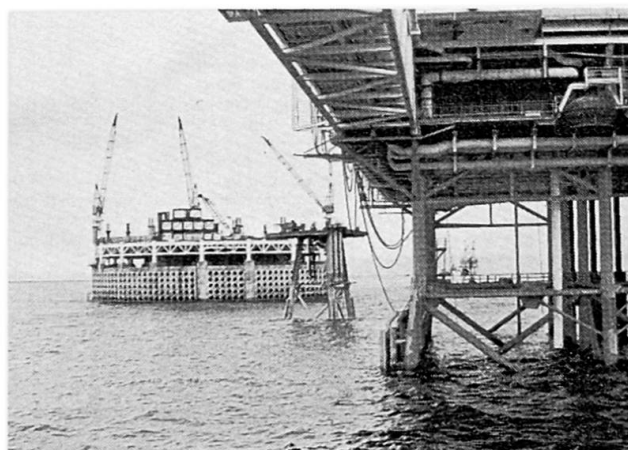


Fig. 19 Ekofisk tank installed

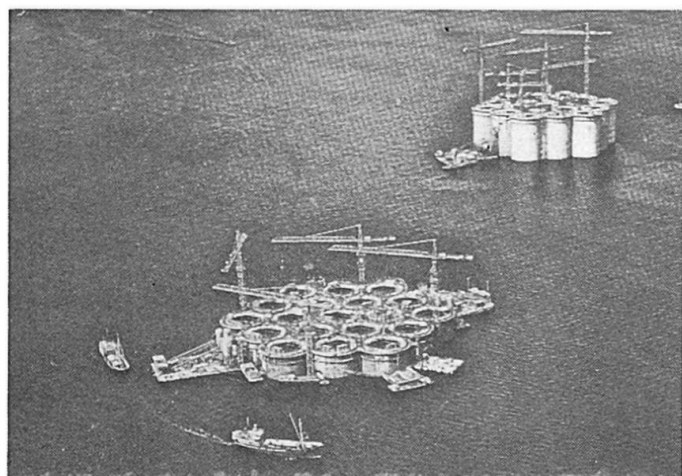


Fig. 20 Condeep Platform under construction

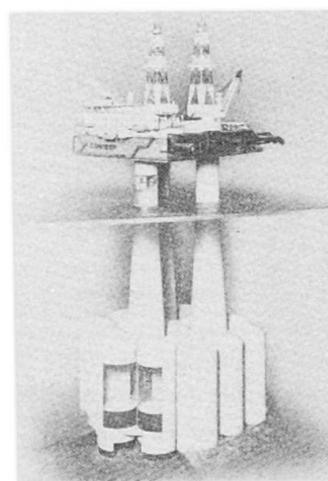


Fig. 21 Condeep Platform

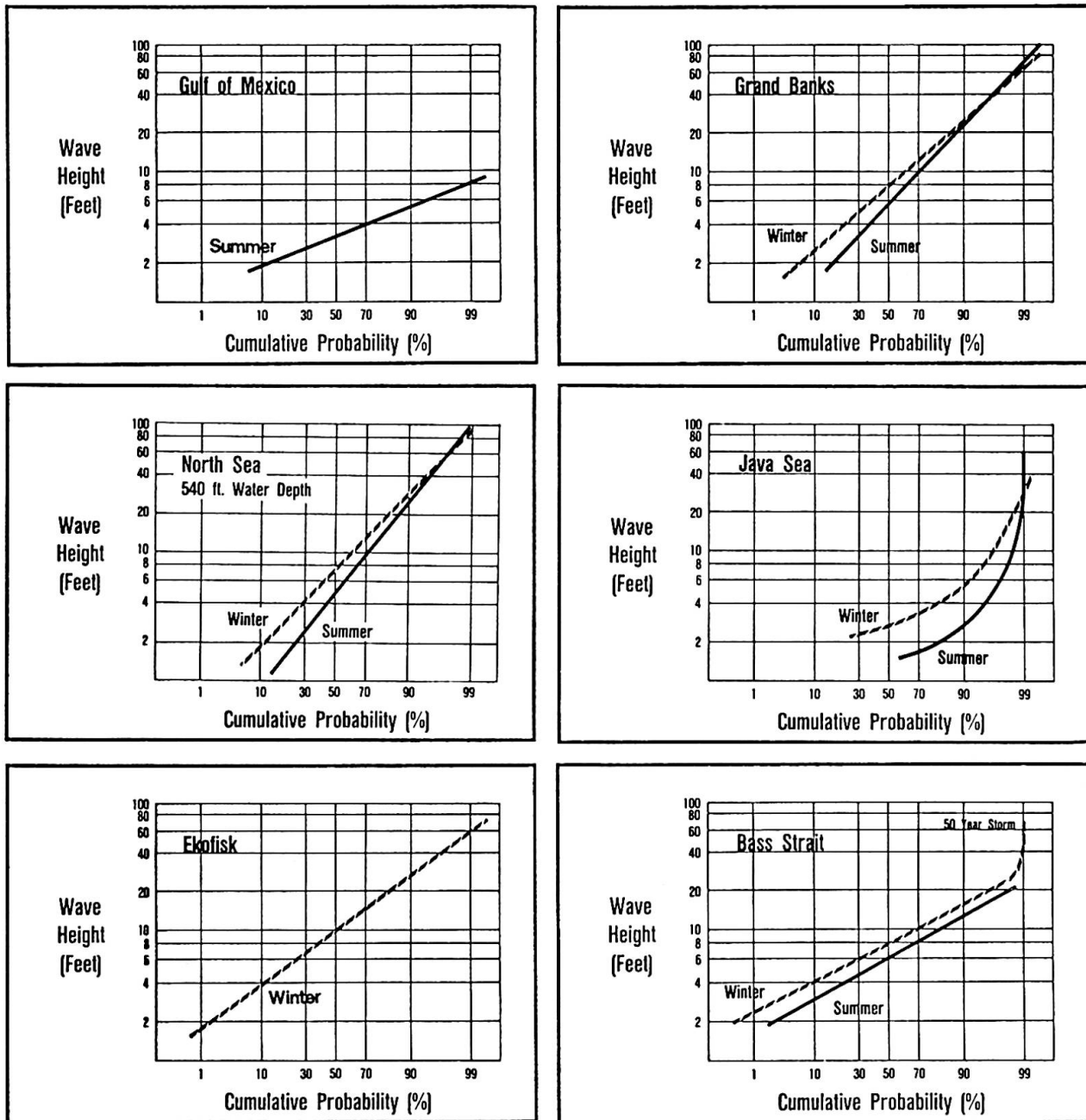


Fig. 22 Wave Data

a steady operation of the barge and to prevent serious overstress in the line. Hence, larger more stable lay barges have been developed. Following the experience gained from operating the more stable semi-submersible drilling rigs, the modern pipelaying barges are also designed using the principle of semi-submergence. These units have lengths of up to 180 m and widths of 60 m. The greater deck lengths result from the requirement to handle double jointed pipes, allowing the barge to advance in larger increments. In order to design pipelines for these highly complex dynamic conditions, advanced programs of structural analyses, considering realistically both the elastic and potentially inelastic response

¹ Y. Goren, "Functional Design of Drilling and Construction Platforms," preprint 1973 Offshore Conference, University of California, Berkeley, California, USA.

of the pipeline during these pipelaying procedures are essential. Under those circumstances the associated structural design of the launching mechanism and the lay barge itself constitute an integral part of the overall design of these units.

DESIGN CONSIDERATIONS FOR OFFSHORE STRUCTURES

For the design of offshore structures the structural engineer has been and will be continually challenged to design for an environment which presents engineering complexities uncommon to the typical design considerations for land structures. In principle, structural design is determined by three major considerations, namely the environmental conditions of loading, the service requirements and the material properties. However, the final structural formulation will depend on the accuracy of specific analytical procedures which permit the evaluation of the structural response to any given set of loads. A design will be considered safe and functionally acceptable when the analytically-derived values do not surpass the limits set by the behavior criteria. The factor of safety to be reflected in these criteria, depends primarily on reliability of material properties, knowledge of design loads, accuracy of analytical procedures used to evaluate the structural response, serviceability, maintenance, and repair and replacement costs. The factor of safety can be reduced when engineering aspects as material properties and design loads are well defined and analytical procedures are highly reliable. On the other hand, economic factors associated with serviceability, maintenance, repair and replacement might well require an increase of the factor of safety. In the following sections, certain of the more important considerations in offshore design will be reviewed briefly.

ENVIRONMENTAL LOADINGS

The predominant environmental design loads are a direct reflection of the extreme sea state (waves, ice-flow, surface and tidal currents). The maximum design waves and wave spectra used in design differ significantly for different locations in the world. (See Fig. 22 and Table II.) Unfortunately, information of this nature is far from complete and often insufficiently accurate data has to be used to design drilling and stationary structures. The lack of this information is particularly critical in the design of drilling rigs which, during their design lifetimes, may have to operate in several different locations. Depending on the location, potential earthquake forces can play a predominant role in the design of stationary offshore structures. The study of offshore tower structures subjected to random earthquake excitations poses problems which are not encountered in similar land structures. Firstly, in the trussed ocean structures, the hydrodynamic forces on the structure introduce a non-linearity in the governing equations of motion, even when the material non-linearities are absent. Secondly, such structures have very high fundamental periods - from 2 seconds for towers in depths of about 120 m to over 5 seconds for those in depths of 300 m. This phenomenon prolongs the time to reach a stationary process. The response of offshore tower structures to earthquakes is, therefore, essentially a transient response. For the relatively more rigid gravity structures the non-linearity will be less severe. However, on the other hand the earthquake loads will be far greater and the capacity to absorb energy in a ductile fashion will be significantly less or non-existent.

MATERIAL CONSIDERATIONS

The previous aspects as applied to the design of relatively simple structures under well defined service conditions will normally pose little problem. However, for structures, which, because of the environmental circumstances will

TABLE II WAVE AND CURRENT DATA ¹

Location	100 Year Wave		50 Year Wave		Current Maximum Speed	Average Wave Height—Feet		Wave Height Above 10 Ft. Percentage
	Height Fl.	Length Fl.	Height Fl.	Length Fl.		Winter	Summer	
North Sea								
240 Ft. Water Depth	76	1093	74	1051		6.75	4.0	11.9
540 Ft. Water Depth	94	1394	89	1330	4.3 Ft./Sec.	8.33	5.65	22.8
Java Sea								
150 Ft. Water Depth	28		26.5		1.5 Knots.	2.62	2.25	Less than 1%
"Tsunami"—Max.	(100-135)							
Bass Strait			69 Highest 10%		2.2 Knots.	8.28	7.14	20.4
Gulf Of Mexico								
Eugene Island (88 Ft.)	53.5	949				4.87	3.83	Less than 2%
300 Ft. Water Depth	58	1232	48.5 (25 yrs)	986				
Grand Banks								
600 Ft. Water Depth	79		70 (25 yrs)		3 Ft./sec. 5 Ft./sec. (100 yrs)	7.77	5.65	20.9

be subjected to severe and variable loads, advanced engineering judgment regarding the behavior and material properties is of decisive importance. Off-shore structures, or ocean structures in general, fall undoubtedly in the latter category. The complexity of the design of such structures is not only affected by the environmental and material considerations but also by economic aspects. The need of advanced engineering principles is of utmost necessity to assess the economic feasibility of developing offshore energy resources.

The design of offshore structures normally considers load conditions under both towing and in-service conditions. One of the main loadings to be considered is undoubtedly the survival load which reflects an extreme storm or earthquake condition and depends on the projected life of the structure. This condition will normally be the governing factor in the design of fixed structures in shallow waters of relatively calm seas. However, when fixed and mobile offshore structures are to be exposed to hostile seas and are placed in deep water environments the typical survival design loads may well cease to be the decisive load condition. Particularly, if based on an operational service capability, the life expectancy of the structure has to be well defined. In that instance, the repeated loads causing relatively low cyclic stresses may well become a pre-dominant factor in the design of the structure. In that instance, an optimum design approach to develop a fatigue-resistant structure has to be considered as well. This aspect may be particularly critical for steel structures and the design of tubular steel connections. A similar concern has also been voiced regarding the fatigue strength of reinforced and prestressed concrete in sea water. Corrosion and corrosion fatigue are important factors which affect the design of offshore steel structures. Cathodic protection has proven to be effective to improve the life expectancy of offshore structures. In case of concrete offshore structures, limiting crack widths will be necessary to prevent corrosion of the reinforcing steel and possible fatigue. Also cyclic temperature effects due to the storage of hot oil and cold sea water may pose certain problems in thick-walled concrete storage facilities. Information regarding the generally complex material problems in both steel and concrete is necessary to enable the structural engineering to develop an optimum offshore design.

will stand in water 140 m deep. These two structures should be installed by 1976. The base of these gravity structures are typically constructed in a graving dock. After the walls of the lower cylinders are built high enough so that the entire unit can float, the dock is flooded and the structure is towed to a deep-water site to complete the construction, including installation of the deck sections. A limiting factor in the construction of these units is the lack of sufficient deep water facilities.

In addition to the five Condeep structures under contract, six other concrete gravity structures are presently under construction. Three of these units are of the Sea Tank design while a fourth structure is designed by Andoc. The two designs are in concept similar to the Condeep design, except that the oil-storage base is square rather than pentagonal and the decks are supported by four columns rather than three. The two remaining structures are designed by C.G. Doris and are similar in concept to the original Ekofisk storage tank, using the perforated breakwater concrete wall.

Recently, also steel gravity structures have been introduced. Four structures designed by Technomare and intended for the 85 m deep Loango field near the mouth of the Congo River are presently being fabricated. These structures consist of a steel tubular trussed tower, with the six columns arranged in a pentagonal array. This tower structure is supported by a triangular base truss with a flotation cylinder located at each corner.

With future exploration and production moving to ever increasing depths neither the steel piled jacket nor the gravity structure seem to be a feasible solution, particularly when the sea environment is very hostile. Hence, recent studies have been focused on the development of tension-leg platforms. One design would have a steel tubular trussed frame - in principle similar to the triangularly based semi-submersible drilling rigs as shown in Fig. 4 - held down by vertical pre-tensioned cables from each of the three corner flotation columns. Such a system would require deep-water sea bed anchors drilled into the ocean floor.

SUPPLEMENTAL OFFSHORE FACILITIES

In addition to the production platforms the development of offshore resources requires equipment necessary to bring oil and gas to shore. Foremost in this category are underwater pipelines and pipelaying barges. Pipeline platforms and offshore storage tanks play an integrated role in the oil and gas transportation system. However, from a structural viewpoint their design criteria are similar to the stationary structures discussed earlier.

Laying pipelines in calm shallow waters has been extremely easy as compared to the complexities of laying lines in deeper waters and under adverse weather conditions. The larger water depths (up to 200 m), the increasing distance to shore, the higher operating pressures and the larger forces require large-diameter thick-walled line pipes. The launching of these deepwater pipelines from conventional lay barges has become virtually impossible. Therefore new very large barges have been developed and are presently in operation. In order to guide the pipe from the barge deck into the water the so-called stinger, which provides a predetermined curvature to the line has become a standard feature of these modern lay barges. These stingers can be structurally articulated (multiply hinged) or be built in a few sections with adjustable roller supports. Under all circumstances it is essential that the line remains under tension to prevent collapse. While the stinger configuration and pipe tension permit a control of the pipe deformation, the sea state will be the ultimate limiting factor in the design of the line. The roll, heave and pitch of the lay barge should be minimized to allow

SUMMARY

The search for offshore oil and gas and the subsequent development has opened up an almost entirely new field of structural engineering. Several types of structures, often very large, have been developed for both exploration and production. The design of these units is highly complex and requires detailed information regarding environmental and service conditions as well as material properties.

RESUME

Les forages marins à la recherche de pétrole et de gaz, et les suites qu'ils comportent, ont ouvert un champ presque entièrement nouveau aux charpentes. Plusieurs types de structures, parfois gigantesques, ont été réalisées tant pour l'exploration que pour la production. Le dimensionnement de ces ensembles est extrêmement complexe et nécessite une information détaillée concernant les conditions d'environnement et d'exploitation, et les qualités des matériaux utilisés.

ZUSAMMENFASSUNG

Die Suche nach Öl und Gas im küstennahen Meer und die zugehörigen Entwicklungen haben ein praktisch neues Gebiet im Bauwesen eröffnet. Verschiedene Typen von oft gewaltigen Bauwerken wurden für Suche und Förderung entwickelt. Entwurf und Berechnung derselben ist in hohem Masse komplex und erfordert eingehende Information über Umwelts- und Betriebsbedingungen sowie über Materialeigenschaften.

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Foundation Structures for Tall Buildings

Structures des fondations pour les maisons hautes

Foundationen für Hochhäuser

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1. FOREWORD

A foundation must transmit the load of a structure to the underlying soil or rock safely and without excessive settlement. Decisions in design of all foundations must always be made with this object in view. To achieve the object the following design principles are stipulated in the Structural Standards for Building Foundations of the Architectural Institute of Japan, which is a nation-wide organization of building engineers and architects:

i) The foundation should be supported by a strong, stable soil stratum or rock; the support by incompetent soils should be avoided.

ii) The foundation of a building should not be supported by different soil strata with noticeably different characteristics.

iii) A building should not be supported by foundations of different types.

iv) The stresses induced in the soil at the foundation base should be distributed as uniformly as possible throughout the plan of a building, and should afford a sufficient margin for safety against failure of the supporting soil strata as well as development of excessive settlement.

v) The bases of columns should be tied with foundation beams of sufficient stiffness so that the entire foundation forms a rigid grid to act as a unit.

Even though this paper is concerned with the foundation design for tall buildings, one needs hardly add anything to these general principles. However, because of characteristics peculiar to the structural systems and loading conditions of tall buildings, a number of problems may be pointed out which require special considerations. In relation to the foundation of a tall building, such problems will briefly be summarized and discussed in the following sections from three major standpoints:

1. Static weights
2. Wind forces
3. Seismic forces

2. STATIC WEIGHTS

Tall buildings are characterized, first of all, by their large weights far heavier than usual, low to medium-rise buildings. The fact frequently imposes rigorous problems on the design of foundations in various aspects.

AVERAGE WEIGHT - In Fig.1, average weights (dead plus design live loads excluding weight of foundation) are shown in metric tons per

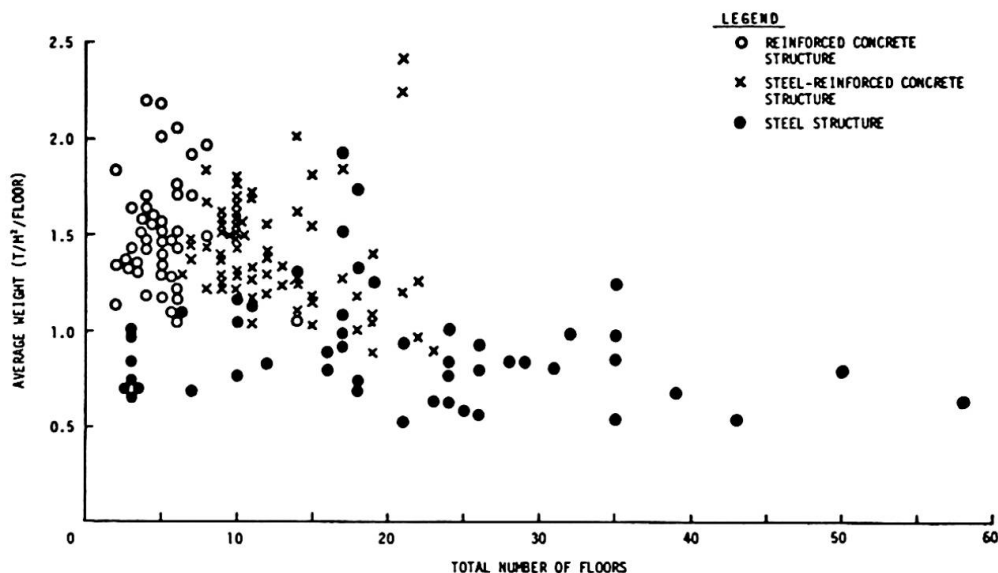


Fig.1

unit area per one floor with respect to typical Japanese buildings of different structural types. In spite of the wide range of values for low to medium-rise buildings, it may be seen for tall buildings that the values nearly tend to converge and may likely be between 0.5 and 0.8 tons/sq.meter/floor. In this connection, it is a common practice to accommodate a tall building with the basement of reinforced concrete or steel-reinforced concrete whatever the structural material of the superstructure may be. Unit weight of the basement alone is approximately equal to 1.75 tons/sq.meter/basement floor on the average.

In Fig.2, average weights of foundations (footing, mat, foundation beam, excluding weight of piles or piers) are also shown in terms of the total number of floors of a building, indicating that the values are in the range from 0.12 to 0.17 tons/sq.meter/floor for typical, tall buildings.

Hence, the estimate of the total average weight of a tall building including foundation may probably be in the range of 0.6 to 1.0 tons/sq.meter/floor. Although little information is available to the author concerning the weights of tall buildings in foreign countries, they may likely be of no significant difference from the above rough estimate.

BEARING CAPACITY - Fortunately, almost all major cities in this country are underlain by firm and stiff sandy or gravelly strata of sufficient thickness, which geologically belong to Diluvial deposits and can be encountered from the ground surface within a depth from 10 to 30 meters. Therefore, all of tall buildings designed so far are supported by those strata using either a spread foundation

or a pier foundation of rather short length, being able to comply with Item i) in the aforementioned design principles. Up to the present time, no tall building has been constructed with long, flexible pile foundation.

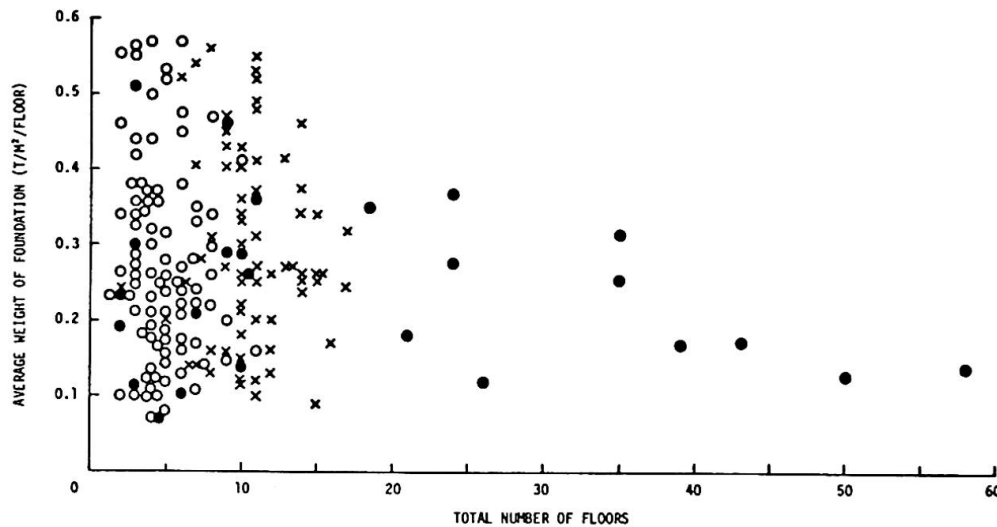


Fig.2

In practice, the ultimate bearing capacity of soil is estimated on the basis of Terzaghi's bearing capacity formula (Terzaghi 1951)

$$q_{ult} = \alpha c N_c + \beta \gamma B N_\gamma + \gamma D_f N_q \quad (1)$$

where

- q_{ult} : ultimate bearing capacity
- c : cohesion of soil
- γ : unit weight of soil
- α, β : shape factor
- N_c, N_q, N_γ : bearing capacity factor
- D_f : depth of embedment
- B : width of foundation

The ultimate bearing capacity thus calculated at the design stage is usually verified by performing a field plate-loading test before or during construction. In Fig.3, a hatched zone is shown which represents the range of bearing pressure vs. settlement curves in the field plate-loading tests for sand and gravel strata supporting tall buildings. These tests are ordinarily performed by using a small loading plate 30 by 30 or 45 by 45 centimeters square. The majority of the loading tests give results in reasonable accord with those estimated by eq.(1). Furthermore, in cases where actual tall buildings provided with mat foundations of large dimensions and the basement of considerable depth are involved, real bearing capacity may probably be much larger than are shown in Fig.3 because of the effects of large width

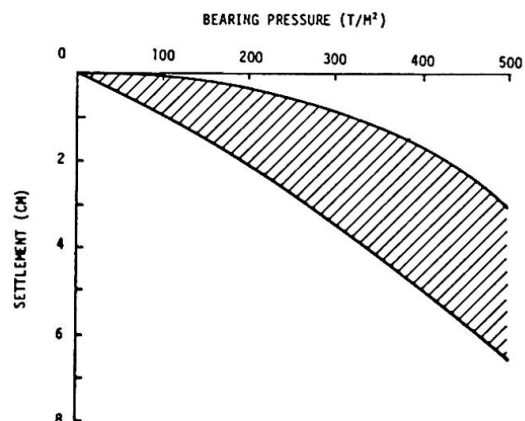


Fig.3

and deep embedment, which are accounted for by parameters B and D_f in the second and third terms of eq.(1), respectively.

When a pier foundation is used beneath the base level of a tall building, the bearing pressure at the bottom of a pier sometimes approaches to about 400 tons/sq.meter in our recent experience. In such a case, the full-size loading test of a pier is usually required and, so far, the safety of the bearing soil strata has been verified to be still sufficient. In some cases, for the purpose of estimating the ultimate bearing capacity at the design stage, Meyerhof's formula (Meyerhof 1950) is utilized, which is known to be pertinent for a deep foundation allowing larger bearing capacity factors than those in eq.(1). However, the compatibility of this analytical approach with the field loading test results does not seem to be completely established yet.

If the bearing pressure comes to an extremely high magnitude, the safety of foundation must be ensured not only against the sliding failure, which is primarily governed by shearing strength of the soil and analytically represented by Terzaghi's or Meyerhof's formula, but also against the so-called crushing failure which is related to crushing strength of individual soil grains themselves. Judging from a few test results performed so far, the ultimate bearing capacity resulting from crushing failure seems to be approximately

1800 tons/sq.meter for sandy gravel
2000 " for cemented sand.

In consideration of the aforementioned average weight of tall buildings, these knowledge and experience may provide a basis for stating that there is no practical limitation of the height of a building from the geotechnical standpoint, provided that competent materials with bearing characteristics not inferior to those shown in Fig.3 can be encountered within a reasonable depth from the ground surface.

SETTLEMENT - Any building undergoes settlement during construction and, under adverse conditions, may suffer a long-continuing, post-construction settlement caused by consolidation of the underlying cohesive soil deposits.

The former is usually referred to as immediate settlement. The immediate settlement results primarily from elastic compression of the soil mass beneath the loading area and, in addition, is associated with the recompression of rebound or heave taking place as a consequence of stress relief by excavation if the construction of a deep basement is involved.

The elastic settlement may be evaluated on the basis of theoretical analysis of an elastic solid. In the aforementioned A.I.J. Structural Standards, the following formula (Steinbrenner 1934, Fox 1948) is recommended for the evaluation:

$$S_o = \mu_H \mu_D q \sqrt{A} / E \quad (2)$$

where

S_o : elastic settlement
 q : average loading intensity
 A : contact area of foundation
 E : modulus of elasticity of soil mass
 μ_H, μ_D : settlement factors

With respect to a few tall buildings in the city of Tokyo, the results of computation on the basis of eq.(2) are compared in Table 1

with actually observed elastic settlements, indicating fairly reasonable agreement.

Table 1 : Computed and Observed Elastic Settlements of Tall Buildings

Building	Total Number of Stories	Settlement(cm)	
		Computed	Observed
A	39	2.2	0.8
B	43	1.5	1.5
C	50	1.3	1.5

For the computation in Table 1, moduli of elasticity were estimated on the basis of the load-settlement curves obtained by field plate-loading tests. In addition, in the case of Building C, the average modulus of the supporting soil mass was also measured dynamically by seismic exploration. It is interesting to note that the results of seismic exploration can provide another reasonable determination of static soil modulus if the observed value is pertinently modified by taking into account the strain levels within the stressed soil mass (Seed 1969).

Under the present situation, the analytical procedure to evaluate the settlement of a pier foundation seems not yet successful even as a crude approximation for practical purposes. Therefore, the evaluation is usually made by performing a full-size loading test or on the basis of available, previous data under appropriate conditions similar to the new site.

Differential settlements have entailed no critical consequences so far for tall buildings of a simple shape, probably because of the physical and structural reasons: (1) Since the buildings are designed so as to rest on a firm soil stratum in compliance with Item i) of the design principles, the maximum settlement is already limited to such an extent that no significant differential settlements occur, and (2) the tall buildings essentially possess high, structural stiffness to withstand vertical distortions and it functions, in turn, to minimize the differential settlements by redistribution of the column loads. Particularly, the presence of the basement having thick walls and a rigid foundation grid makes an important contribution in this respect.

An exception is the case shown in Fig.4, i.e., a tall building with structurally united low-rise annex. Large difference in weights of the high-rise and low-rise portions of the building shown in Fig.4(a) will probably result in significant differential settlements. A preventive measure is usually taken in practice in such a way that both portions are first constructed separately and, immediately prior to the completion, they are connected and finished. For a tall building with partial pier foundation as shown in Fig.4(b), which

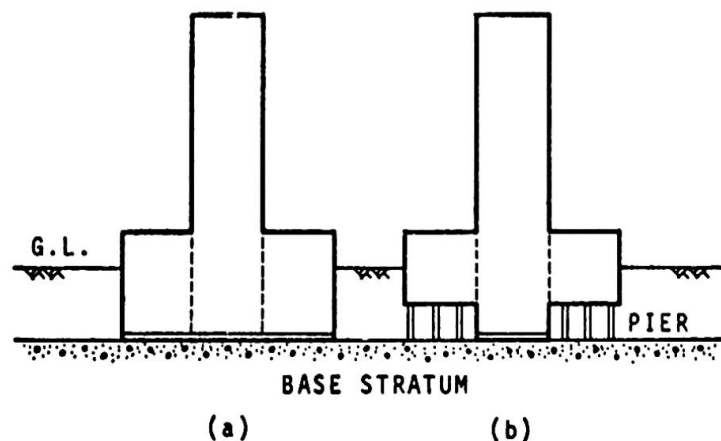


Fig.4

inevitably violates Item iii) of the design principles, the same process of construction is frequently adopted since, as pointed out previously, much uncertainties are involved in estimating the settlement of a pier foundation.

The rebound at the bottom surface of excavation frequently presents a troublesome problem. Reports from a few building sites (Endo et al 1969) disclosed that the observed rebounds amounted to

32 mm for a 16 meters deep excavation
60 mm for a 25 meters "

The rebound taking place at the bottom of excavation can not be noticed unless special measurements are made. It is required for a precise observation of the rebound to install reference points immediately below the proposed bottom level of excavation and to carefully protect them against damage by excavation works. On the other hand, it appears possible to a certain extent to calculate the approximate amount of rebound by referring to the slope of unloading branch of load-settlement curve obtained by a field plate-loading test, provided that the stratification of underlying soil deposits is not too much complicated.

When there is no noticeable non-uniformity in the distribution of column loads, no special measures are taken in practice even if appreciably large rebound is anticipated. However, for a building with extreme difference in weight distribution, the problem of different recompression is usually solved by providing construction joints as described previously.

The second type of settlement results from consolidation of underlying cohesive soil deposits, which increases continuously even after the completion of a building. In fact, in the majority of cities in our country, the firm bearing stratum is usually underlain by clayey soil deposits. Fortunately, however, they are the sediments in the geological era of old Diluvium and in most cases highly over-consolidated, resulting in no critical consequence from the standpoint of consolidation settlement.

FLOATING FOUNDATION - Where a firm stratum of sufficient load supporting capacity can not be encountered within a reasonable depth from the ground surface as is frequently seen in a city underlain by marine or lacustrine sediments of low strength and high compressibility, an effective way of constructing a building of low to medium height is to use the so-called floating foundation.

The basic concept of floating foundation is that the weight of a building is compensated by the weight of the excavated soil so as to impose no additional loads upon underlying soil deposits as a result of constructing the building. The term of floating foundation may be rather misleading and, in a strict sense, it should be referred to as a compensated foundation (Zeevaert 1972).

Now, referring again to the average weights of tall buildings described previously, assume approximately

average weight of superstructure	= 0.65 t/m ² /floor
average weight of basement	= 1.75 t/m ² /floor
average weight of foundation	= 0.15 t/m ² /floor
unit weight of soil mass	= 1.50 t/m ³
average story height of the basement	= 4.00 m

and let

N : total number of floors
N_B : total number of floors in the basement

W : total weight of the building per unit area

W_e : total weight of the excavated soil mass per unit area,

then

$$W = 0.65 \times (N - N_B) + 1.75 \times N_B + 0.15 \times N \quad (3)$$

$$W_e = 1.50 \times 4.00 \times N_B \quad (4)$$

To establish full compensation, eqs. (3) and (4) must be equated:

$$W = W_e \quad (5)$$

From eqs. (3), (4), and (5), one obtains

$$N_B \approx 0.16N \quad (6)$$

and Table 2 shows a few numerical solutions of eq. (6). The above computation is merely a very crude arithmetic; nevertheless, eq. (6) or Table 2 implies that the application of the concept of fully compensated foundation may practically be precluded for a tall building of more than approximately 40 stories.

Table 2 : Required Depth of Basement for Fully Compensated Foundations

Total Number of Floors	20	30	40	100
Required Number of Basement Floors	3	5	6	16

3. WIND FORCES

Winds acting on tall buildings develop large temporary loading which must be delivered ultimately to the soil through the foundation or the basement walls. Shear force and overturning moment resulting from wind loads at the foundation level of a tall building are both characterized by their extremely large magnitude. Judging from experience of designing tall buildings up to the present time, wind forces and wind moment at the foundation level become larger for a typical building of more than 50 to 60 stories than those resulting from earthquakes even in this country of extremely high seismicity.

As a tall building is usually accommodated with the basement of a considerable depth, the large shear force is resisted by the difference of earth pressures acting on leeward and windward faces of the basement as well as the frictional forces of surrounding soils along its side and bottom faces. If a building is supported on pile or pier foundation, the lateral resistance at the top of the piles or piers contributes as well to withstanding the shear force. If this is the case, however, little benefit of frictional resistance along the bottom face of foundation may be expected because of loose contact of the soil or separation resulting from ground subsidence.

In practical analyses, the resistances by Rankine's pressure and shearing strength of the soil, acting on each corresponding face of the basement, are usually taken into consideration. As to the lateral resistance of piles or piers, the beam-on-elastic-foundation method or its extension to plastic range are frequently referred to. As is well known, however, the deformations required for full mobilization of these resistances are not necessarily the same and, moreover, may likely exceed the acceptable limit of movement. It is an important but difficult question at the present time to calculate the contribution of each resisting component compatible with the tolerable displacement, because so many complex factors are involved (DeSimone 1972).

The overturning moment causes an increase of bearing pressure or pile load on the leeward side and a decrease on the other side of the foundation; the former must be withstood by bearing capacity of the soil with an adequate margin for safety. If the overturning moment becomes still larger and the decrease of bearing pressure at the windward edge of the foundation exceeds the static pressure, the problem of uplift is encountered, which is an important matter peculiar to a tall building. The occurrence of uplift may be interpreted as the commencement of a transient motion from stable to unstable state of a structure, it can not be overlooked and should be avoided if possible.

To illustrate the possibility of uplift by wind loading, now assume a simple, prismatic model of a building as shown in Fig.5. The model is assumed to be directly placed on the ground surface. The total height is assumed equal to $3.5N$ meters, where N is the total number of stories and the average story height of 3.5 meters may be used without introducing much error. If 0.65 tons/sq.meter /floor is again assumed as the average unit weight of tall buildings, it is apparent that the bearing pressure of the supporting soil is equal to

$$\sigma_{\text{static}} = 0.65N \text{ t/m}^2 \quad (7)$$

under static, permanent loading.

Then, consider the building is subjected to wind pressure; the wind pressure coefficient $C = 1.2$ and the wind pressure

distribution $q = 0.12x^{1/4}$ (t/m^2)

at the height of x meters are assumed, which may probably be an acceptable assumption for the purpose of approximate computation. Under these loading conditions, the overturning moment becomes

$$M_{\text{wind}} = \int_0^{3.5N} Cq \cdot B dx = 1.072BN^{9/4} \text{ t}\cdot\text{m} \quad (8)$$

and the maximum bearing pressure is represented by

$$\sigma_{\text{wind}} = (6/BL^2) \cdot M_{\text{wind}} \quad (9)$$

Obviously, the condition to cause no uplift is $\sigma_{\text{wind}} \leq \sigma_{\text{static}}$.

From these equations, the minimum side length required for preventing the initiation of uplift may approximately be expressed by

$$L_{\text{min}} = 3N^{5/8} \quad (10)$$

or as shown numerically

in Table 3. In this connection, it may not be useless to pay attention to the behavior of foundation after the uplift has once taken place. Now, consider a loading plate as shown in Fig.6(a). The plate is also assumed resting on linear springs, which represent

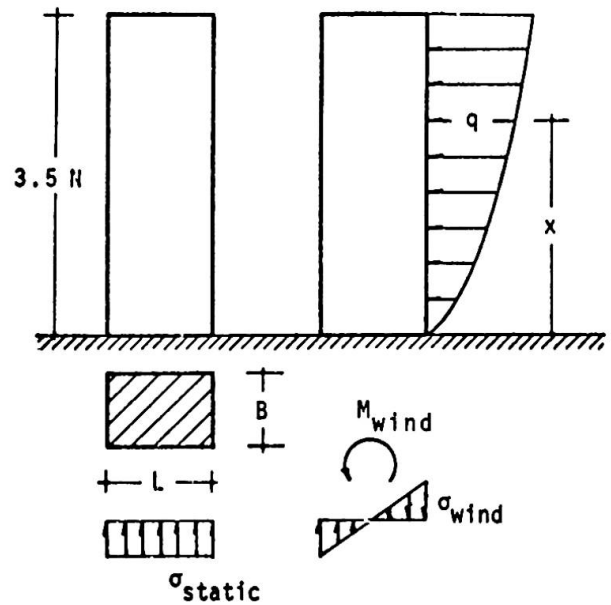


Fig.5

Table 3 : Minimum Side Length for Preventing Uplift

Total Number of Stories, N	20	40	60	90	120	200
Minimum Length, L, in Meters	20	30	40	50	60	80

the subgrade reaction of the supporting soil and can not develop any tensile reaction. Then, corresponding to the combined effect of P and M , three different stress patterns can be distinguished; (b) no uplift, (c) on the verge of uplift, and (d) partial uplift. Obviously, under the action of constant, vertical load, the stress state transfers from (b) to (d) through transition point (c) as the moment gradually increases. The relationship between the moment and the angle of rotation can be expressed by

$$\left. \begin{aligned} M^* &= \theta^* & \theta^* &\leq 1 \text{ for state (b)} \\ M^* &= 3 - 2/\sqrt{\theta^*} & \theta^* &\geq 1 \text{ for state (d)} \end{aligned} \right\} \quad (11)$$

where

$$M^* = M/(PL/6), \quad \theta^* = \theta/(2P/kL^2).$$

The rotational characteristics of the loading plate expressed by eq.(11) are shown in Fig.7. It will be seen in Fig.7 that the behavior of foundation develops non-linearity if the moment exceeds a limit

$$M_{\text{limit}} = PL/6 \quad (12)$$

which is a counterpart of eq.(10) for a tall building subjected to wind loading.

Such non-linearity should be referred to as geometrical non-linearity to distinguish it from the one developed by plastic properties of the soil itself.

No serious problem of uplift has been actually encountered so far in the design of tall buildings, since it can readily be overcome by extending the lower portion and setting back the upper portion of the building or by providing adequately deep basement. However, careful attention should be paid to such a building of urban location where little space is allowed between the building and the property lines.

The behavior of a basement in delivering shear force and overturning moment to the surrounding soil is another difficult problem to be dealt with analytically. A proposal (Ohsaki 1973) is presented on the basis of the theory of elastic halfspace, but it is primarily of theoretical interest and difficult to apply to design purposes. Three-dimensional finite element approach to the problem appears to be useful and, in fact, is utilized frequently in practical design. However, it requires considerable judgment and experience for selecting representative values of soil parameters which should be taken into the analyses and, in addition, the agreement between calculated and actual behaviors has not yet satisfactorily been verified.

4. SEISMIC FORCES

As has been pointed out previously, the major concern in designing the foundation of tall buildings lies in the effects of wind forces rather than seismic forces under the majority of situations; nevertheless, the dynamic effects of an earthquake must still be of great interest to the building engineers, since they might affect the design of not only the foundation but the overall structure to a considerable extent.

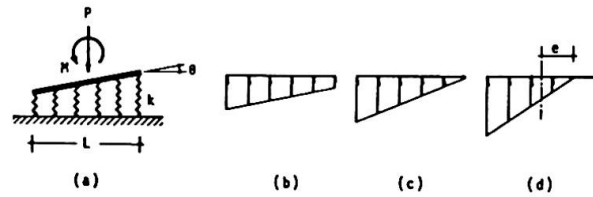


Fig.6

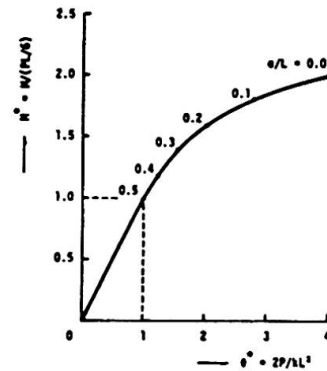


Fig.7

Usually, the influences of the foundation and the underlying soil deposit upon a building are discussed by dividing them into three categories:

(1) soil amplification, which implies that the stiffness and thickness of entire soil strata affect the motion at the surface of the soil,

(2) dynamic soil-structure interaction, which represents the combined effects on the ground motion of the presence of a building and the deformation and energy-dissipation characteristics of the soil immediately beneath the building, and

(3) resonance, which may take place between the building and the ground motion thus developed, resulting in high stresses and large distortions in the building.

The foundations of tall buildings are carried down through soft soils to a stiff soil stratum or rock from necessity for bearing heavy weight. First, this fact may likely minimize disadvantageous effects of soil amplification, whether the building is directly rested on the bearing stratum by the spread foundation or it is supported on the pier foundation. A number of reports are available indicating that the difference between the response spectra of the earthquake motions observed at the base of buildings and those observed at the level of the bearing strata is hardly noticeable with buildings supported on piers of high stiffness (Ohsaki 1969). A case where a tall building is still associated with the large earthquake motion as a result of soil amplification is that the support of long, flexible piles is involved. It is also reported frequently that the characteristics of response spectra of the earthquake motions at the base of pile-supported buildings exhibit a tendency to resemble those which would be observed at the ground surface of the same site (Ohsaki 1969). It is extremely probable that piles of large flexibility develop the same movement with the amplified motion of the surrounding soil deposit.

Secondly, for a building with the foundation carried down to a stiff bearing stratum, the effects of soil-structure interaction are of minor significance from the practical viewpoint. The interaction induces rocking and swaying motions to a building and, as a result, shifts the fundamental period of the building toward the longer side. Numerically, however, its effect on buildings up to 40 stories high is not likely to exceed 4 percent if the shear wave velocity for the underlying soil is approximately 500 meters/sec (Whitman 1972). Furthermore, the response spectrum of input acceleration has in general a downward slope in the range of fundamental periods of tall buildings and, consequently, interaction always acts beneficially to reduce the stresses in structural elements in a tall building.

Thus, the dynamic design of a tall building subjected to seismic forces is almost solely related to characteristics of the motion of the stiff bearing material itself, which have been considered to rarely involve harmful components to tall, flexible buildings.

In recent years, however, a new finding is being frequently pointed out that even the seismic motions of rock or rock-like hard stratum involve the wave components of extremely long periods, which might have considerable damage-potential to a tall building on account of the resonance. Fig.8 represents two examples of velocity spectra for such rock motions during earthquakes of considerably short, epicentral distance. This fact of long-period inclu-

sion is also observable in a large number of microtremor records obtained at the outcrops of rock or at the deep-seated hard strata, while the true character of such wave components has not yet been unmasked from the seismological standpoint. It might possibly be of

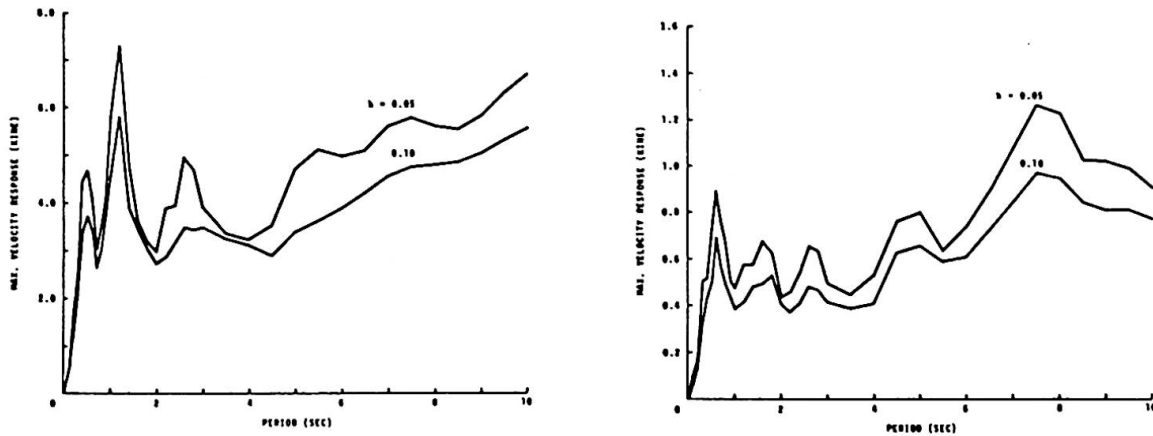


Fig.8

no little significance since, so far, such nature of input earthquake motions has seldom been taken into consideration as a basis of designing tall buildings.

5. CONCLUSION

A foundation in general must transmit any load of a structure eventually to the underlying soil or rock safely and without excessive settlement. Decision must always be made with this object in view for the foundation of a tall building as well.

However, because of characteristics peculiar to the structural systems and loading conditions of tall buildings, a number of problems are encountered which require special considerations.

Tall buildings are characterized, first of all, by their extremely heavy weights, and this fact frequently imposes rigorous problems on the design of foundations in various aspects. The average weight of tall buildings is estimated statistically to be in the range of 0.6 to 1.0 tons/sq.meter/floor.

Where a strong, stable stratum can be encountered within a reasonable depth from the ground surface, the ultimate bearing capacity and differential settlements usually give no critical consequence in spite of the heavy weight of tall buildings. To the differential settlements resulting from the large difference in distribution of loading intensity and the rebound of the bottom of excavation, attention should be paid however. Where the site is underlain by deep sediments of soft soils, the concept of floating foundation may hardly be applicable to a tall building, although it is quite effective for a building of lower height.

The foundation of a tall building is subjected to large lateral force and overturning moment during high winds. Behaviors of the foundation and the basement walls in transmitting these loads to the surrounding soils are considerably difficult to deal with analytically, being sometimes associated with another problem of uplift.

Seismic forces affect the design of foundations in a number of ways such as soil amplification, dynamic soil-structure interaction and resonance. If, however, the foundation is carried down through soft soils to a stiff bearing stratum, the disadvantageous effects

of amplification and interaction are of minor significance, except in the case where a long, flexible pile foundation is involved. A finding that even the seismic motions of rock involve the wave components of extremely long period may be of no little significance for the design of tall buildings, requiring further studies.

In this Introductory Report, presentations are mostly made in general terms and it is not intended to discuss any specific problem in detail. A few simple, numerical examples are presented, but they are only for illustrative purposes.

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SUMMARY

In this introductory Report, a number of problems related to the design of foundations of tall buildings are briefly summarized and discussed primarily from the standpoints of static weights, wind forces and seismic forces with a few illustrative, numerical examples.

RESUME

Ce rapport introductif présente un résumé sur les problèmes concernant le dimensionnement des fondations pour maisons hautes; ils sont examinés essentiellement en vue des charges statiques, du vent et des effets sismiques. Quelques exemples sont présentés.

ZUSAMMENFASSUNG

Der vorliegende Einführungsbericht behandelt eine Anzahl von Problemen, welche sich beim Entwurf von Hochhaus-Fundamenten stellen. Die Probleme werden kurz zusammengefasst und vor allem im Hinblick auf statischen Lasten, Windkräfte und Erdbebeneinwirkung an Zahlenbeispielen diskutiert.

Foundationen für weitgespannte Brücken

Foundation Structures for long span Bridges

Structures des fondations pour les ponts de grande portée

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In den Berichten über weitgespannte Brücken finden wir meist nur knappe Angaben über die Gründungen, obwohl gerade sie oft mehr Können und Wagemut der Ingenieure erfordern als die Überbauten. Die Ingenieure widmen sich offensichtlich dem sichtbaren Teil der weitgespannten Brücken weit mehr als den unsichtbaren Gründungen mit dem Ergebnis, daß die Überbauten einen höheren Reifegrad der Entwicklung sowohl in technischer als auch in wirtschaftlicher Hinsicht erreicht haben als die Gründungen. Die Arbeitskommissionen der IVBH beschlossen daher, in den nächsten Jahren den Gründungen mehr Aufmerksamkeit zu schenken. Es besteht kein Zweifel, daß aus Erfahrungen und Beobachtungen bei ausgeführten Gründungen manche Erkenntnis gewonnen werden kann, die dazu beitragen wird, künftig solche Gründungen einfacher und wirtschaftlicher zu bauen. Der Generalbericht zu diesem Thema hofft daher sehr, daß zum Tokio-Kongress 1976 wertvolle und nützliche Berichte über interessante Gründungen für große Brücken eingehen werden. - In diesem Vorbericht sollen nur einige Probleme angesprochen werden.

Mehr Baugrunduntersuchungen nötig

Die wichtigste Vorarbeit für Gründungen ist zweifellos die Baugrunduntersuchung, die in groben Zügen schon der Wahl der Trasse des Verkehrsweges vorausgehen muß, damit unnötige Schwierigkeiten umgangen werden. Neben einer gründlichen geologischen Analyse sind nach wie vor Bohrungen möglichst mit ungestört entnommenen Kernproben das beste Mittel, die Baugrundverhältnisse zu erkunden. Für schwer belastete Fundamente sollte man dabei weder an der Zahl noch an der Tiefe der Bohrungen sparen, auch wenn der Geologe einen gleichmäßigen Baugrund erwartet oder über die Art der tieferen Schichten sicher zu sein glaubt. Jeder erfahrene Brückenbauer weiß, daß der Baugrund immer wieder Überraschungen bietet, sei es im Fels oder in Sedimentschichten. Für jede größere Gründung sind daher innerhalb der geplanten Gründungsfläche mindestens 4 bis 6 Bohrungen bis in eine Tiefe durchzuführen, die etwa $1,5 \sqrt{F_g}$ (F_g = Gründungsfläche) entspricht. Die Kosten solch tiefer Bohrungen lohnen sich im Durchschnitt, weil nicht rechtzeitig erkannte Unregelmäßigkeiten im Baugrund während der Bauausführung stets zu äußerst unangenehmen Änderungen und Mehrkosten führen. Es gibt zahlreiche Beispiele in der Geschichte des Großbrückenbaues für solchen Kummer.

Trifft man Fels an, so ist der Brückeningenieur in der Regel glücklich, doch sollten gerade auch Schichtung, Schichtneigung, Klüftigkeit und dergleichen des anstehenden Felsens gründlich überprüft werden.

Dem Verfasser passierte es einmal bei dem Verankerungswiderlager einer Hängebrücke, daß der Fels zwischen zwei nur rund 20 m voneinander entfernten Bohrungen eine über 30 m tiefe Kluft aufwies, die anzeigte, daß der dem Fluß zu gelegene Felsteil sich schon talwärts geneigt hatte und daher zur Aufnahme großer Horizontalkräfte ungeeignet war. Es genügte nicht, den Spalt auszuheben, zu reinigen und mit Beton zu füllen, sondern die Verankerung mußte durch vorgespannte Bodenanker zusätzlich gesichert werden.

Felsgründungen

Wenn man gut gelagerten Fels antrifft, dann ist die Gründung in der Regel einfach. Die hohe Tragfähigkeit sollte jedoch in Zukunft mehr ausgenutzt werden, indem man Pressungen von 20 bis etwa 60 kg/cm² je nach Güte des Felsen zuläßt. Es ist heute auch nicht schwierig, Biegemomente hoher Brückens Pfeiler oder Verankerungskräfte von Hängebrücken oder Schrägkabelbrücken mit gebohrten, vorgespannten Felsankern aufzunehmen. Schon vor rund 15 Jahren hat ein Vergleich zwischen einer durch Betongewicht gesicherten Verankerung einer Hängebrücke und einer weitgehend aus gebohrten Felsankern bestehenden Lösung deutlich gezeigt, daß die Felsanker wesentlich billiger werden. Inzwischen wurde sowohl die Bohrtechnik als auch die Technik der Felsanker verbessert, so daß die Überlegenheit heute noch größer sein müßte. Dies gilt vor allem, wenn man gesundes Urgestein antrifft.

Caisson-Gründungen

Hat man angeschwemmten Boden, sei es Sand, Schluff, Mergel oder Ton, so sollte man die Fundamentgröße und damit die Bodenpressung hauptsächlich im Hinblick auf die für den Überbau erträglichen Setzungen wählen. Dabei spielen eigentlich nur die Setzungsdifferenzen oder zu Schrägstellung führende ungleiche Setzungen hauptsächlich bei statisch unbestimmten Hauptträgersystemen eine Rolle. Meist sind heute die Überbauten weitgespannter Brücken so schlank und gegen ungleiche Setzungen so unempfindlich, daß man beachtliche Setzungen ohne Nachteile in Kauf nehmen kann. Bei den heute weit verbreiteten Gummitopflagern, ob fest oder auf Teflon gleitend, kann man zudem ohne hohe Kosten den Überbau mit hydraulischen Pressen nachstellen, so daß Setzungen nachträglich ausgeglichen werden können. Die zulässige Bodenpressung nimmt mit der Gründungstiefe zu, weil die Grundbruchsicherheit durch die auflagernde Bodenschicht zunimmt. In der Regel ist es günstiger, Gründungen großer Brücken mit höherer Bodenpressung tiefer zu machen als die Lasten weiter oben mit niedriger Bodenpressung auf eine großflächige Fundamentplatte abzutragen, die dann große Biegemomente erleidet und entsprechend viel Stahl braucht. Es wäre erwünscht, daß gerade über diese Frage künftig mehr gearbeitet wird. Aus Probelastungen großflächiger Pfähle weiß man, daß der Spitzendruck auf sehr hohe Pressungen gesteigert werden kann, ohne daß Grundbruchgefahr besteht, und so kann man zweifellos auch bei Caissons mit größerem Durchmesser ziemlich hohe Bodenpressungen ausnützen, ohne damit das Maß der Setzung viel zu vergrößern oder die Sicherheit in unzulässiger Weise zu verringern. Natürlich gibt es hier Ausnahmefälle bei Böden

mit verhältnismäßig hohem Porenwassergehalt, wo die Setzungen dann über sehr lange Zeit anhalten und mehrmaliges Nachstellen oder dergleichen bedingen würden. In der Tendenz sollte man jedoch die höhere Tragfähigkeit der Böden in größeren Tiefen mehr als bisher ausnützen.

Tiefe Gründungen können einerseits mit Caissons, andererseits mit Pfählen ausgeführt werden. Caissons, die mit Druckluft abgesenkt werden, sind zwar sehr zuverlässig, aber wegen der erschwerten Arbeitsbedingungen wenig beliebt und auch nur bis ~ 30 m Tiefe geeignet. Die offenen Caissons können sehr tief gegründet werden und werden gern dort gewählt, wo die Gründung im Bett von Flüssen mit stark wechselndem Wasserstand, d.h. mit starken Hochwassern und hohen Strömungsgeschwindigkeiten, ausgeführt werden muß. In solchen Flüssen muß man mit erheblichen Verlagerungen der Flußsohle (Kolk, scour) rechnen, wobei in Flüssen wie Indus oder Ganges 20 bis 30 m tiefe Auskolkungen nicht ungewöhnlich sind. Sie bedingen Gründungstiefen von 40 bis 60 m, um die Pfeiler für Hochwasserkkräfte standfest zu machen. Für solche Verhältnisse ist zweifellos der kreisrunde Caisson aus Stahlbeton immer noch die beste Lösung, wobei die Wanddicke reichlich gewählt werden muß, damit der Caisson der Ausbaggerung folgend absinkt. Sinkhilfen mit tixotropem Betonit oder mit Injektionsspülung sind merkwürdigerweise in diesen Ländern noch nicht verbreitet. Der Kreiszyylinder ist günstig, weil die Bodenpressung vorwiegend Druck und nur bei ungleicher Verteilung geringfügig Biegemomente erzeugt, die in der Regel nicht gefährlich werden können. Eine schwache Ringbewehrung genügt, weil Biegemomente durch eine entsprechende Biegeverformung die Druckverteilung günstig beeinflusst. Kommt man mit einem Gründungszylinder nicht aus, so ist es in der Regel besser, z.B. zwei zylindrische Brunnen nebeneinander zu stellen und sie über dem Niedrigwasser kräftig miteinander zu verbinden, als zu rechteckigen Caissons überzugehen, die zur Aussteifung Zwischenwände brauchen und selbst dann noch verhältnismäßig viel Biegebewehrung erfordern (Bild 1).

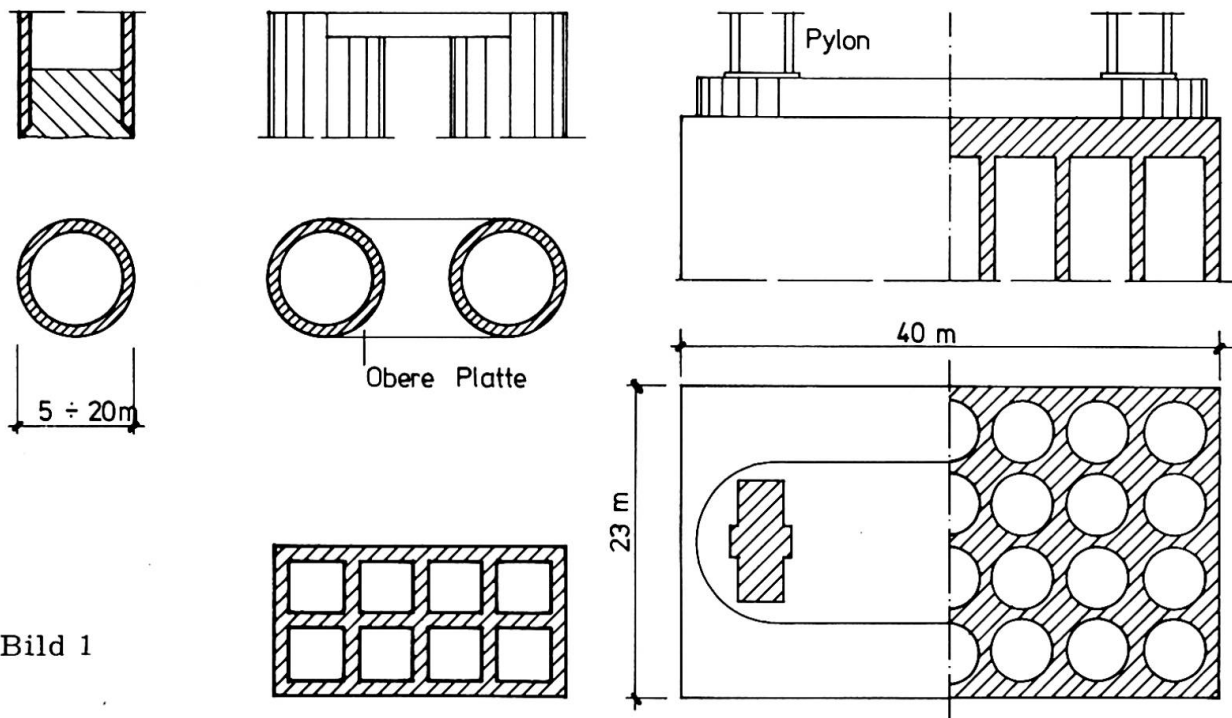


Bild 1

Die Caissons können bei Niedrigwasser auf Grund oder auf einer mit einer Spundwand geschützten Inselanschüttung hergestellt werden. Hat man die Gründung in schiffbarem tieferem Wasser auszuführen, so wird man heute den unteren Teil der Caissons in der Regel einschwimmen und zwischen wenigen Führungspfählen absenken oder - wie bei japanischen Brücken ausgeführt - mit großen Schwimmkranen genau am gewünschten Ort versetzen. Die japanischen Brückeningenieure haben für eine ihrer neuen Brücken mit einem Mammut-Schwimmkran von 2000 t Hubkraft einen vorgefertigten Caisson von rund 12 m Durchmesser und 30 m Länge versetzt - eine beachtliche Leistung.

Die Amerikaner verwendeten mehrfach Caissons, die aus einer ganzen Batterie von Kreiszyklindern aus Stahlblech zusammengesetzt waren, die zum Einschwimmen oben luftdicht verschlossen wurden. Zum Absinken auf den Flußgrund werden zunächst die Zwischenräume zwischen den Stahlzylindern soweit wie nötig ausbetoniert. Der Boden wird dann in den Stahlzylindern ausgebagert, bis der Caisson auf den tragfähigen Grund abgesenkt ist. Mit dieser Methode wurden die Pylonen der Tejo-Brücke Lissabon auf der Südseite bis auf 83 m unter dem Mittelwasserspiegel gegründet (Bild 1, rechts). Das Verfahren ist zwar sicher, aber wegen des hohen Stahlverbrauches für die meisten Länder zu teuer.

Pfahlgründungen

Wenn keine großen Kolk-tiefen und starken Strömungen zu berücksichtigen sind, dann beherrscht heute die Pfahlgründung selbst für sehr große Brückenlasten das Feld. Ein grosser Fortschritt war 1958 bis 1960 bei der Gründung der Brücke über den Maracaibo See in Venezuela erzielt worden, wo einerseits Ramm-pfähle mit Durchmessern bis zu 100 cm und Längen von über 50 m, und andererseits Bohr-pfähle mit Durchmessern von 135 cm und Längen bis über 60 m erfolgreich verwendet wurden. Bei Probelastungen wurden die Bohr-pfähle mit rund 2000 t belastet, ohne die Grenztragfähigkeit zu erreichen. Erstmalig wurde dort ein erhöhter Spitzendruck durch Zement-injektionen an der flachen Pfahls Spitze erzielt.

Wenige Jahre danach wurde bei der zweiten Brücke in Abidjan (Westafrika) ein Verfahren angewandt, um die Mantelreibung durch Injektionen unter Druck wesentlich zu vergrößern. Auch bei der größten Brücke Südamerikas von Rio de Janeiro nach Niteroi herrschen Pfahlgründungen vor. Bei den 1973 begonnenen beiden großen Schrägkabelbrücken über den Rio Parana bei Zarate-Brazo Largo wurden Bohrpfähle mit 2,0 m Durchmesser auf rund 70 m Tiefe unter dem Wasserspiegel in über 30 m tiefem Wasser gegründet (Bild 2), wobei die

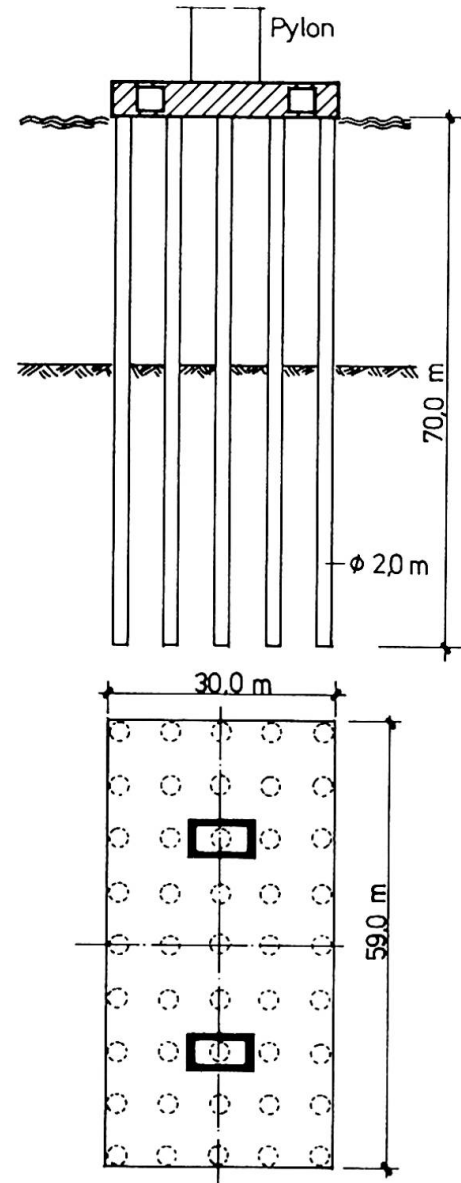


Bild 2: Pfahlgründung für Paraná-Brücken, Argentinien

Tragfähigkeit der Pfahlspitze wieder durch Zementinjektionen verbessert wurde. 35 Pfähle genügen so, um rund 36 000 t Last unter den Pylonen einer 330 m weit gespannten Schrägkabelbrücke für Eisenbahn und Straße zu tragen. Es besteht kein Zweifel, daß hier die Entwicklung noch nicht am Ende ist, sowohl die Durchmesser als auch die Längen der Bohrpfähle können weiter gesteigert werden.

In den dicken Kopfplatten, mit denen große Pfahlgruppen oben miteinander verbunden werden, treten je nach den Größenverhältnissen zwischen der Grundfläche der Brückenpfeiler und der Pfahlgruppe erhebliche Querkräfte und Biegemomente auf, die bei der Größe der Brückenlasten von vielen Tausend Mp sehr starke Bewehrungen bedingen. Man findet häufig, daß diese Bewehrungen aus sehr dicken Stäben ($\varnothing$ 40 bis 50 mm) in 4 bis 6 Lagen einfach übereinander angeordnet werden. Viele Ingenieure haben noch nicht erkannt, daß damit eine große Gefahr verbunden ist, weil die hohen Querkräfte extrem hohe Verbundspannungen zur Folge haben, die zum Spalten des Betons in der Ebene der Bewehrungslagen führen können. Diese Gefahr wird natürlich verstärkt, wenn mehrere Lagen dicht übereinander liegen. Solche Bewehrungen muß man in einem gegenseitigen Abstand von mindestens 4- bis 6fachem Stabdurchmesser verlegen, wobei die gesamte Bewehrung auf eine Höhe von 0,1 d bis 0,15 d verteilt werden sollte. Diese mehrlagigen Zuggurte müssen außerdem lotrecht verbügelt und auf die 1,2fache Breite der Pfahlköpfe beschränkt werden, wenn man Aufhängebewehrung zwischen den Pfählen vermeiden will. Man vergleiche hierzu die Berichte von W. Taylor über Versuche zu Pfahlkopfplatten der Lower Yarra Bridge, Melbourne, Australien, (vgl. CaCA-Technical Reports, London).

In der weiteren Entwicklung wird es zweifellos richtiger sein, diese großen Pfahlkopfplatten horizontal vorzuspannen, sie damit rissefrei zu halten und die Korrosionsgefahr zu bannen.

Gründungen in tiefem Wasser

In der Welt sind einige Großbrücken geplant, die Meeresstraßen mit verhältnismäßig tiefem Wasser überqueren und für die Brückenpfeiler in 60 bis 100 m tiefem Wasser gegründet werden müssen. In der Regel sind hierfür mit offenen Caissons hergestellte, massive Betonblöcke von riesenhaften Abmessungen vorgesehen, die einen erheblichen Aufwand erfordern und die Brücken

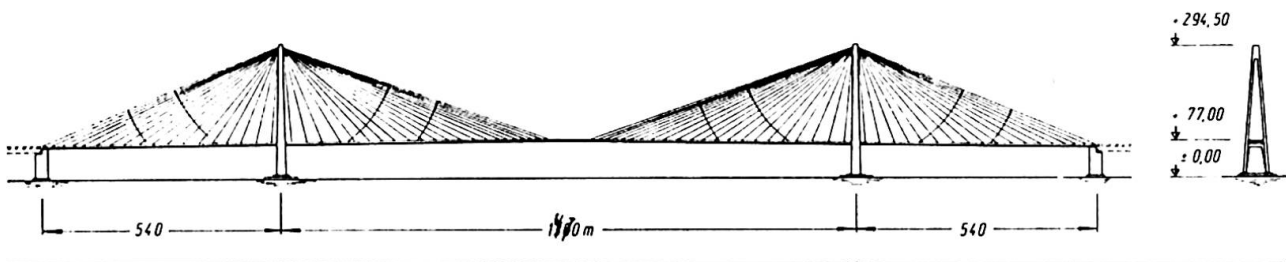


Bild 3: Entwurf Gruppo Lambertino für Brücke über die Straße von Messina, 1972

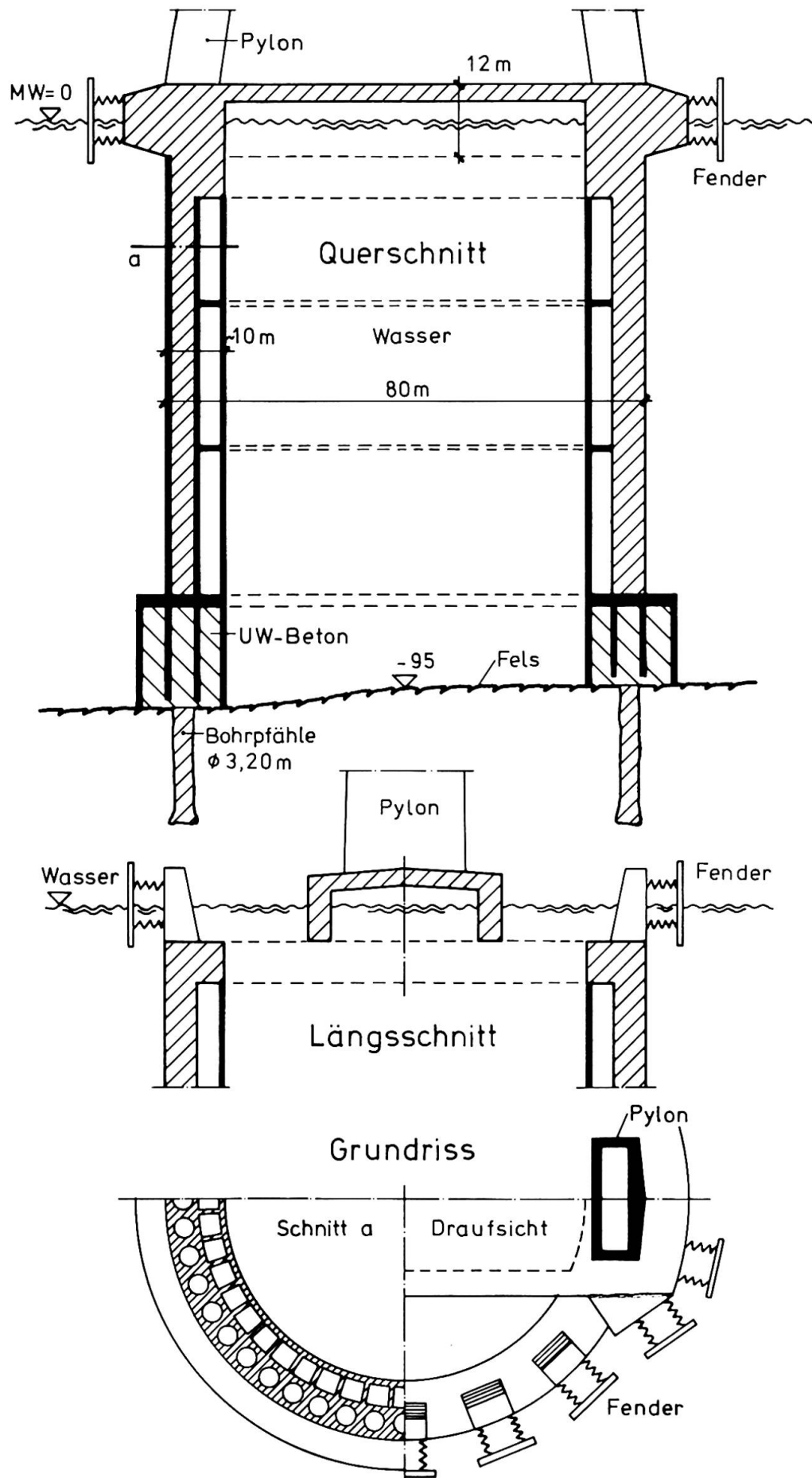


Bild 4: Vorschlag für die Gründung der Pylonen der Schrägkabelbrücke des Bildes 3

unnötig verteuern. Der Verfasser hat für einen Entwurf einer Brücke über die Straße von Messina als Schrägkabelbrücke von 1470 m Spannweite die Gründung der großen Pylone mit einem ringförmigen Zylinder von 80 m Durchmesser vorgeschlagen, in dessen Innenraum das Meereswasser verbleibt (Bild 3 und Bild 4) [1]. Der untere Ring wird dabei in einem Trockendock hergestellt, dessen Boden die Form der Felsoberfläche an der Gründungsstelle hat. Der Ring wird geflutet und in tieferes Wasser gebracht. Daraufhin wird die Zylinderwand mit Gleitschalung weitergebaut, wobei außen eine 5 m dicke, mit kreisförmigen Röhren durchsetzte Ringwand angeordnet ist, an die sich eine weitere rund 6 m dicke Zellenkonstruktion für den Auftrieb anschließt. Ähnlich wie beim Bau des großen Nordsee-Ölbehälters Ekofisk wird der Pfeiler schwimmend in genügend tiefem Wasser, jedoch in Ufernähe weitergebaut. Der fast fertige Pfeiler wird dann an seinen Bestimmungsort geschleppt, an versenkten schweren Ankerblöcken gegen die Tidenströmungen festgehalten und durch Fluten der inneren Ringkammern auf die gesäuberte Felsoberfläche abgesenkt. Der Fels besteht dort aus einem Konglomerat. Die Fuge zwischen dem Felsboden und den Wänden der großen Ringkammer kann mit vorweg eingesetzten, mit Stahlgitter bewehrten Gummischläuchen mit Druckwasser abgedichtet werden. Danach werden die Ringkammern über Zuleitungen ausbetoniert.

Durch die kreisförmigen Röhren in der äußeren Zylinderwand hindurch werden nun Bohrpfähle in den Fels vorgetrieben, deren Tiefe von der Felsqualität abhängig zu wählen ist. In diesen Bohrpfählen werden Spannglieder verankert, mit denen die vom Eigengewicht der Brücke her ohnehin schon große lotrechte Druckspannung im Zylinder noch vergrößert wird, um ihn gegen Erdbeben- bzw. Meeresbeben-Kräfte sicher zu machen. Vor diesem Vorspannen werden die Zylinderröhren natürlich ausbetoniert, so daß der äußere 5 m dicke Mantel des Zylinders massiv wird. Falls erforderlich, können auch die Zellen des unmittelbar anschliessenden inneren Ringes vollbetoniert werden.

Die Pylonenbeine der Brücke stehen unmittelbar auf dem Ring und werden durch einen 12 m hohen und rund 32 m breiten U-förmigen Riegel miteinander verbunden. Nur dieser Querriegel ragt aus dem Wasser heraus. An den Zylinderwandungen sind rundherum kräftige Fender an aus dem Wasser herausragenden Pfeilern befestigt, die einen eventuellen Schiffstoß mit großem Verformungsweg von etwa 3 bis 4 m abfangen.

Nach dem erfolgreichen Bau von Ekofisk in der rauen Nordsee, die bis zu 20 m Wellenhöhe aufweist, sollten keine Zweifel mehr daran bestehen, daß auch Brückenpfeiler solcher Abmessungen in Wassertiefen von 80 bis 100 m gegründet werden können.

Dieser Vorschlag für die Gründung eines Pfeilers in der Straße von Messina wurde hier nur als Anregung für solche Projekte beschrieben.

Zum Kongress werden zweifellos manche interessante Berichte über Gründungen weitgespannter Brücken vorliegen, zudem im gastgebenden Land Japan die zur Zeit größten Brücken der Welt gebaut werden.

- [1] F. Leonhardt und W. Zellner: Vergleiche zwischen Hängebrücken und Schrägkabelbrücken für Spannweiten über 600 m
IVBH-Abhandlungen Band 32-I, Zürich 1972

ZUSAMMENFASSUNG

Gründlichere Baugrunduntersuchungen und höhere Pressungen werden empfohlen. Grosse Caissongründungen sollten bevorzugt mit zylinderförmigen Caissons gebaut werden. Bei den Pfahlgründungen werden Bohrpfähle bereits bis 2,5 m Durchmesser und bis 70 m Tiefe eingesetzt. Einige Hinweise für die Bewehrungen von Pfahlkopffplatten werden gegeben. Für Pfahlgründungen in sehr tiefem Wasser, 60 bis 100 m, wird ein Vorschlag mit einem offenen Zylinder beschrieben, der für eine 1470 m weit gespannte Brücke über die Strasse von Messina entworfen wurde.

SUMMARY

More, better and deeper soil investigations and higher allowable soil pressures under large bridge foundations are recommended. Caissons should preferably be designed as circular cylinders. For pile foundations, drilled piles with diameters up to 2,5 m and depths of 70 m have been built. Some advice for pile cap reinforcement is given. For foundations in very deep water (60 to 100 m) a proposal with an open cylinder is described, as it was designed for a cable stayed bridge with 1470 m span across the Straits of Messina.

RESUME

Il est recommandé de prévoir de meilleures analyses des sols de fondation et de considérer des pressions admissibles supérieures pour les fondations des ponts de grande portée. Il est préférable d'envisager des caissons cylindriques. En ce qui concerne les fondations en pieux, il faut savoir que des pieux d'un diamètre de 2,50 m et d'une hauteur de 70 m ont déjà été réalisés. Quelques indications sont données pour la protection de la tête des pieux. Pour le cas de fondations en eau très profonde (60 à 100 m), la proposition d'un cylindre ouvert est faite, comme c'est le cas pour le projet de pont suspendu, de 1470 m de portée, sur le détroit de Messine.

V

Emploi des aciers à haute résistance et à protection naturelle pour les structures hautes ou à grande portée

Anwendung hochfester Stähle, inklusive wetterfester Stähle, für hohe und weitgespannte Tragwerke

Application of high-strength Steels including weathering Steels to high-rise and long-span Structures

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Structural Behaviour Including Hybrid Construction

Comportement sous charges en incluant les constructions hybrides

Tragverhalten, einschliesslich hybride Tragwerke

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1. INTRODUCTION

In tall buildings, short columns with stocky sections are generally used and sections of beams are also relatively stocky compared with those used in bridges. This selection were motivated from the structural consideration (strength and deformability requirements against large external forces), and from the functional requirements to keep the members as compact as possible. In such a situation, characteristic behaviors of structures using extra-high strength steels will mainly develop in their post yielding and post buckling region.

On the contrally, rather slender and thin-walled members are used in large span bridges. To use the higher strength steels advantageously in these structures, the effective slenderness of members and width-thickness ratio of plate elements must be kept as small as possible, as is obvious from the fact that all slender compression elements have the same Eulerian strength regardless of the yield strength of the material. The optimum combination of those two ratios in the design of a member is a matter of consideration. Residual stress and initial imperfection will affect the strength of those members with intermediate slenderness. The investigation of post buckling and thus ultimate strength state of plate and box girders including the consideration of fatigue effect is another important problem.

This report summarizes the status of knowledge on the topics above mentioned. The problem of crack formation which might occur during or after welding is also surveyed briefly though this topic would be dealt with more extensively in sub-theme Vc, "Fabrication and election problems" in this session. Though the assigned theme is the structural behavior of extra-high strength steels, it is considered to be better that the behavior of extra-high strength steels should be discussed in the comparison with those of mild steels and high strength low-alloy steels which are currently in use all over the world.

This report will, of course, not be complete since it is impossible to know of, and evaluate, all the research which has been conducted everywhere. The writers would, therefore, welcome any corrections and supplements as well as contributions based on new developments.

2. MATERIALS

A large number of steels are currently used for structural applications. These steels may be grouped into three or four general classifications: carbon

steels(A), high strength low-alloy steels(B), heat treated high strength carbon steels and heat-treated constructional alloy steels(C). Heat treated high-strength carbon steels, introduced in 1964, are carbon steels that have been heat-treated by quenching and tempering to obtain high strength and toughness and are currently produced as plates. A minimum yield stress 552 MN/m² in thicknesses of 19 mm and less, and 483 MN/m² in thicknesses over 19 to 38 mm is available, and thus it will fill the need for a constructional steel intermediate between B class steels and C class steels above cited. Steels in this category are, however, not yet covered by specifications. All the steels discussed are weldable with no loss of strength, but the welding materials and procedures must be in accordance with approved methods. Typical stress-strain curves for those structural steels are shown in Fig.1. A knowledge of the stress-strain relations take on during the elastic and plastic ranges of behavior is an essential requisite to structural analysis. In elastic range there are accepted average values of the modulus of elasticity E , while the characteristic values in plastic range are not so firmly recognized, though they are essential to calculation of inelastic strength and deformation of members and frames. Properties which characterize the plastic range are:

σ_y = yield stress level, ϵ_{st} = strain at initial strain hardening, $E_{st} = (d\sigma/d\epsilon)_{\epsilon_{st}}$ = strain-hardening modulus, and $Y = \sigma_y/\sigma_u$ = yield ratio of material (σ_u = tensile strength), which sometimes can be used as an alternative of E_{st} and ϵ_{st} for large strain range. Values of E_{st} and ϵ_{st} are plotted in Figs.2 and 3 respectively, which were collected from available reports. They are including SS41, G40.8, A36 for carbon steels(A), SM50, SM53, G40.11, Fe52, A441, A572 for high strength low-alloy steels(B) and SM58, HW80, A514, (CT100) for heat treated constructional alloy steels(C).

Specified mechanical properties of various grades of high strength structural steel through various countries are tabulated in APPENDIX[1,2], which will offer a convenience to the reader who may be unfamiliar with a designation used in this report and wishes to identify it with grades with which he is familiar. Typical mild steels are included for reference and which are identified by # mark. More detailed information of those groups of steel is given in Refs.3 and 4.

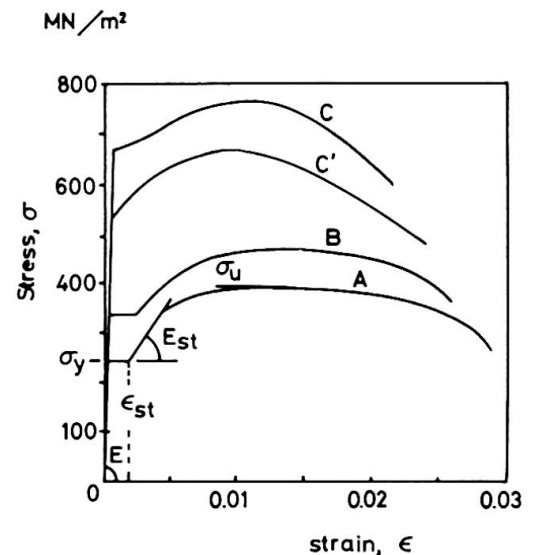


Fig.1 Typical Stress-Strain Curves for Structural Steels

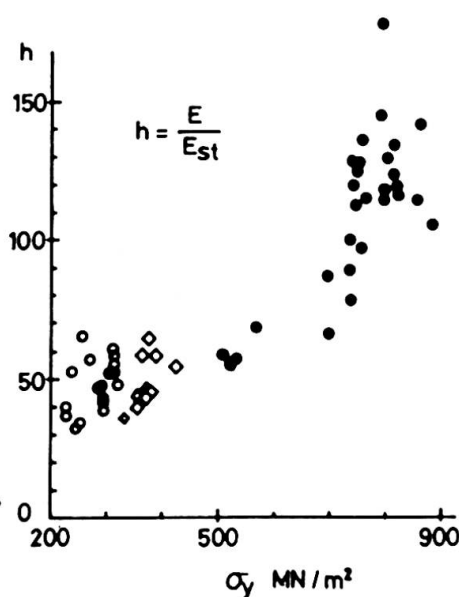


Fig.2 E_{st} vs. σ_y Relation

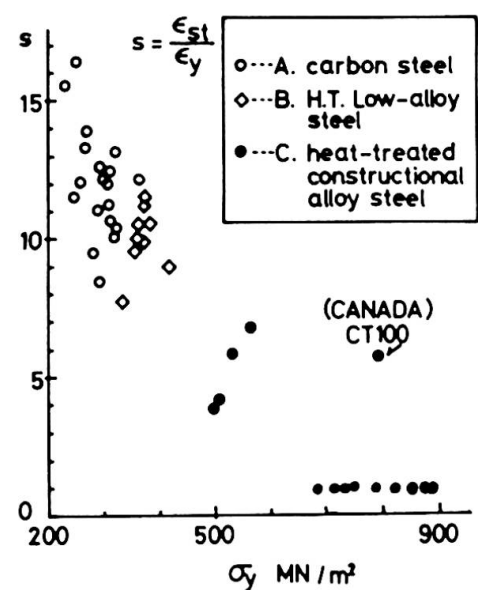


Fig.3 ϵ_{st} vs. σ_y Relation

3. TENSION MEMBERS

Although tension members are commonly designed on an elastic basis, it is nevertheless considered desirable that as this level is exceeded and the structure proceeds towards its ultimate load, capacity for distortion should still exist in the member. It is the recognized design philosophy that structures in strong seismic area should have enough plastic deformation capacity in horizontal direction so as to be able to absorb the input of earthquake energy. In Tokachioki Earthquake (1968, Japan), diagonal bracings of many steel structures had been broken off at their rivetted or bolted holes or at their threaded parts of end connections without substantial plastic elongation of the members as a whole.

In Ref. 5, deformation capacity of tension plates with variable cross section was studied theoretically and experimentally. Consider a model shown in Fig. 4(a), cross section of which varies continuously along the length and has the minimum cross sectional area A_0 at 0. The yield will occur at 0 when the tensile force equals $A_0 \sigma_y$, but no plastic deformation can be observed at this state as yield line at 0 y can not spread.

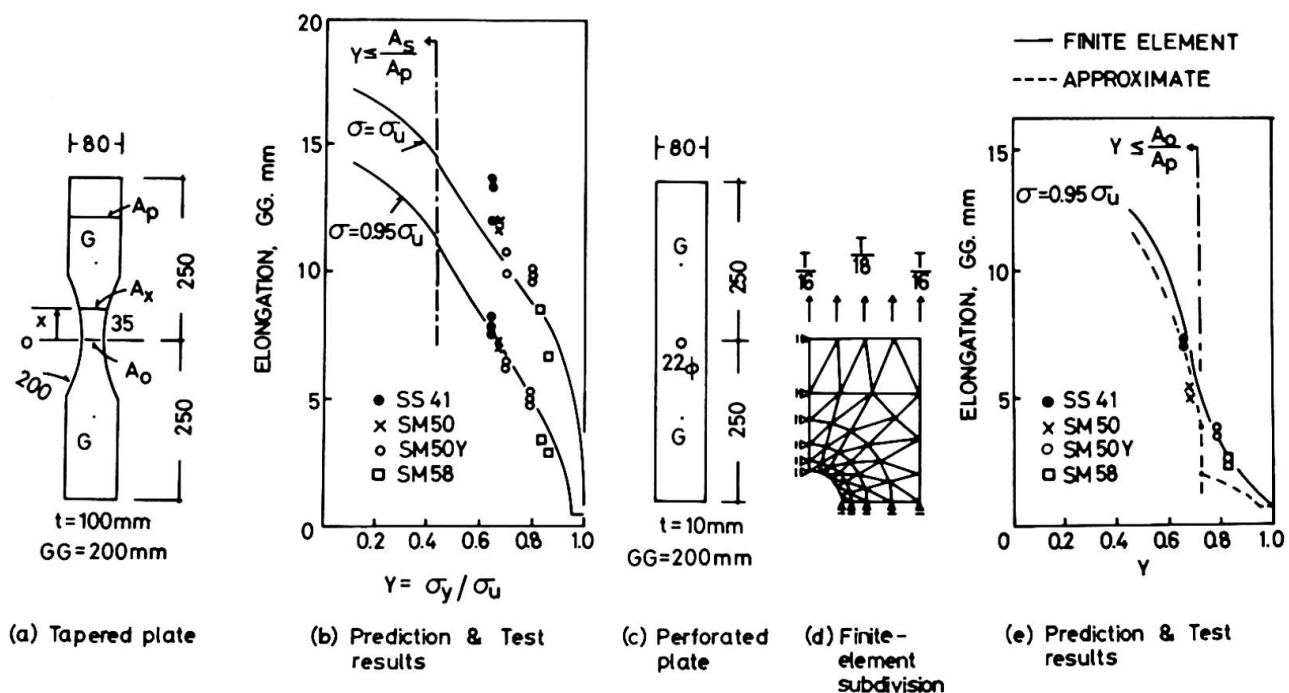


Fig.4 Deformability of Tension Plates of Variable Section

The spread of plastic region along the length is only possible when strain-hardening takes place at 0 corresponding to the increase of tension. The maximum spread of the plastic region x is determined by the condition that $A_0 \sigma_u = A_x \sigma_y$ or $A_0 = Y A_x$, where σ_u is the tensile strength of the material, $Y = \sigma_y / \sigma_u$ = yield ratio of material and A_x is the sectional area at x . As an extreme case, if the material be elastic-perfectly plastic, namely $Y=1$, no spread of plastic region could be expected and thus the bar would break off in a brittle manner as soon as the stress at minimum cross section reaches yield point. In Fig. 4(b), test results are compared with the prediction obtained from the above simple analysis. When the change of sectional area along the length is steep as in cases of bolt or rivet hole, the effect of stress concentration can not be ignored. This case was analysed by means of finite element technique allowing for the elastic-plastic-strain hardening relationship of the material for the model (c) in Fig. 4. Theoretical prediction and test results are compared in Fig. 4(e). For both cases, correlation between test results and

theoretical prediction seems to be satisfactory. From this study, it can be said that if a tension member system should have enough deformability, the following condition should be satisfied at its connections,

$$A_n \geq Y A_g \quad (1)$$

where, A_n = net cross sectional area at bolted, riveted or threaded part
 A_g = gross cross sectional area of a tension member

Similar tests were carried out using A514 steel plates and arrived at the same conclusion[6]. Tension tests of large, bolted butt splices of A514 steel fastened by A490 bolts had also confirmed above conclusion[7], and pointed out that "A514 steel joints using A490 bolts do not produce yielding on the gross section if the elements of the joint are designed according to current AISC specification (1967)".

4. BEAMS AND BEAM-COLUMNS, -IN PLANE BEHAVIOR-

In-plane behaviors of beams and beam-columns will be discussed, i.e. it is assumed that local buckling and lateral-torsional buckling are precluded in the present discussion. Those problems will be surveyed later herein. As far as elastic behavior is concerned, essentially there would be no difference among different grades of steels since they have common modulus of elasticity. On the contrary, plastic behavior of those members will depend largely on the grade of steel.

Moment-axial thrust-curvature diagram of a wide flange section made of carbon steel (A) and of heat-treated constructional alloy steel (C) are shown in Fig. 5 [8,9]. Assumed stress-strain relationship of these two steels are depicted in the figure. Though a typical wide-flange shape was chosen here, it is known that the moment-curvature diagrams are almost identical for all practical wide-flange shapes. Remarkable difference can be seen between these two diagrams and this difference will reflect on the load-deformation curves of members since the latter is obtained by integration of the former over the length.

4.1 Beams

It is widely recognized that the load-deflexion curve of a beam subjected to uniform moment shows the similar pattern to that of stress-strain diagram of the material used since yielding is spreaded over the whole uniformly bent segment. Thus beams of A and B class steels show the flat plateaus after yielding which correspond to the yielding flow portions of stress-strain curves of their steels. Increase of the moment due to strain-hardening will be observed only after the large deformation had taken place which is immaterial from the structural view point. In beams of C class steels, the increase of moment due to strain-hardening will occur immediately after attaining full plastic moment because these steels have almost no yield plateau (see Fig. 5), but as the strain-hardening modulus of these steels are very small, this upgrade slope is negligible, and thus the plastic behavior of beams under uniform moment is almost the same for all grades of steels and enough ductility can be expected.

In contrast, yielding in the beam under moment gradient can not spread unless the moment is increased, and so strain-hardening sets in as soon as the full plastic moment is reached, resulting in the upward swing of the curve. This behavior is illustrated by a schematic picture in Fig. 6. The maximum load is approximately defined by M_p/Y , and hence the maximum increment of the moment of C class steels, Y of which is very high, is very small. Behavior of A class steels is shown in solid line and that of C class steels is shown in dashed line in the figure.

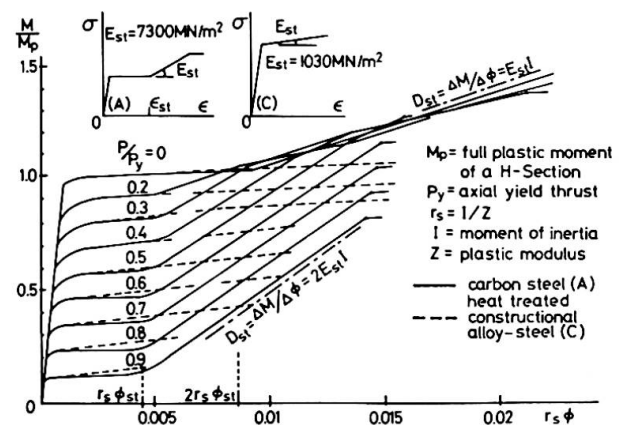


Fig. 5 Moment-Thrust-Curvature Diagram

Although this difference will influence the sequence of hinge formation, rigidity and capacity of energy absorption of a structure in the plastic region when these beams are framed in, the more important is the problem of ductility. In Ref.10, it was reported that tension flanges of two A514 steel beams built up by welding, had ruptured at the vicinity of the concentrated load. These beams were simply supported and had a concentrated vertical load at mid-span as shown in Fig.7. Both beams developed full plastic moment in the vicinity of the concentrated load, and in the course of sustaining continuously increasing load abrupt unloading occurred due to a rupture of the tension flange. In one beam, the rupture proceeded straight across the flange and vertically through the web and the weld of the stiffener. In another beam, the rupture extended diagonally across the flange and through the base metal of the web. Both ruptures stopped before reaching the compression flange. Yield stress and tensile strength of tension flanges of both beams were 766 and 849 MN/m² respectively and thus the yield ratio was 0.902.

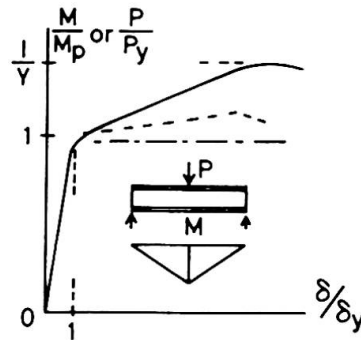


Fig.6 Lord-Deflexion Curve

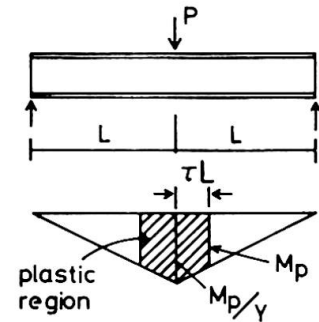


Fig.7 Spread of Plastic Region

This unusual behavior could be explained as follows: In Fig.7, when the maximum moment M_p/Y is attained at the concentrated load, the spread of the yielding of tension flange along the length of the beam τL is determined from the bending moment diagram as $\tau = 1 - Y$. Note that τ is only the function of the yield ratio of the material and independent of the span of the beam and the moment gradient. In the present case, $Y = 0.902$ and τ is calculated to be 0.098 which means that only one-tenth of the span can be yielded even at the ultimate strength state, and then tension flange will break off without developing enough rotation of the beam.

In general, rotation capacity of a beam had been governed by the local buckling of the compression flange and/or lateral-torsional buckling. But for beams of extra-high strength steel with high yield ratio, deformation capacity of tension flange might become another criterion of the rotation capacity as seen above.

4.2. Beam-Columns

When a beam-column is subjected to end moments under constant axial force, the moment will reach a peak and thereafter unloading will take place, and frequently it will not be possible to achieve full plastic moment M_{pc} . This reduction in both bending and deformation capacity differentiates the performance of a beam-column from that of a beam. The reduction in moment capacity is due to the combined effect of the "secondary" moment introduced by the axial force times the deflexion and the reduction of stiffness due to yielding.

Moment-end rotation ($M-\theta$) or moment-deflexion ($M-\delta$) relationships can be obtained by integrating the moment-thrust-curvature relationship as was given in Fig.5. Though the determination of $M-\theta$ or $M-\delta$ curve is performed by numerical integration because of the complicated nature of cross sections, the numerical integration procedure is very simple in concept. The important point is the use of realistic stress-strain relationship of the material and thus the use of realistic moment-curvature relationship of the member section to assess the behavior of beam-columns made of different grades of steel.

The most recent research on this topic has been summarized in Refs.11 and 12. Among the many available solutions, that given in Ref.13 is referred here, because it represents so-called "exact" solution including the influence of residual stress on column strength, and is directly applicable to columns fabricated from as rolled wide-flange shapes which are subjected to bending about their strong axis.

In the development of the theory, it is assumed that, a) stress-strain relationship is elastic-perfectly plastic (no account for strain-hardening), b) the residual stress pattern used is typical for rolled columns of A36 steel and the maximum compressive residual stress is $0.3\sigma_y$ (there are some evidences to show that the magnitude of the residual stress is independent of the yield stress of the material which will be discussed later). Tests carried out on beam-columns of A class steels (A7, A36, A37, SS41) [14, 15, 16, 17] and of B class steels (A441, SM50) [18, 19, 20] had confirmed this solution well. Based on that solution, tables for the ultimate strength of eccentrically loaded beam-columns have been furnished in Ref. 21. Although the tables were furnished for steels with a yield stress of 33 ksi. (228 MN/m²), it is suggested the tables can be applied to steels of other yield points by substituting a modified slenderness L/r :

$$(L/r)_{\text{mod.}} = L/r \sqrt{\sigma_y/33}$$

In some design situations, particularly when resistance to earthquake motions and blast shocks is involved, it is necessary to count on the rotation capacity or deformability of beam-columns. In Ref. 22, rotation capacity of beam-columns is surveyed. Available test results show that the rotation capacities are relatively small when compared to beams though they were for relatively long beam-columns under heavy axial load. Rotation capacities of the order of 4 to 13 were reported on tests of short beam-columns with one end moment [23]. Though above informations of rotation capacities are limited on beam-columns made of mild steels, AISI design manual on "Plastic Design of Braced Multistory Steel Frames" allow the higher strength steels to use the $M-\theta$ curves furnished for A36 steel by modifying the slenderness and rotation by the following equations:

$$(L/r)_{\text{equiv.}} = (L/r)_{\text{actual}} \cdot \sqrt{\sigma_y/36} \quad (2)$$

$$\theta = \theta_{\text{chart}} \cdot \sqrt{\sigma_y/36} \quad (3)$$

where σ_y is the yield stress of the particular steel expressed in ksi. [24].

The method of modifying the slenderness and the end rotation by the square root of the ratio of yield stresses as was proposed in eqs. 2 and 3, and in Ref. 21 seems to be reasonable as far as the elastic-perfectly plastic relationship of the material is assumed. When beam-columns become shorter, material properties in plastic region such as E_{st} , ϵ_{st} and Y will play the more important role, while the effect of the secondary moment will become less significant, and thus the ignorance of the strain-hardening will cause the errors of conservative side. And if the $M-\theta$ charts are constructed on the basis of the realistic $\sigma-\epsilon$ relationship of a particular grade of steel, the adoption of the cited method of modification is only possible when the $\sigma-\epsilon$ relationship of the other grade of steel which is intended to apply is exactly similar to that used in making the charts.

Considering the substantial difference of $\sigma-\epsilon$ relationship which exists between mild steels and extra-high strength steels, the adoption of that modification method to short columns of extra-high strength steels would produce a poor prediction.

Cantilever beam-columns subject to horizontal shear force Q under constant axial force P at the top of them were analysed for two different grades of steels based on the realistic $\sigma-\epsilon$ relationships of respective steels, and the obtained interaction curves are shown in Fig. 8 [25]. Mechanical properties of the steels in plastic region are shown in the figure. Yield stress of SM58, though it belongs to C class steel, is not so high and E_{st} is not so low, but interaction curves differ remarkably from those of mild steel (SM41). Consider a curve for $L/r=13.6$ of SM41 in Fig. 8(a). When the steel is replaced by SM58, the equivalent slenderness is calculated as $(L/r)_{\text{equiv.}} = 13.6 \sqrt{499/262} = 18.8$ according to eq. 2, and the curve for $L/r=18.6$ in the figure can be considered as the approximate equivalent curve to be applied. This curve shows the remarkable difference from the actual interaction curve for SM58 with $L/r=13.6$ as is shown in Fig. 8(b).

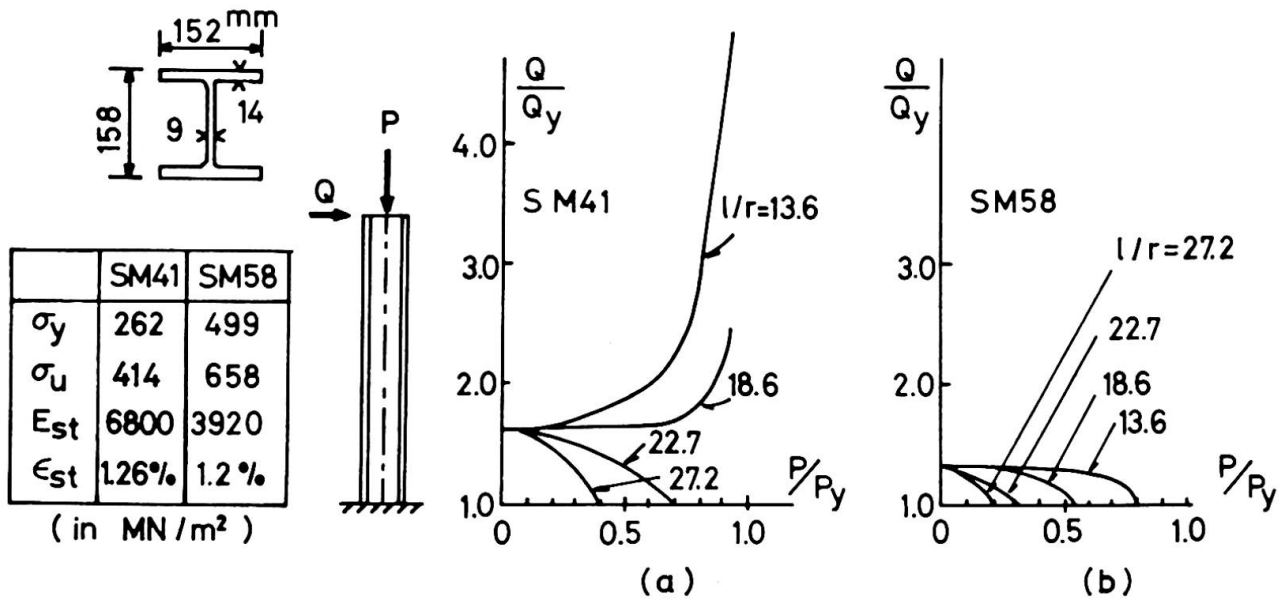


Fig.8 P-Q Interaction of Beam-Columns

5. BUCKLING

The previous discussion concerned the in-plane behavior of beams and beam-columns. Insofar as in-plane behavior is concerned, deformability of beams are unlimited unless flanges of them are broken off, and beam-columns, though they have peak moments due to the combined effect of the "secondary" moment and the yielding of material, often exhibit a rather slow falling-off in the descending part of their $M-\theta$ curves. This optimal behavior is curtailed by the occurrence of lateral-torsional buckling and local buckling.

Flexural buckling is the only phenomenon when columns are compressed centrally which will scarcely occur in practice. Nevertheless, this concept forms a generally accepted basis for column strength and design.

Research on the buckling problems in the past decade was concentrated to those of members with intermediate range between the small and large slenderness wherein secondary factors, such as residual stress, initial crookedness and eccentricity have the greatest effect on the buckling strength of members. A comprehensive review of the research on these problems carried out by 1966 is given in Ref.11, and theoretical predictions, experimental results and design formulae reported by the end of 1969 are summarized in Ref.26. Main findings obtained from those researches which are pertinent to the high-strength steels are:

Columns

- 1) As for thermal residual stress introduced in rolled shapes, the magnitude of the compressive residual stress appears to be independent of yield stress level [19,27,28,29]. This gives a favourable situation to high-strength steel columns.
- 2) For welded built-up H-shapes (irrespective of grades of steel) the maximum tensile residual stress closely approaches the yield stress and the adverse effects of the concurrent compressive residual stresses are large [30].
- 3) The ultimate strength of A514 steel columns of circular cross section with initial curvature and with both concentric and anti-symmetric residual stresses, the former of which is introduced by cooling process and the latter is introduced by cold-straightening of columns, was investigated theoretically and experimentally [31,32,33,34]. This research verified the proposed numerical procedures to determine the ultimate strength of columns under these conditions.

Based on these researches, a systematic evaluation of the effect of residual

stress and initial curvature on the column strength of wide-flange sections has been made[35]. Residual stress was held at a constant level of 69 MN/m². It has been shown that the maximum effect of either residual stress or initial crookedness, alone or in combination, always occurs at the slenderness which yields the Euler critical stress equals σ_y . The effect of residual stress is less pronounced for higher strength steels than it is for mild steel, thus as the yield stress increases, initial curvature and eccentricity take on increasing importance in relation to residual stress.

Beams and beam-columns

Recent investigations on lateral-torsional buckling of beams and girders have been reviewed in Ref.36. The strength of a laterally unsupported beam of relatively short length, like that of a corresponding column, will be determined by inelastic rather than by elastic behavior.

An approximate method of estimating the effect of plastic action on the buckling strength of beams and girders is to assume that the relationship between elastic and inelastic buckling strength is the same for beams as it is for columns. The inelastic buckling strength of beams can then be estimated from a column curve. This approach was evaluated by comparison with a theoretical solution[37], and also was confirmed by experiments[38].

A review of solutions of lateral-torsional buckling of beam-columns in inelastic range is given in Ref.39. Though few analytical solutions of this problem are available[40,41], the nonlinearity of σ - ϵ relationship makes it extremely difficult to express explicitly the variation of the several quantities that must be evaluated. Because of this difficulty, several approximate types of solutions are generally adopted. Among those approximate solutions, the following modified interaction formula[42] is used in specifications most widely.

$$\frac{P}{P_u} + \frac{M_o}{M_u(1-P/P_e)} \leq 1 \quad (4)$$

where,

P =applied axial load

P_u =axial load producing failure in the absence of bending moment(including the possibilities of buckling in the weak plane)

P_e =elastic critical load for buckling in the strong plane

M_o =maximum applied moment, not including the "secondary" moment.

M_u =bending moment producing failure in the absence of axial load(including the possibilities of lateral buckling)

Eq.4 was confirmed by an experiment[14].

More recent researches on buckling of members made of high-strength steels carried out after 1966, i.e. after the edition of Ref.11 are briefly reviewed below: Column tests of built up box-sections made of HW80 steel loaded centrally and eccentrically were carried out[43], and confirmed the theoretical estimations of Ref.35 cited previously. A series of research on British new high-strength steel Grade 55 were carried out in order to formulate design rules. This steel is denoted Pearlite-Reduced Structural Steel and minimum yield stress is 447 MN/m². 130 strut tests(concentric loading) were carried out as one project[44], and demonstrated that the test results closely related to the prediction of Perry-Robertson formula, and thus showed that the BS449 formula could be safely extrapolated to this steel. As another project, 30 tests were carried out on beams of universal I-sections under a uniform bending, including the measurements of thermal residual stresses[29]. It was shown that the bending stresses for the design of beams in Grade 55 steel could be determined by the method given in BS153(1958) and BS449(1959) with some amendments.

A simple method of design of laterally unsupported beams which covers all grades of steels was presented and this prediction was compared with test results given in Ref.29 above to show a good agreement[45].

Compression tests of square columns built-up by welding were carried out to

determine the buckling strength of high-strength steel plate elements. Steels used were A514 in one test[46], and HW80 in another test[47]. The following conclusions were obtained in both tests:

- 1) Considerable post-buckling strength exists in a plate buckled in the elastic range, while a plate buckled in the elastic-plastic range has a relatively small reserve of post-buckling strength.
- 2) The effect of residual stresses on the buckling strength of a plate is less pronounced for these C class steels than it is for A class steels.
- 3) The plate elements (with intermediate width-thickness ratios) of these C class steels are stronger than those of A class steels when compared on a nondimensional basis.

The foregoing concerned the buckling of members of intermediate range. Ultimate strength and deformability of members in extreme ranges which subject to local and/or lateral buckling are another current topics: The research of the ultimate strength of thin-walled structures, in one extreme, is very important in the design of plate and box girders, and this topic, including that of hybrid girders, will be dealt with separately in another section later herein. The deformability and ultimate strength of members with very small slenderness or width-thickness ratio, in the other extreme, is a topic of increasing interest in relation to the development of the plastic design and of the earthquake resistant design wherein the assessment of the energy absorption capacity of members and frames is most important. Because the behavior is determined almost entirely by the plastic properties of the material in such a short and stocky members, characteristic behavior of extra-high strength steels will be paramount in this range, and this problem will be surveyed hereafter with a special emphasis layed on the deformability or ductility.

5.1 Local Buckling

In plastic analysis it is tacitly assumed that the moment capacity of the member will remain at the level of the plastic moment until enough hinges have developed to form a mechanism. It is, therefore, necessary that the moment capacity not be impaired by local or lateral-torsional buckling until the required rotation has been achieved. In earthquake resistant design, this requirement is more direct to secure the energy absorption capacity.

The first solution to the problem of local buckling in the strain-hardening range was given in Refs. 48, 49, the applicability of which was limited to A class steels (A36).

Two solutions are referred here which are capable to cover the higher strength steels:

- 1) In Ref. 50, the limitations of width-thickness ratios of wide-flange sections are given as;

For flanges subject to uniform compression along the length,

$$\frac{b}{t} = \sqrt{\frac{G_{st}}{\sigma_y}} + 0.381 \left(\frac{E_{st}}{\sigma_y} \right) \left(\frac{t_w^2}{t} \right) \left(\frac{2bt}{dt_w} \right)^{1/2} \quad (5)$$

$$G_{st} = \frac{2G}{1 + \frac{E}{4E_{st}(1+\nu)}}$$

where, b = one-half of flange width
 t = thickness of flange
 t_w = thickness of web
 d = depth of web

G =shear modulus

In the case of beams under moment gradient,

$$\frac{b}{t} = \frac{3.56}{\sqrt{\epsilon_y(3 + 1/Y)(1 + h/5.2)}} \quad (6)$$

where, $h=E/E_{st}$

The problem is treated as a classical buckling with bifurcation of the equilibrium position under assumptions that local buckling will occur when a) the average strains in the plate are at the strain-hardening strain ϵ_{st} , b) a long enough portion of the plate has yielded so that at least one-half (for uniform compression) or one full (for moment gradient) wave length of the buckle can develop, and the strain-hardening modulus in shear G_{st} is evaluated by assuming the discontinuous yield process.

2) In Ref.51, the deformation which a plate can develop without reducing its yield strength, expressed in terms of average compressive strain ϵ , is given as a function of width-thickness ratio and material property as;

For flanges subject to uniform compression along the length,

$$\frac{b}{t} = \left(\frac{2\phi_1 + \phi_2}{2\sqrt{2}} \right) \frac{1}{\sqrt{\epsilon}} \quad (7)$$

where, $\phi_1 = \frac{4(\alpha^2 - 1) + (\beta_1 - 0.5)}{2\beta_1 + 1}$, $\phi_2 = \frac{\alpha^2 - 1}{\alpha}$, $\beta_1 = \sqrt{0.25 + 4(\alpha^2 - 1)}$, $\alpha = 1/Y$

For webs subject to uniform compression along the length (unloaded edges are clamped),

$$\frac{d}{t_w} = \sqrt{2}(\phi_1 + 0.5\phi_3) \frac{1}{\sqrt{\epsilon}} \quad (8)$$

where, $\phi_3 = \frac{2(\alpha^2 - 1) - (\beta_2 - 1)}{\beta_2 - 1}$, $\beta_2 = \sqrt{1 + 4(\alpha^2 - 1)}$

Eqs.7 & 8 are upper bound solutions obtained by assuming a collapse mechanism and then applying the work theorem to it.

Available test results are plotted in Fig.9 for plates supported at one unloaded edge and free at the other. Those test results were reported in Refs.52,53 for C class steels, in Refs.51,54,55 for B class steels and in Refs.48,51,53,54,55 for A class steels respectively. These plots are compared with the prediction of eq.7 in the figure. Considering the variety of the degree of restraint existing among those test specimens, eq.7 seems to explain the general tendency well, though it is too conservative for C class steels.

It should be noted that the test results on C class steels are showing lower $\epsilon_{max}/\epsilon_y$ values than those of A and B class even though compared on a nondimensional slenderness basis. This fact seems to show that the axial deformability of flanges is not only a function of the square root of the inverse of the yield stress, but also a function of the material properties in plastic range such as E_{st} , ϵ_{st} and Y . This is the similar situation to that of in-plane behavior of beam-columns as was discussed in section 4.2 earlier.

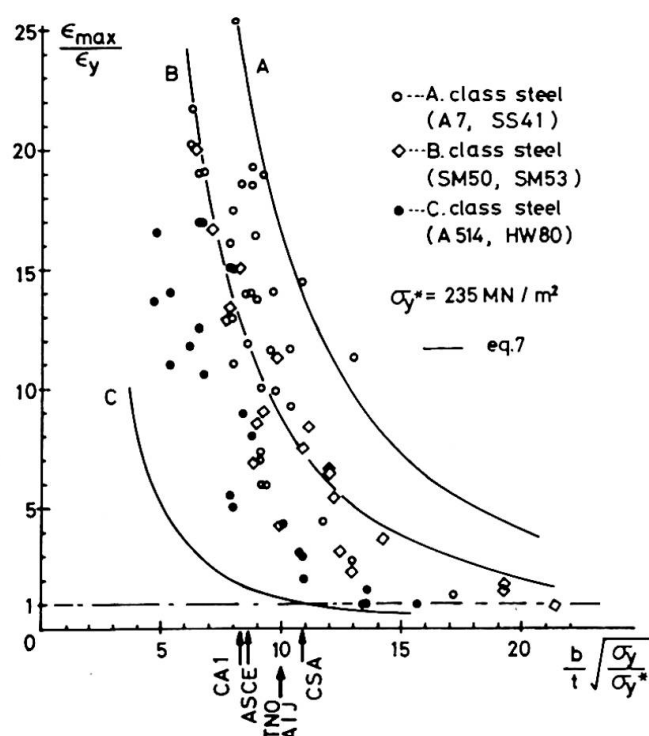


Fig.9 Deformability of Flanges

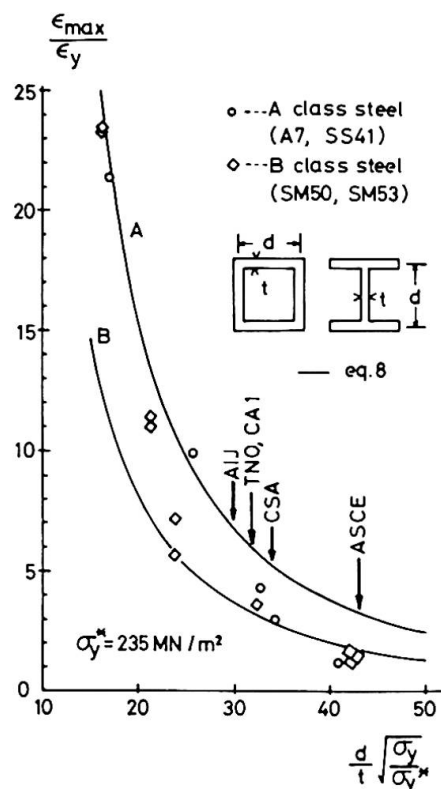


Fig.10 Deformability of Webs

Test results on plates supported on both unloaded edges are plotted in Fig.10. These were reported in Refs.51,55 for B class steels and in Refs.48,55 for A class steels. Prediction from eq.8 is also depicted in the figure. Though the number of test data is relatively small, both are in good correlation.

In both figures, the minimum width-thickness ratios specified by various specifications and recommendations (AIJ[56], ASCE[12], CA1[57], CSA[58], TNO[59]) are identified, applicability of most of which are limited up to B class steels.

5.2 Lateral-Torsional Buckling

The solutions of lateral-torsional buckling in the vicinity of plastic hinges are available in Refs.60,61 for beams under uniform moment and in Refs.62,63, 64 for beams under moment gradient.

Beams under uniform moment[60]

The analysis is based on the experimentally observed behavior of a segment of a beam. The available test results indicate that the compression flange starts to deflect laterally as soon as M_p is reached. This lateral deflexion increases, while at the same time M_p is maintained and rotation in the plane of bending continues until local buckling occurs in the most compressed portion of the compression flange. Local buckling does not commence until the average strain is equal to ϵ_{st} at the center of the segment if b/t of the compression flange is equal to or less than the critical ratio given by eq.5. In the analysis it is assumed that the compression flange and one-half of the web act as a column under the yield axial load, and the reduced flange rigidity is evaluated on the basis of a discontinuous yield concept. Thus the critical unbraced length of a uniformly bent simply-supported beam is derived as,

$$\frac{KL}{r_y} = \frac{\pi}{\sqrt{\epsilon_y}} \frac{1}{\sqrt{1+0.7R\left(\frac{h}{s-1}\right)}} \quad (9)$$

where,

K =effective length factor, $K=0.54$ if the adjacent segments are elastic and

$K=0.8$ if the adjacent segments are fully yielded

L =unbraced length of a beam

r_y =the least radius of gyration of the wide-flange shape

$s = \epsilon_{st}/\epsilon_y$

$R = \theta_u/\theta_p - 1$ = rotation capacity, in which θ_u is the rotation when the

moment capacity reaches M_p on the unloading branch of an $M-\theta$ curve, and

$\theta_p = M_p L/EI$ = the idealized rotation corresponding to elastic theory applied to the case where $M=M_p$ (Fig.11).

Beams under moment gradient [62,63]

In Ref.63, a beam model which is partly elastic and partly strain-hardened is assumed. The differential equations of lateral-torsional buckling are solved, with the elastic moduli E and G in the elastic portion, and with E_{st} and G_{st} in the yielded portions. After some assumptions and approximations having been introduced, the final formula is derived as,

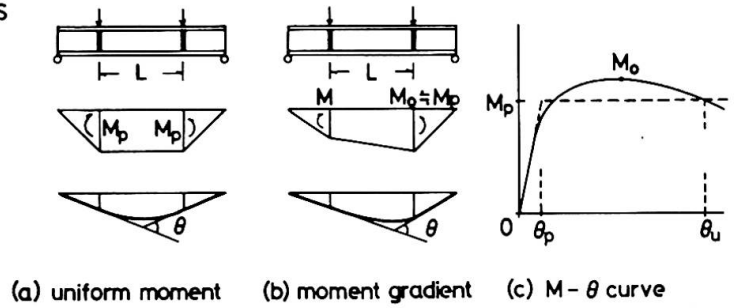


Fig.11 Moment-Rotation Relationship of Beams

$$\begin{aligned} \frac{L}{r_y} &= (60 + 40 \frac{M}{M_p}) \sqrt{\frac{36}{\sigma_y}} & \text{for } M/M_p > -0.625 \\ \frac{L}{r_y} &= 35 \sqrt{\frac{36}{\sigma_y}} & \text{for } M/M_p < -0.625 \end{aligned} \quad (10)$$

in which M and M_p are moments as defined in Fig.11 (positive in the clockwise direction).

On the other hand, it was shown in Ref.62 that, for practical cases of inelastic beams under moment gradient, failure would be initiated by local buckling rather than by lateral buckling, and thus the lateral bracing spacing should be determined by considering the beam to be under uniform moment. Taking into account of this argument, the following empirical formula is suggested in Ref.12 as an alternative of eq.10,

$$\frac{L}{r_y} = \frac{1375}{\sigma_y} + 25 \quad \text{for } -0.5 < \frac{M}{M_p} \leq 1.0 \quad (11)$$

In eqs.10 and 11, σ_y is expressed in ksi..

5.3 Rotation Capacity

In previous sections, local buckling and lateral-torsional buckling were treated as independent problem, namely provisions against local buckling and late-

ral-torsional buckling were made by limiting width-thickness ratio and unbraced length respectively based on an approximation given in terms of the strain-hardening strain ϵ_{st} . But actually those two phenomenon are intricately interconnected with each other. Hence, the deformability of beams and beam-columns can be assessed more directly by evaluating the rotation capacity R which had been defined earlier. Rotation capacity is also a good measure of earthquake resistance capacity of members and structures as R and dissipated energy are linearly related to each other.

Beams under uniform moment

The relationship in eq.9 connects the geometrical and material properties of a beam under uniform moment with the rotation capacity R . Many experimental investigations were carried out on beams under uniform moment with elastic and inelastic adjacent beams. Tests reported in Ref. 65 include A, B and C class steels, and beams tested in Ref. 10 were C class steel, and those in Refs. 66, 67 were of B class steels and those in Refs. 68, 69, 70 were of A class steels. The comparison between those experimental rotation capacities and theoretical curves from eq.9 are shown in Fig. 12. The theory explains the test results very well though it is rather conservative. But it does not give a useful information on beams with C class steel, because $s = \epsilon_{st}/\epsilon_y$ of C class steel is almost unity and then eq.9 always gives $R=0$ regardless of the value of λ .

Beams under moment gradient

There are some solutions of this problem [62, 71], but they are not given in an analytical form and hence can not indicate the significance of the various parameters involved. A linear relation between R and a parameter

$\Lambda^{-2}(t/b)(\sigma_{y0}/\sigma_y)$ was suggested in Ref. 72. Available test results are plotted in Fig. 13, in which a modified parameter $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{y0}/\sigma_y)$ is taken in horizontal axis, where $\Lambda = KL/r_y$,

$\rho = M/M_p$ (moment ratio, positive in clockwise direction) and $\sigma_{y0} = 248 \text{ MN/m}^2$ = reference yield stress. These tests were reported in Refs. 73, 74, 75, 76, 77, for A class steels, in Ref. 78 for A and B class steels and in Refs. 67, 79, 80 for B class steels and in Ref. 72 for A, B and C class steels. In the tests of Ref. 78, (t/b) was variable being kept Λ as constant, while in Refs. 72, 77, Λ was variable being kept (t/b) as constant.

Test results are rather scattered. Roughly speaking, a bi-linear relationship can be observed between R and $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{y0}/\sigma_y)$, and there seems to be a kind of yield point above which substantial increase of R could not be expected regardless of the value of $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{y0}/\sigma_y)$. There are another group of test results which appears to form a similar bi-linear relationship. In these tests, the stiffness and strength of lateral bracings and their inevitable restraint against rotation about weak axis will influence the test results. The evaluation of effective length factor K is also a matter of consideration. These limited number of

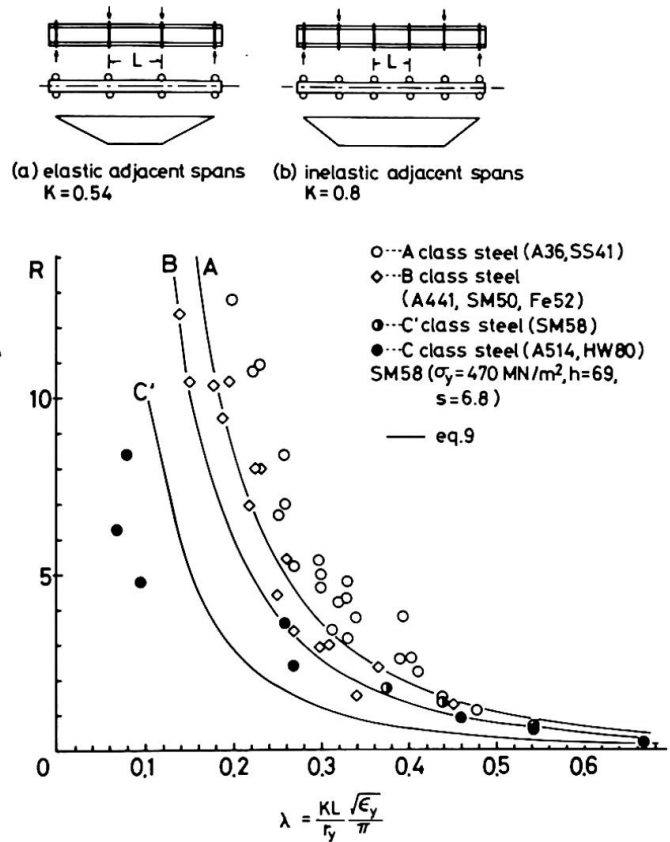


Fig.12 Rotation Capacity under Uniform Moment

test results are concerned so far, rotation capacity is inversely proportional to the yield stress.

Though above discussions were all on steel beams themselves, the restraining effect of floor slabs should be taken into account to understand the realistic behavior and to obtain the more economical design of tall buildings.

A number of experiments on the lateral-torsional buckling of unbraced wide-flange beam-columns were carried out [15, 18, 23, 81, 82, 83]. But neither any analytical information nor

systematic research are available yet on the rotation capacity of these inelastic beam-columns. In Ref. 84, it has been recommended that for beam-columns in single curvature bending, the bracing rules for beams under uniform moment should be applied, and for the moment at one end only, the rules for beams under moment gradient apply if the axial load ratio P/P_y obey the relationship

$$\frac{P}{P_y} \leq \frac{1 - L/r_x(1/\pi)\sqrt{\sigma_y/E}}{1 + L/r_x(1/\pi)\sqrt{\sigma_y/E}} \quad (12)$$

If P/P_y exceeds this limit, the bracing should be proportioned according to the rules for beams under uniform moment. In case of full double curvature the bracing spacing rules for beams under moment gradient apply.

6. FRAMES

The strong column-weak beam concept is popular in frame design, where the majority of the plastic hinges develop in the beams and the inelastic action in the columns is limited. The British approach to the plastic design of tall frames is based on the deliberate exclusion of plastic hinges from the columns [85, 86]. Most of earthquake resistant structures also have been designed according to this concept. This design method seems to have been adopted because a) the analysis becomes simple, b) the problem of lateral-torsional buckling of beam-columns has been studied inadequately as was reviewed in previous section. However, this concept should be carefully evaluated with the following facts in mind; 1) If the lateral-torsional buckling of beam-columns is studied adequately and the possibility of premature failure is eliminated, the plastic behavior of beam-columns may be superior than that of beams since the flexural rigidity of beam-columns in plastic range is approximately twice that of beams as is obvious from the $M-P-\phi$ relationship given in Fig. 5. 2) Actual beams will be strengthened in such cases that, a) they are connected with floor slabs by shear connectors, and b) beams are jointed rigidly both in x and y directions to an internal column and the frame is subjected to horizontal force in oblique direction to the plane of bays (x and y), wherein both beams in x and y directions resist to the horizontal force synthetically. In such a situation, plastic hinges will apt to form in the column even if weak-beams were assumed in the initial design, and when columns were not designed adequately to prevent local and/or lateral-torsional buckling in plastic range, premature failure will occur in columns.

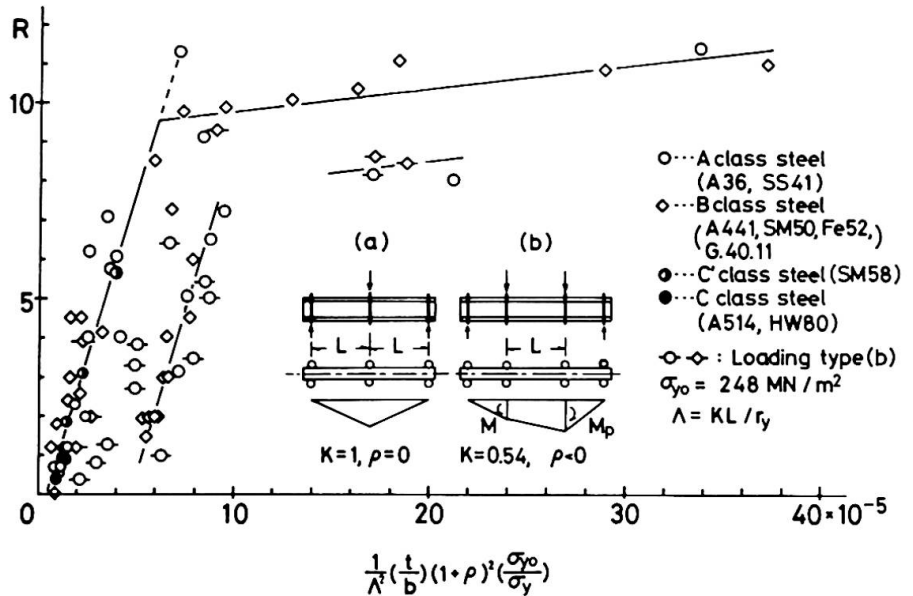


Fig. 13 Rotation Capacity under Moment Gradient

Present discussion, however, will be restricted to planer steel frames because the knowledge on the inelastic behavior of space frames and of composite beams is not enough to be reviewed generally.

Multistory frames can be categorized in two types: "braced frames" where the primary resistance to lateral loads, frame buckling and frame instability is provided by a vertical bracing system, and "unbraced frames" in which the bending resistance of the frame members themselves must account for the total frame strength and stiffness in resisting lateral loads and frame instability.

Plastic behavior of "braced frames" was investigated extensively at Lehigh University. Tests and analysis on subassemblages and full size frames were carried out for A class steel (A36) and B class steel (A441) [87,88,89,90,91]. Results of these researches were incorporated into AISC specification [92] and AISI design manual [24].

In Britain too, the research of the braced multistory frames was developed and design recommendations were published from Joint Committee of the Institute of Welding and the Institution of Structural Engineers [86,93], where the use of high-strength steel up to Grade 50 was permitted. One full-scale frame fabricated in Grade 43 steel [94], and another full-scale frame made of Grade 50 steel [95], both of which were designed in accordance with the recommendations of the reports, were carried out to establish the accuracy of the simplified design method proposed therein. Hybrid construction was adopted in the latter frame, namely Grade 50 steel was used in columns while Grade 43 steel was used in beams. Column-to-beam connections in this frame were of semi-rigid type, and the frame collapsed in beam mechanism type.

Research on unbraced frames is in progress. A large number of tests had been carried out on unbraced frames made of A class steels. Plastic design procedures of unbraced multistory frames are proposed [96,97,98]. A test on unbraced frame made of B class steel (A441) under combined gravity and lateral loads was reported in Ref. 99. This frame is a hybrid frame, where A441 steel is used in columns and A36 steel is used in beam. The test showed the behavior of the frame could be predicted by methods conventionally used for mild steel frames.

When high-strength steels are used in unbraced tall frames subject to combined gravity and lateral loads, the following factors will influence their load-deformation relationship more strongly than they will do on mild steel frames: High strength steel columns will carry the relatively higher axial forces and these axial forces will a) introduce additional moments due to $P-\Delta$ effect, b) reduce the elastic stiffness of the beam-columns and c) introduce additional moments due to member shortening (bending deformation of frames). In plastic range, strain-hardening effects become remarkable. These effects are illustrated diagrammatically in Fig. 14 taking an example of lateral load-deformation relationship of the lower story of a tall building. Q_y and Δ_y are lateral load and horizontal sway respectively, at which the first hinge develops in the frame. Of all, the effects of $P-\Delta$ moment and strain-hardening are dominating. Two pairs of model frames, one of which is

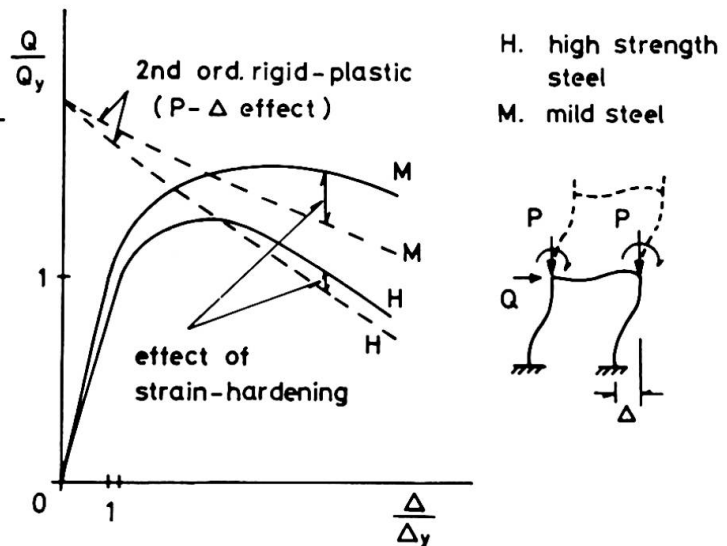


Fig. 14 Load-Deflexion Curve of Frames

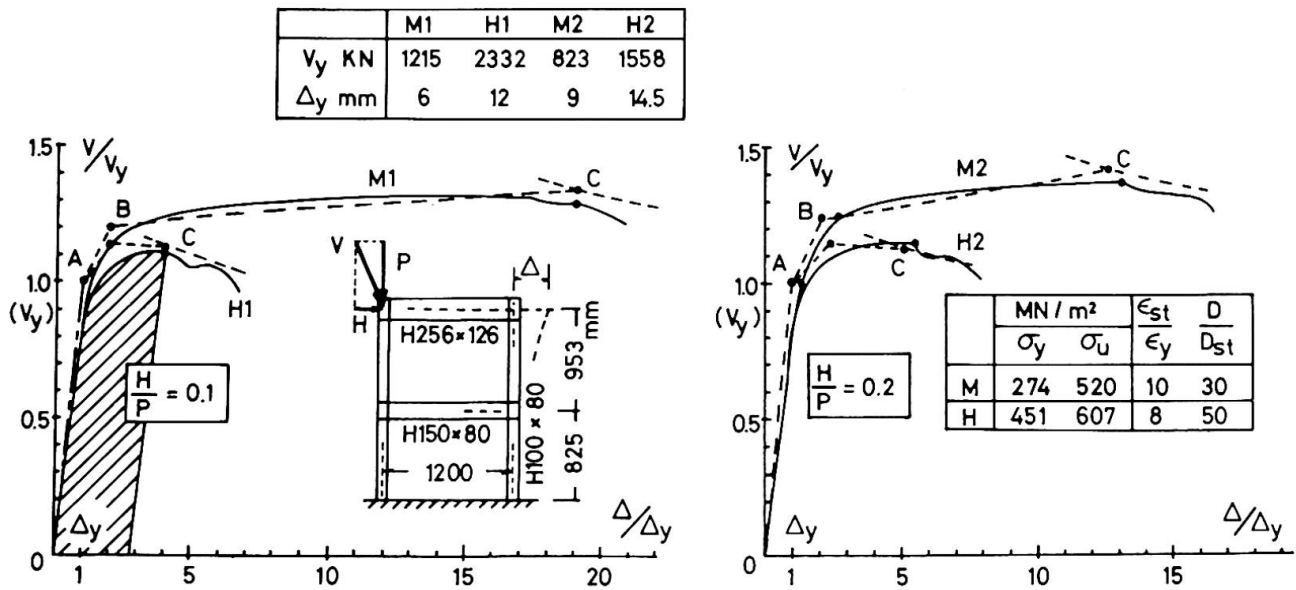


Fig.15 Test Results of Frames of Different Grades of Steels

made of A class steel(SS41) and another is made of C' class steel(SM58), were tested to compare their deformability[100]. Frames were under combined vertical and horizontal loads, and were loaded up to the collapse in proportional loading condition. The ratio of horizontal loads to vertical loads was kept to 0.1 for one pair of frames(M1,H1) and to 0.2 for another pair of frames(M2,H2). Load-deformation curves of these frames are shown in Fig.15, where V_y and Δ_y are the resultant load and the corresponding horizontal deflexion respectively at which the first hinge develops in the frame. Frames made of A class steel are indicated by M and those made of C' class steel are indicated by H. Theoretical predictions are depicted by dashed lines. Difference of deformability (ductility) between frames of A and C' class steels is quite remarkable. Dissipated energies are calculated(with respect to H1 frame, for example, the energy was calculated from the shaded area, assuming that $P \approx V$, and ignoring the work done by the vertical load), and they are 151 KJ for M1, 82 KJ for H1 and 224 KJ for M2, 208 KJ for H2. Again, mild steel frames are superior than high-strength steel frames from the view point of energy absorption capacity.

7. PLATE AND BOX GIRDERS

As the conclusion of the 8th Congress of IABSE, held at New York in September 1968, concerning the design of thin walled deep plate girders[101], it was pointed out that "the linear theory of plate stability is not an adequate basis for design of struts and girders consisting of thin-walled sections. Such design method must consider the initial geometrical imperfections of the plates as well as the residual stresses and the different yield stresses over the section and over the length of the structural member" and furthermore, the Association has recommended that the study on this theme should be continued. As a result of this recommendation a colloquium on "Design of Plate and Box Girders for Ultimate Strength" was held in London in 1971 under the sponsorship of IABSE[102], wherein main topics of a) ultimate strength of plate girders subjected to shear, -plate girders without intermediate stiffeners, b) general analytical methods, -ultimate strength of plate girders subjected to bending and to shear and bending and c) hybrid girders, fatigue problems, effects of concentrated loads, box girders, special problems, were discussed thoroughly by some twenty specialists from all over the world.

The failures of four large steel box girder bridges occurred between 1969 and 1971 emphasize the need for more research in this field. A striking test result on a box girder was reported in that Colloquium[103] that the mean collapse stress had been less than the critical stress given by the linear buckling theory for an ideally perfect flange. An international conference on "Steel Box Girder Bridges" organized by the Institution of Civil Engineers was held in London in 1973 where again the importance of the introduction of the ultimate strength concept to the design was emphasized[104].

It must be beyond the scope of this report to review and discuss on the structural concepts, the methods of analysis and the experimental observations presented in above colloquium and conference. But if one might be permitted to add a comment, most of the contributions presented seem to have been devoted to the construction of collapse models and the analyses of them, and the effects of plastic behaviors of comprising elements which show substantial difference among grades of steels were not counted on. These difference of material properties in plastic range were not taken into account even in the evaluation of test results, though a number of tests had been carried out on girders made of C class steels.

It might become possible to obtain the more optimum and the more reliable design, if the knowledge of plastic behavior of flanges and stiffeners made of various grades of steels, of which evaluations had been made in previous sections, were properly reflected on the research and analysis of those girders.

Based on the recent studies on hybrid beams (fabricated beams and girders which use a stronger steel in the flanges than in the web) the subcommittee of the Joint ASCE-AASHTO Committee on Flexural Members had published a report in the form of design recommendations in 1968[105]. The subcommittee concluded that composite and noncomposite hybrid beams can be designed efficiently using an allowable stress based on the moment required to initiate flange yielding. This allowable flange stress is a function of the beam dimensions and the ratio of the yield strengths of the two steels, and is slightly lower than the allowable stress normally used for the flange steel. The bending stress in the web does not have to be checked when this reduced allowable flange stress is used. However, the shear stress in the web must be limited to the normal allowable stress for the web steel. The available fatigue data indicated that these hybrid beams can generally be designed for fatigue as if they were made entirely of the grade of steel used in the flanges. This Joint Committee report led to the adoption of design specifications for highway bridges and for buildings in USA: The American Association of State Highway Officials adopted provisions for both noncomposite and composite hybrid girders in 1969[106], and the American Institute of Steel Construction adopted provisions for noncomposite hybrid girders in 1969[92].

At present time, it is assumed that the hybrid beams will be designed on the basis of the allowable stress design, namely on the basis of the initial flange-yield moment. But it is desirable that both hybrid girders and homogeneous girders should be designed on the same concept based on the ultimate strength state. Ref. 107 presented at the IABSE Colloquium seems to be on the verge of this approach.

Effect of web breathing on the fatigue strength is an important problem, especially for hybrid girders, which belongs to the problem of structural fatigue (or low cycle fatigue) rather than that of classical material fatigue, but the results of these tests should be carefully evaluated in consideration of the extent of the web breathing which likely to occur under the assumed service load level in association with the possible degree of imperfection of the actual girder.

8. FRACTURE AND FATIGUE

8.1. Welded Joint and Transitional Mode of Fracture

Over the period from 1937 to 1940, brittle fractures took place in succession and led to the failures of three steel Vierendeel type welded bridges in Belgium.

During the same period, brittle fracture was also observed in Rüdersdorf in Germany, an all welded steel plate girder made of St52. The fracture initiated from micro-cracks existed in heat affected zone due to welding.

As the results of an investigation to the cause of this fracture[108], it has been suggested that: 1) bead bend test proposed by Kommerrelé simulates well the brittle fracture behavior of welded steel plates. 2) in order to prevent brittle fractures, adequate notch toughness is required as one of the material properties of steels. 3) notch toughness depends on temperature and 4) carbon equivalent, which plays a role for the initiation of cracks at heat affected zone, is an important factor together with such factors as structural restraints and pre-heating during welding. Since then continued efforts have been paid resulting in structural steels with greatly improved notch toughness. In Europe, however, there exists little intention of using extra-High strength steels for welded structures, which is due to the inherently more sensitive character of the steels to cracking at welds.

While in USA, quenched and tempered high strength steel with tensile strength exceeding 789 MN/m^2 (C class steel) has been introduced around 1955 for structural applications in welded constructions. Coincident with the introduction of this A514 steel, similar heat treated steel with tensile strength exceeding 592 MN/m^2 (SM58) has also been developed in Japan. The most important feature of this type of steels is an easy attainment of extremely high strength with few low alloy elements in spite of low carbon content (about 0.1% or more), and hence with low equivalent carbon content. This feature made it possible to perform crack free welding under moderate pre-heating condition. Moreover proper alloy elements and heat treatment contribute to maintain adequate notch toughness. The good weldability of the heat treated steels helped to increase their wider use in welded structures. It has to be noticed, however, that because of the decrease of notch toughness level by the formation of upper bainite arising from welding heat cycles at weld fusion lines, and such welding defects as cracks, undercuts, various angle changes of joints and residual stresses, fractures may take place under low stress level. In order to avoid these failures, US Steel Corp., recommended to limit the amount of heat input depending on the thickness of plates. Improvements have also continued with such intension as to make it possible to weld this type of steels with larger heat input. Based on intensive research works[109], 75mm thick 785 MN/m^2 strength heat treated steel has been successfully used in the welded construction works of Osaka Port Bridge in Japan, which opened to traffic on July 1974.

Numerous surveys and investigations have been performed in relation to various accidents due to brittle fractures occurred in such welded structures as ships and pressure vessels. One of the findings of these research works is that tensile tests on notched wideplate resemble the situation and satisfy the conditions at the locations of brittle fractures in practical structures[110]. Various reports were prepared on the initiation and propagation of brittle fractures[110,111]. Considering the stress intensity factors K_I computed based on the theory of elasticity at the tips of notches and the critical stress intensity factors K_{IC} obtained experimentally from tensile tests on notched wideplate together with their temperature depending characters, the characteristics of brittle fractures are being explained in terms of equivalent notch effects which represent various defects and size effects [111,112,114]. Efforts[111] have been paid to evaluate the relationship among K_{IC} , $\sqrt{E}(\text{CVN})$ and yield stress σ_y . The efforts revealed that for steels with increasing yield stresses, a larger energy absorbing capacity $\sqrt{E}$ is necessary to maintain the same K_{IC} value. From the view point that the effects of plastification around the tips of notches have to be considered in addition to the characteristic factors based on the analysis of linear fracture mechanics, the concept of crack opening displacement (COD) is being proposed[111,113,114].

8.2. High Cycle Fatigue

The fatigue strength of materials free of defects may proportionally increase with the increase of its static strength. On the other hand the influence of the defects present in practical structures which act to reduce fatigue strength is being accelerated with the increase of tensile strength. This is due to the fact that high cycle fatigue is governed by localized stress concentrations arising from structural shape and material defects. As a natural consequence, experimental results scatter largely from tests to tests. S-N curves do not necessarily represent fatigue strength of steels, but they are simply statistical representations of test results.

Due to the presence of numerous defects, the fatigue strength of welded joints with high strength steels is rarely improved compared with that with ordinary strength steels[114,115,116], nevertheless efforts are being continued to improve the fatigue strength.

In the field of fracture mechanics, attempts have been made successfully[114, 116] to explain some of the characteristic behaviors of high cycle fatigue by making use of the concept of K_I and K_{IC} similar to those familiar in the analysis of brittle fractures.

8.3. Low Cycle Fatigue

Low cycle fatigue is inherently different phenomenon compared with high cycle fatigue: the mechanism of low cycle fatigue failures is similar to that of static failures of structures, and hence low cycle fatigue strength depends largely on structural behaviors under static loading. One of the features of high strength steels under the circumstances for low cycle fatigue is that C class steels soften when subjected to repeated loading and plastic energy stored at the tip of a crack increases, whereas A and B class steels harden under repeated loading and the same energy decreases.

APPENDIX. HIGH-STRENGTH STEELS IN THE WORLD

Country	Standards	Designation	Min. Yield Stress MN/m ²	Tens. Strength MN/m ²	Class	Y	Remarks
INTERNATIONAL	ISO	# Fe 42	245	412 ~ 490	A	0.594	
		Fe 52	343	490 ~ 608	B	0.70	
AUSTRALIA	AS A 186 and A 187 (weathering)	# Grade 250	248	> 412	A	0.602	Columbium and/or Vanadium * up to 13mm ** up to 9mm
		" 350	344	> 481	B'	0.715	
		* " 400	412	> 515	B'	0.8	
		** " 500	481	> 550	B'	0.872	
BELGIUM	NBN 631	# Fe 42	250	420 ~ 500	A	0.595	
		Fe 52	350	520 ~ 620	B	0.672	
CANADA	CSA	# G 40.8	262	448 ~ 586	A	0.585	Weathering steel
		G 40.11	345	483 ~ 655	B	0.715	
		G 40.18	689	793 ~ 931	C	0.87	
CZECHO-SLOVAKIA	CSN 73 - 1401	# 10370	250	350	A	0.715	
		11523	360	520	B	0.692	
		11483	380	480	B	0.791	
EAST GERMANY	TGL, 12910	# St 38-s	230	380	A	0.605	
		11523	350	640	B	0.547	
		11483	370	620	B	0.596	
		St 52-3	350	520	B	0.673	
ENGLAND	BS 4360	# Grade 40	230	400 ~ 480	A	0.575	Columbium and/or Vanadium
		" 50	345	500 ~ 620	B'	0.69	
		" 55	430	550 ~ 700	B'	0.782	
FRANCE	NFA 35-501	# Grade E24	235	363 ~ 441	A	0.647	
		" E30	294	461 ~ 559	B	0.637	
		" E36	353	510 ~ 608	B	0.692	

(continued)

Country	Standards	Designation	Min. Yield Stress MN/m ²	Tens. Strength MN/m ²	Class	Y	Remarks
FRANCE	NFA 35-501	Grade A50	294	490 ~ 588	A	0.60	Impact tests Not Req'd. Limited Weldability
		" A60	333	588 ~ 706	A	0.567	
		" A70	363	686 ~ 833	A	0.530	
INDIA	IS 226, 961, 2062	# St42(S,W)	230	410 ~ 530	A	0.56	
		St55 HTW	340	540 min	B	0.63	
		St 58 HT	350	570 min	B	0.613	
ITALY	UNI 5334.64	# Fe 42	230	420 ~ 500	A	0.55	
		Fe50.1,50.2	290	500 ~ 600	B	0.58	
JAPAN	JIS G - 3101 ~ 3106	# SS41, SM41	235	403 ~ 510	A	0.583	Columbium and/or Vanadium
		SM 50	315	490 ~ 607	B	0.642	
		SM 50Y	353	490 ~ 607	B'	0.72	
		SM 53	353	520 ~ 642	B	0.678	
	WES	SM 58	452	570 ~ 718	C'	0.794	
		HW 70	686	785 ~ 932	C	0.874	
		HW 80	785	864 ~ 1030	C	0.908	
		HW 90	883	951 ~ 1130	C	0.93	
NETHERLANDS	EURO NORM 25 - 72	# Fe 360	225	360 ~ 440	A	0.625	Limited Weldability Limited Weldability
		Fe 510	345	510 ~ 610	B	0.677	
		Fe 590	325	590 ~ 710	A	0.55	
		Fe 690	355	690 ~ 830	A	0.515	
POLAND		# St 35	226	373 ~ 464	A	0.606	
		18G 2A	342	490 ~ 626	B	0.697	
RUSSIA	GOST 380 Gost 5058	# BG 3	240	440 ~ 470	A	0.546	Columbium and/or Vanadium
		14Г 2	330	470	B	0.702	
		15Г C	340	480	B	0.702	
		10Г 2C	350	500		0.7	
		10XCHA	400	540		0.741	

(continued)

Country	Standards	Designation	Min. Yield Stress MN/m ²	Tens. Strength MN/m ²	Class	Y	Remarks
SWEDEN	SIS	# 1412	260	430 ~ 510	A	0.605	
		2172	300	490 ~ 590	B	0.612	
		2132	350	510	B	0.686	
		2142	390	530	B	0.736	
SWITZERLAND	SIA 161 (1972)	# St 24/37	227	364 ~ 443	A	0.623	
		St 36/52	345	512 ~ 611	B	0.674	
U.S.A.	ASTM	# A 36	248	400 ~ 552	A	0.62	A242: weathering steel Columbium and/or Vanadium Weathering Steel
		A242, A441, AA441	317	462	B	0.686	
		A572 Gr 45	310	414	B'	0.748	
		50	345	448	B'	0.77	
		55	379	483	B'	0.785	
		60	414	517	B'	0.8	
		A 588	345	483	B	0.713	
		A 514	690	793 ~ 930	C	0.87	
WEST GERMANY	DIN 17100	# St 37.283	240	370 ~ 450	A	0.647	
		St 46.223	290	440 ~ 540		0.658	
		St 52.3	360	520 ~ 620	B	0.691	

1. Type: A = carbon steel, B = high strength Low - alloy steel, C = heat treated constructional alloy steel
2. Listed are weldable steels only
3. Mechanical properties of steels thickness of which are 16 ~ 40mm are listed
4. $Y = \frac{\text{Yield stress}}{\text{Tensile strength}} = \text{Yield ratio of material (maximum)}$
5. Typical mild steels are included for reference and which are identified by # mark.

REFERENCES

For brevity, the titles of a number of professional societies have been abbreviated as follows:

AIJ Architectural Institute of Japan
 AISC American Institute of Steel Construction
 AISI American Iron and Steel Institute
 ASCE American Society of Civil Engineers
 ASTM American Society for Testing and Materials
 CRC Column Research Council, USA
 IABSE International Association for Bridge and Structural Engineering
 ICE The Institution of Civil Engineers, England
 ISE The Institution of Structural Engineers, England
 JSCE Japan Society of Civil Engineers
 WRC Welding Research Council, USA

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SUMMARY

The state-of-the-art on the structural behaviour of members and structures made of high-strength steels is outlined. Characteristics of the structural behaviour of those made of high-strength steels are discussed in relation to those made of mild steel in order to provide the information by which a designer can decide whether a grade of high-strength steel should or should not be used in a particular part or in a particular structure in association with the imposed loading condition.

RESUME

L'état des connaissances actuelles est présenté sur le comportement structural des éléments et des constructions réalisés en acier à haute résistance. Les caractéristiques du comportement structural sont comparées pour des constructions en acier à haute résistance et en acier doux. Cette comparaison facilite le choix de l'ingénieur quant à l'emploi d'un acier à haute résistance ou d'un acier doux pour des éléments ou des ensembles de structure en fonction des conditions de charge imposées.

ZUSAMMENFASSUNG

Die Autoren berichten über die heutigen Kenntnisse über das Tragverhalten von Bauteilen und Bauwerken aus hochfestem Stahl und deren charakteristisches Tragverhalten bei Verwendung hochfester Stähle und von normalem Baustahl wird verglichen. Dadurch ergeben sich die Entwurfsgrundlagen, nach denen entschieden werden kann, ob in einem bestimmten Bauteil oder Bauwerk im Zusammenhang mit den vorgeschriebenen Belastungsannahmen ein spezieller hochfester Stahl angewendet werden soll.

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Entwurfsprobleme

Design Problems

Problèmes de conception

OTTO JUNGBLUTH

Prof. Dr.-Ing.

Technische Hochschule Darmstadt
Darmstadt, BRD1 Verfestigungsmechanismen höherfester Stähle

Von allen Gebrauchseigenschaften technischer Werkstoffe steht wohl die Festigkeit an erster Stelle. Aus verschiedenen Überlegungen läßt sich schließen, daß die theoretischen Grenzen einer polykristallinen Eisenlegierung etwa zwischen 20 und 8000 N/mm^2 ^{x)} liegen können. Daraus folgt, daß noch ein weiterer Bereich für Entwicklungsarbeiten offen ist. Technisch kann die Zugfestigkeit von Stahl z. Zt. etwa zwischen 300 und 3000 N/mm^2 variiert werden und liegt somit weit über den erreichbaren Zugfestigkeiten von Titan, Aluminium und Kunststoffen (Bild 1).

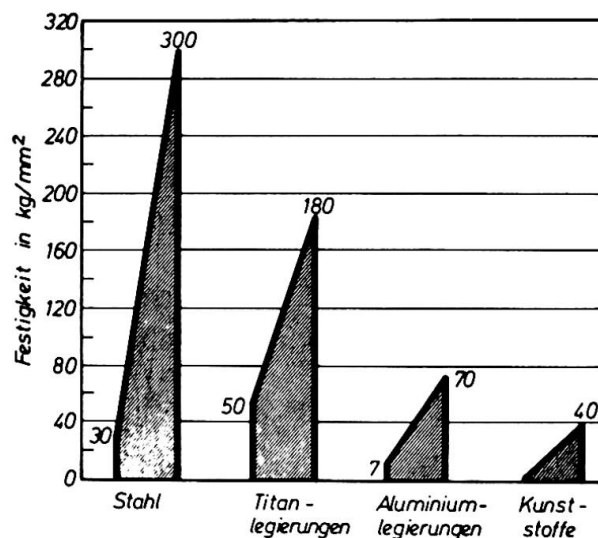


Bild 1: Technische Festigkeit von Werkstoffen

Die Werkstoff-Forschung unterscheidet z. Zt. verschiedene Mechanismen zur Steigerung der Festigkeit von Stahl (Bild 2). Da in den meisten Stählen nicht nur ein Mechanismus, sondern mehrere von ihnen eine Rolle spielen, muß es das Ziel der Werkstoff-Forschung sein, die einzelnen Beiträge der verschiedenen Mechanismen zur Festigkeitserhöhung so zu kombinieren, daß hoch- und ultrahochfeste Stähle entwickelt werden, wobei gleichzeitig eine Reihe von anderen wichtigen Stahleigenschaften außer der Festigkeit erhalten bleiben oder sogar zugezüchtet werden muß.

^{x)}in den Bildern wurde noch die alte Bezeichnung "kp" verwendet:

1 kp = 10 N (N = Newton)

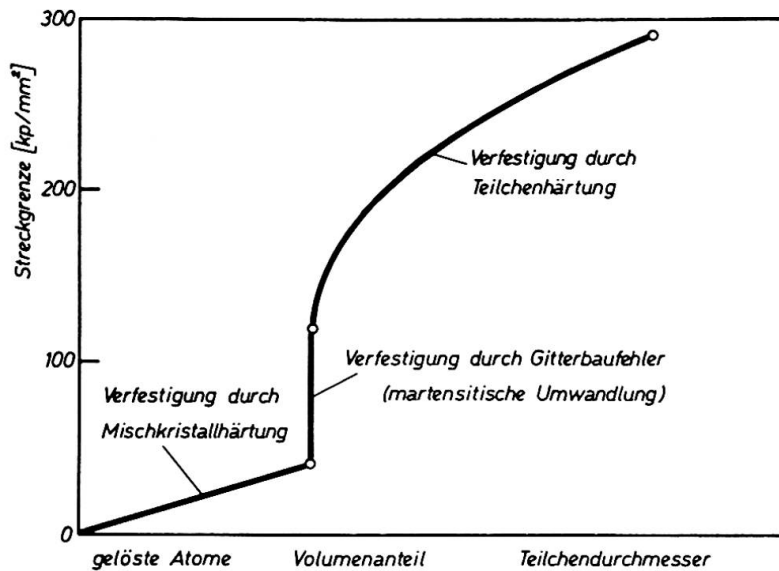


Bild 2:

Beiträge verschiedener Verfestigungsmechanismen

Etwa seit den fünfziger Jahren begann in der Stahlindustrie Europas, Japans und der USA die systematische Entwicklung schweißgeeigneter Baustähle mit Streckgrenzenwerten über 360 N/mm^2 (Bild 3). Streckgrenzenfestigkeiten bis zu dieser Höhe konnte man bereits seit den dreissiger Jahren (St 52) herstellen, wenn auch mit begrenzter Schweißeignung. Diese Werkstoffentwicklung dürfte für die Gruppe der niedrig legierten, normalisiert geglähten mit Streckgrenzenwerten bis ca. 500 N/mm^2 und für die Gruppe der legierten, flüssigkeitsvergüteten mit Streckgrenzenwerten bis ca. 1000 N/mm^2 nunmehr zu einem gewissen Abschluß gekommen sein. Auf dem Weltmarkt werden z. Zt. über hundert Sorten dieser hochfesten, schweißgeeigneten Baustähle angeboten /1/, z. B.

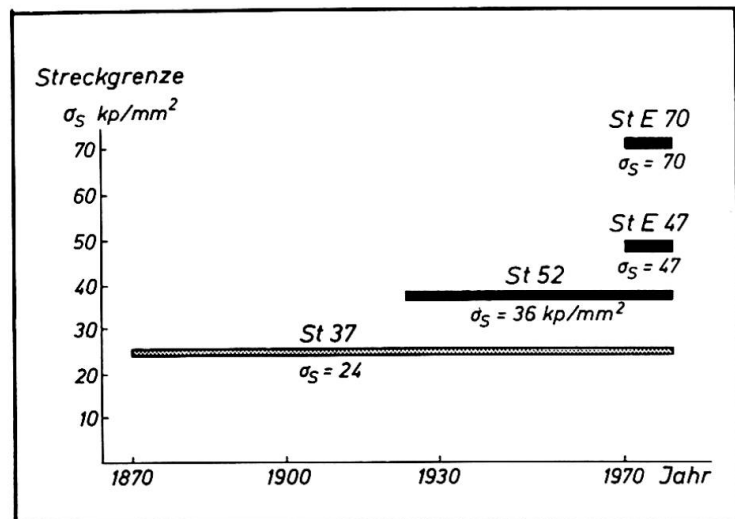


Bild 3: Entwicklung der Streckgrenzen

PROTENAX
FB 70

DUCOL W 30
HW 50
EX-TEN 70

HSB-77-V
SUPERELSO

LOYKON
WELTEN 80

N-A-XTRA 100

(Belgien)

(Deutschland)

(Frankreich)

(Großbritannien)

(Japan)

(USA)

Der Verfasser beschränkt sich aber beispielhaft auf die beiden Festigkeitsklassen St E 47 und St E 70 mit den Streckgrenzen $\sigma_s = 470 \text{ N/mm}^2$, die in der Bundesrepublik Deutschland seit dem 1. 1. 1973 baubehördlich allgemein zugelassen sind. Allen diesen hochfesten Baustählen gemeinsam ist die

Feinkörnigkeit

und damit in engem Zusammenhang stehend die

Sprödbbruchunempfindlichkeit

als Voraussetzung ihrer Schweißreignung /2/ /3/ /4/. Da zwischen dem Mikrogefüge des Werkstoffs Stahl und seinen mechanischen Eigenschaften sehr komplizierte Beziehungen bestehen, haben die letzten Jahrzehnte zu einer Intensivierung der Forschungsarbeiten auf diesem Gebiet geführt, um die verschiedenen Verfestigungsmechanismen zu klären /5/ /6/. Man hat erkannt, daß für die Festigkeit von Stahl folgende Verfestigungsmechanismen und Einflußgrößen maßgebend sind:

1. Verfestigung durch Verfeinerung des Ferritkorns
2. Mischkristallverfestigung durch im Eisengitter gelöste Fremdatome
3. Verfestigung durch Ausscheidungshärtung
4. Kaltverfestigung durch Erhöhung der Versetzungsdichte
5. Verfestigung durch isotherme Zwischenstufenumwandlung

Je nachdem, welche Verfestigungsmechanismen in geeigneter Dosierung und Kombination zur Wirkung kommen, kann man vier Baustahlgruppen unterscheiden:

1. Die unlegierten Stähle
mit Streckgrenzenwerten von etwa 210 bis 360 N/mm²
2. die niedriglegierten normalgeglühten Stähle
mit Streckgrenzenwerten von etwa 260 bis 500 N/mm²
3. die niedriglegierten vergüteten Stähle
mit Streckgrenzenwerten von etwa 500 bis 1000 N/mm²
4. die gezogenen kaltverfestigten Stähle
mit Zugfestigkeiten von etwa 1200 bis 2000 N/mm².

Während die drei ersten Gruppen schweißgeeignet hergestellt werden können, ist dies für die vierte Gruppe der gezogenen kaltverfestigten Stähle nicht der Fall.

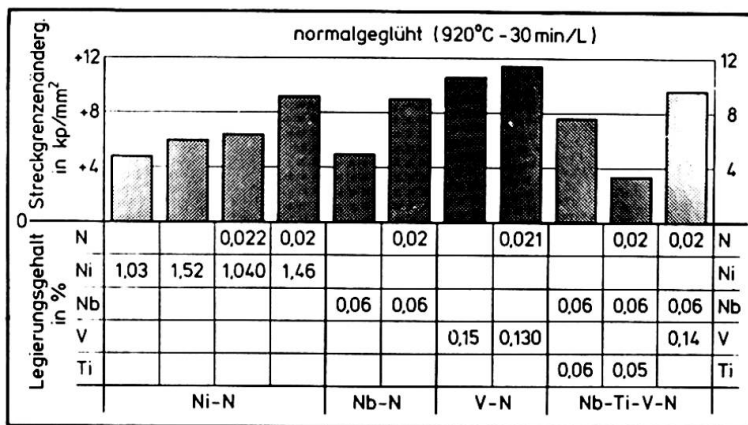


Bild 4: Wirkung der Legierungszusätze bei normalgeglühten Feinkornbaustählen

Bei den niedriglegierten Baustählen bis etwa 500 N/mm² Streckgrenze führt die Zulegierung geringer Mengen von Vanadin, Titan und Niob durch die Abbindung des Stickstoffs zur Bildung von Nitriden und Karbonitriden und durch die so erzielte Kornverfeinerung zu höherer Festigkeit (Bild 4) bei guter Zähigkeit. Vielfach wird auch noch Nickel zulegiert insbesondere, um die Kerbschlagzähigkeit bei niedrigen Temperaturen zu verbessern. Bild 5 zeigt einige typische chemische Zusammensetzungen dieser Stahlsorte.

Stahlsorte	Stahlart	% C	% Si	% Mn	% P	% S	% N	% Cr	% Cu	% Nb	% Ni	% Ti	% V	% Mo	% Zr	% B
St E 47	Ni-V	≤ 20	.10/.50	1.1/1.7			≤ .020	-	-	-	.15/.80	-	.10/.20	-	-	-
	Cu-Ni-V	≤ 18	.10/.55	1.1/1.5	≤ .035	≤ .035	≤ .020	-	.30/.70	≤ .03	.40/.70	-	.08/.20	-	-	-
	Ni-Ti	≤ 20	.10/.80	1.1/1.8			≤ .007	-	-	-	.50/.80	.10/.20	-	-	-	-
St E 70	Ni-Cr-Mo-B	≤ 20	.15/.35	.60/1.0	≤ .035	≤ .035	≤ .015	.40/.85	.15/.50	-	.70/1.0	-	.03/.08	.40/.80	-	.002/.008
	Cr-Mo-Zr		.50/.90	.70/1.1				.80/1.0	-	-	-	-	-	.20/.80	.08/.12	-

Bild 5: Chemische Zusammensetzung niedriglegierter normalgeglühter Baustähle, nach /8/

Diese Stähle kommen im normalgeglühten Zustand (z. B. 920°C 30 min/Luft) zum Einsatz, wobei in manchen Fällen das Normalglühen durch geregelte Temperaturführung bei und nach dem Walzen ersetzt werden kann.

Während man bei diesen niedriglegierten Baustählen den Kohlenstoffgehalt auf 0,15 bis 0,20 % begrenzt, stellt man seit einiger Zeit auch noch kohlenstoffärmere sogenannte perlitreduzierte Baustähle mit einem C-Gehalt von 0,02 bis 0,12 % her. Besonders bei dieser Sondergruppe wird zur Erzielung einer günstigen Kombination von Festigkeits- und Zähigkeitseigenschaften der Gefüge- und Ausscheidungszustand in der Weise optimal aufeinander abgestimmt, daß in gegenseitiger Abhängigkeit die Temperaturführung und der Ver-

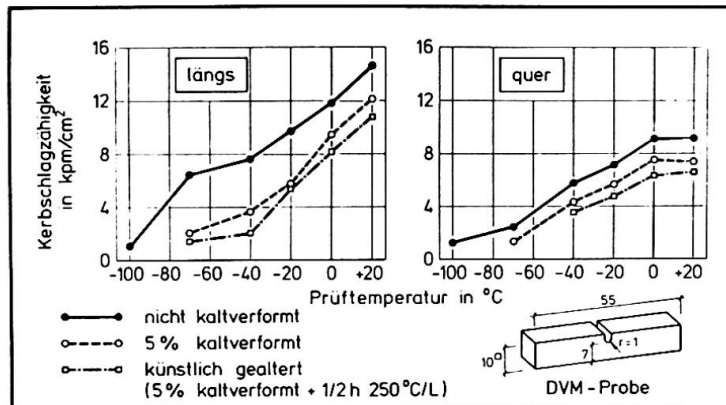


Bild 6: Kerbschlagzähigkeit der ferrit/perlit-Stähle (St E 47)

formungsgrad des gesamten Walzprozesses gezielt eingestellt werden. Neben der chemischen Analyse spielt also auch die thermomechanische Behandlung (niedrige Walztemperatur bei hohem Verformungsgrad im letzten Stich - bei Breitband z. B. 850°C und 30 %) eine entscheidende Rolle [7].

Diese niedriglegierten Ferrit-Perlitstähle besitzen unter Beachtung bestimmter Verarbeitungsrichtlinien gute Schweiß-

eigenschaften, was durch die gute Kerbschlagzähigkeit auch bei sehr niedrigen Temperaturen bewiesen wird (Bild 6).

Die zweite noch höherfeste Gruppe schweißgeeigneter Baustähle mit durch Wasservergütung erzieltm Gefüge, das aus angelassenem Martensit und unterem Bainit besteht, umfaßt im allgemeinen den Streckgrenzenbereich von ca. 500 bis 1000 N/mm². Die wesentlichen Legierungselemente zur Sicherung der Durchvergütung sind Chrom und Molybdän, zu denen noch fallweise Kupfer, Nickel, Vanadin sowie Zirkon und Bor treten. Bild 5 zeigt die bekanntesten chemischen Zusammensetzungen, die unter den Markennamen N-A-XTRA und T 1 bekannt geworden sind. Auch diese wasservergüteten Baustähle besitzen nach der Vergütung ein extrem feinkörniges ferritisches Gefüge mit hohem Formänderungswiderstand und gutem Formänderungsvermögen.

Als dritte Gruppe der hochfesten Stähle seien der Vollständigkeit halber noch die patentierten, kaltverfestigten Kohlenstoffstähle mit sorbitischem Gefüge als Rundstähle und Drähte erwähnt, die aber wegen des hohen C-Gehaltes nicht zum Schweißen geeignet sind.

Wegen ihrer außerordentlich hohen Festigkeit haben sie aber

in den letzten Jahren für seilverspannte Brücken und Dächer große Bedeutung gewonnen. Eine typische chemische Zusammensetzung zeigt Bild 7.

	%C	%Si	%Mn	%P	%S
Seildrähte	0,48/0,75	0,10/0,25	0,40/0,85	≤ 0,035 ≤ 0,055	≤ 0,035

Bild 7: Chemische Zusammensetzung von C-Stählen für hochfeste Drähte

2 Festigkeitsklassen und Lieferformen hochfester Baustähle

Da man es bei der Baustahlherstellung unter der Voraussetzung der Schweißreignung in der Hand hat, praktisch jede gewünschte Festigkeitsstufe von 200 bis 1000 N/mm² Streckgrenze zu erschmelzen, ist aus Gründen der rationellen Anwendung im Stahlbau die Beschränkung auf wenige Festigkeitsklassen notwendig. In der Bundesrepublik Deutschland hat man sich, nachdem die klassischen Baustähle mit den Streckgrenzen 240 N/mm² (St 37) und 360 N/mm² (St 52) bereits genormt sind, im Zuge der am 1.1.1974 behördlich eingeführten DAST-Richtlinie 011 auf die Normung der beiden Festigkeitsklassen St E 47 und St E 70^{x)} beschränkt. Trotz unterschiedlicher chemischer Zusammensetzung sind die gewährleisteten mechanischen Eigenschaften für die beiden Stahlfestigkeitsklassen St E 47 und St E 70 gleich (Bild 8). Mit Aus-

Stahl	Erzeugnisform	Behandlungszustand ¹⁾	Mechanische Eigenschaften														Dorndurchmesser beim Falversuch ¹⁾¹⁾¹⁾		
			Streckgrenze für die Dicken in mm ²⁾				Zugfestigkeit für die Dicken in mm ²⁾		Bruchdehnung (L ₀ = 5d ₀) %	Probenform	lage	Kerbschlagzähigkeit ³⁾ bei							
			≤12	>12 ≤16	>16 ≤35	>35 ≤50	≤50	>50				-60°C	-40°C	-20°C	0°C	+20°C	längs	quer	
			kp/mm ² mind.				kp/mm ²		kpm/cm ² mind.										
St E 47	Blech Profil, geschweißt, Hohlprofil (Rohr)	N	47	48	45	44	57 bis 74		17	ISO-V	längs							3a	4a
	nahtloses Hohlprofil (Rohr)		47	48	45	43	—	54 bis 74						4,5	5,5	6,5			
St E 70	Blech Profil, geschweißt, Hohlprofil (Rohr)	V	70			—		80 bis 95	—	16	ISO-V	längs	4	5	6	7	7	3a	4a

¹⁾ N = normalgeglüht, V = vergütet (wasservergütet)
²⁾ Für dickere Erzeugnisse sind die entsprechenden Werte zu vereinbaren.
³⁾ Maßgebend ist der Mittelwert aus drei Versuchen; der kleinste Einzelwert darf 70% des gewährleisteten Mittelwertes nicht unterschreiten.

⁴⁾ Geforderter Biegewinkel jeweils 180°.
⁵⁾ a = Probendicke
⁶⁾ Bei der Bestellung ist zu vereinbaren, ob Längs- oder Querproben zu prüfen sind.
⁷⁾ Nicht für Hohlprofile (Rohre)

Bild 8: Mechanische Eigenschaften der hochfesten Baustähle St E 47 und St E 70 nach /8/

nahme der Rohre sind Walzprofile z. Zt. nur in der Festigkeitsstufe St E 43 lieferbar und daher bisher nicht genormt. Die Ausweitung des St E 47 auch auf Profile insbesondere Breitflansch- und Parallelfanschträger ist aber zu erwarten.

Die folgenden Hinweise auf die Eigenschaften hochfester schweißgeeigneter Feinkornbaustähle basieren auf der Normung nach der deutschen Vorschrift DAST-Richtlinie 011 /8/, sie gelten aber im Prinzip auch für die Vorschriften anderer Länder /9/ /10/ /11/ /12/.

Die härtesten Baustähle bis zu Streckgrenzenwerten von etwa $\sigma_s = 360 \text{ N/mm}^2$, also St 52 nach DIN 17 100 sowie nach ASTM A 441, BS 4360 Grade 50 werden, als schon längere Zeit zum Stand der Technik gehörend, hier nicht behandelt.

^{x)} Die Bezeichnung E steht für Elastizitätsgrenze $\approx$ Streckgrenze

3 Eigenschaften im Hinblick auf den Entwurf

3.1 Zug- und Druckfestigkeit

Auch bei gleicher chemischer Zusammensetzung und Stahlbehandlung sind die Zug- und Druckfestigkeit bzw. die Streck- und Stauchgrenze keine konstanten Größen. Für den St E 47 gilt, daß seine Zugfestigkeit und Streckgrenze mit der Bauteildicke abfallen, und zwar die Streckgrenze mehr als die Zugfestigkeit. Deshalb hat man nach /8/ die zulässige Höchstdicke im allgemeinen auf $t = 60 \text{ mm}$ begrenzt. Bei den wasservergüteten Stählen der Festigkeitsklasse St E 70 dagegen ist keine wesentliche Festigkeitsverminderung mit zunehmender Dicke festzustellen. Hier erfolgte die Höchstdickenbegrenzung auf $t = 50 \text{ mm}$ zunächst wegen der Sicherung der Durchvergütung. Unter Zugrundelegung einer Sicherheit von $\gamma = 1,7$ für den Lastfall Hauptlasten (H) und $\gamma = 1,5$ für Haupt- und Zusatzlasten (HZ) gegenüber der Streckgrenze gelten folgende zulässigen Spannungen (Bild 9):

	Spannungsart	St E 47		St E 70	
		Lastfall		Lastfall	
		H	HZ	H	HZ
1	Druck und Biegedruck	2800	3100	4100	4600
2	Zug und Biegezug	3100	3500		
3	Schub	1800	2000	2400	2700
4	Vergleichsspannung	3500	3750	4600	4900

Bild 9: Zulässige Spannungen für Bauteile

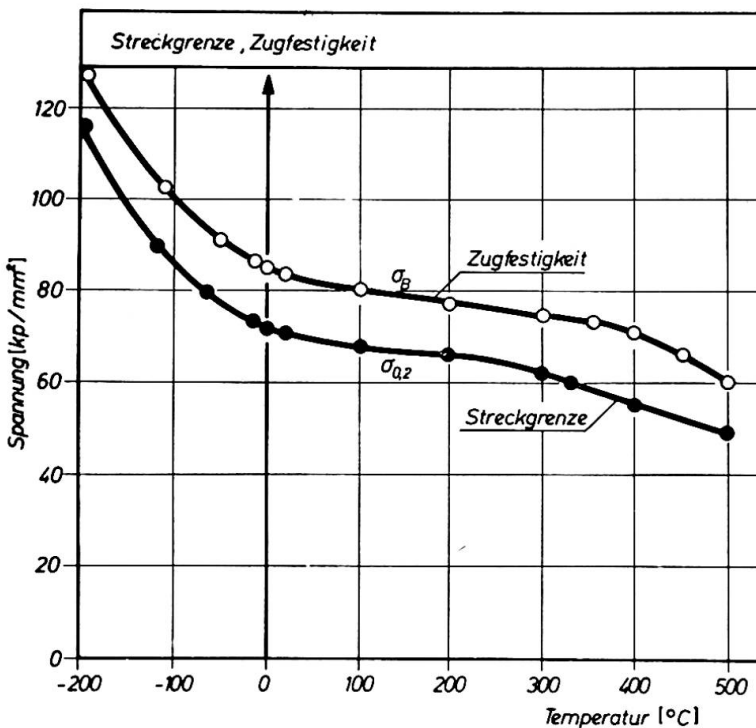


Bild 10: Festigkeit abhängig von der Temperatur (St E 70)

Daß Zugfestigkeit und Streckgrenze in bekannter Weise auch von der Temperatur abhängig sind, zeigt Bild 10, wobei der klimatische Bereich für Oberflächentemperaturen von etwa -50°C bis $+100^{\circ}\text{C}$ als hinreichend konstant angenommen werden kann.

Hinsichtlich der Zug- und Druckfestigkeit ist noch zu bemerken, daß der Bauschinger-Effekt (Verminderung der Elastizitätsgrenze durch Kaltreckung oder -stauchung bei anschließender Lastumkehr) für höherfeste Stähle stärker zur Wirkung kommt. Im allgemeinen sind die Bemessungsvorschriften aber so ausgelegt, daß unter Gebrauchslasten die Elastizitätsgrenze nicht erreicht wird.

Aber für die Verwendung hochfester Baustähle im Hinblick auf den Entwurf von Baukonstruktionen müssen natürlich nicht nur die zulässigen Beanspruchungen des Werkstoffs selbst, sondern auch seiner Verbindungen festgelegt werden. Auf Grund zahlreicher Versuche hat man in /8/ die in den Bildern 11 und 12 wiedergegebenen zulässigen Spannungen für Verbindungen mit hochfesten Schrauben und Schweißnähten genormt, wobei die Werte für Zeile 1, Spalten 3 und 4 nach Bild 12 wahrscheinlich zu vorsichtig angesetzt wurden und demnächst nach oben korrigiert werden sollen.

	1		2	3	4	5	6
	Verbindungsart		Werkstoff des Bauteils				Maß- gebender Querschnitt
			St E 47		St E 70		
			Lastfall				
			H	HZ	H	HZ	
1	Hochfeste Schrauben	ohne Vorspannung	5300	6000	7000	7800	Schaft
2		Vorspannung P _V nach DAST-Richtl. 010 Tab. 9, Spalte 2	8400	9300	12300	13800	
3	Hochfeste Paß- Schrauben	ohne Vorspannung	5600	6200	8200	9200	Loch
4		Vorspannung P _V nach DAST-Richtl. 010 Tab. 9, Spalte 2	8400	9300	12300	13800	

Bild 11: Zulässige Spannungen für geschraubte hochfeste Verbindungen, nach /8/

	1	2	3	4	5	6
	Nahtart	Spannungsart	St E 47		St E 70	
			H	HZ	H	HZ
1	Stumpfnäht K-Naht mit Doppelkehlnaht	Druck und Biegedruck Zug und Biegezug ¹⁾	2800	3100	4100	4600
2	HV-Naht mit Kehlnaht	Zug und Biegezug				
3	HV-Stegnaht mit Kehlnaht Kehlnähte	Druck und Biegedruck Zug und Biegezug Vergleichswert ²⁾ Schub				
4	alle Nahtarten	Schub in Nahtichtung	2000	2300	2200	2500

¹⁾ Nahtgüte, d. h. Freiheit von Rissen, Binde- und Wurzelfehlern und Einschlüssen, ausgenommen vereinzelte und unbedeutende Schlackeneinschlüsse und Poren, ist mit zerstörungsfreien Prüfverfahren nachzuweisen.

²⁾ $\sigma_V = \sqrt{\sigma^2 + \tau^2 + \tau^2}$

Bild 12: Zulässige Spannungen für geschweißte hochfeste Verbindungen, nach /8/

3.2 Elastizität

Es ist bekannt, daß der Elastizitätsmodul niedriglegierter hochfester Stähle sich praktisch nicht von dem der klassischen Stähle niedriger Festigkeit unterscheidet. Auch die Behandlungsart: normalisiert gegläht, walztemperaturgeregelt oder vergütet, hat keinen Einfluß auf den E-Modul. Das heißt: Stahlkonstruktionen aus hochfesten Stählen sind bei Ausnutzung der hohen Festigkeit relativ verformungsnachgiebiger. Diese Eigenschaft kann sich sowohl positiv als auch negativ auswirken. Negativ muß die geringere Verfor-

mungssteifigkeit bewertet werden z. B. bei Geschoßstützen, deren elastische Stauchung unter Gebrauchslasten beim Anschluß der Decken und der Deckenunterzüge an einem steifen, weniger kompressiblen Stahlbetonkern berücksichtigt werden muß. Auch die größere Biegeweichheit von Deckenträgern aus hochfesten Stählen kann besondere Maßnahmen erforderlich machen, z. B. die Ausnützung des kraftschlüssigen Verbundes mit der Betonplatte. Bei Schrägseilbrücken mit Pylonen aus hochfestem Stahl ist deren geringere Biegesteifigkeit hingegen positiv zu bewerten, da sie zu einer stärkeren Lastabtragung über Normalkräfte und somit zu einer größeren Gewichtsersparnis führt. Selbst wenn der Werkstoffaufpreis für hochfesten Stahl größer ist als das Verhältnis seiner Streckgrenze zu der des Vergleichsstahls, kann sich dann noch eine Kostenverbilligung ergeben.

3.3 Plastizität

Im Zugversuch zeigen hochfeste Stähle ein ähnlich elastisch-plastisches Werkstoffverhalten mit ausgeprägter Fließlinie wie die nicht legierten Baustähle (Bild 13). Die Bruchdehnung liegt zwischen 16 % und 25 %. Die Gleichmaß- und Einschnürdehnungen sind zwar geringer als bei den unlegierten Stählen niedriger Festigkeit, haben aber deren Charakteristik. Hieraus wäre eigentlich schon zu schließen, daß auch die hochfesten Baustähle trotz ihres geringeren Streckgrenzenverhältnisses ($\frac{\text{Streckgrenze}}{\text{Zugfestigkeit}}$)

nach der Plastizitätstheorie ^{x)} (plastic design) bemessen werden können. Diese Auffassung wird durch Traglastversuche mit hochfesten Baustählen bestätigt. Sowohl Knickversuche, worauf im nächsten Kapitel noch eingegangen wird, als auch Biegeversuche /13/ mit geschweißten verschieden ausgesteiften Vollwandträgern (Bild 14) und Traglastversuche mit Rahmenträgern (Vierendeel) /14/ (Bild 15) lassen den Schluß zu, daß die Verformungseigenschaften der hochfesten Stähle im plastischen Bereich ausreichen, um die Traglast mit Hilfe der Plastizitätstheorie oder auch mit der vereinfachten Fließgelenkhypothese ermitteln zu können.

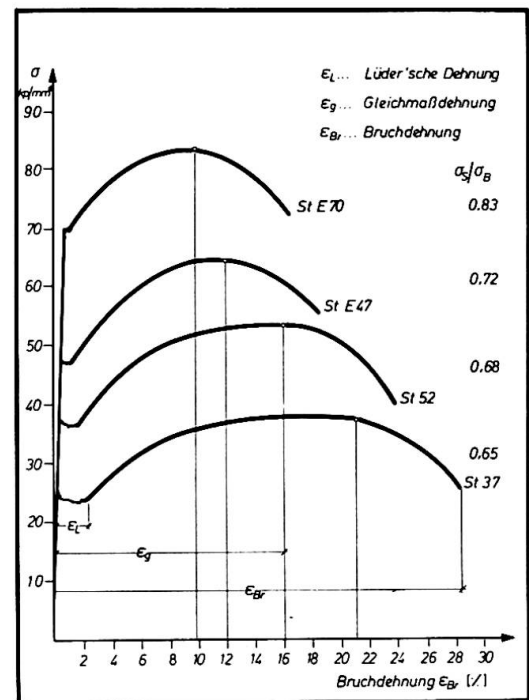
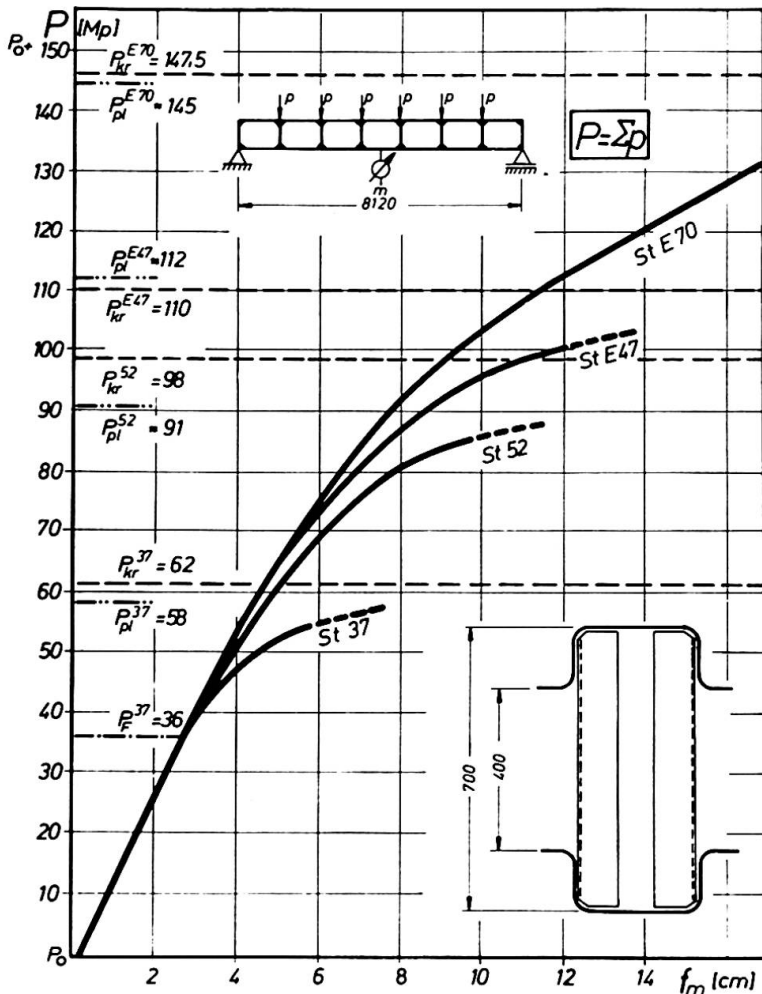


Bild 13: Werkstoffkennlinien von Baustählen verschiedener Festigkeit

Test	α	$P_u^{ex}(k)$	Simply Supported		Clamped	
			$P_u^{th}(k)$	P_u^{ex}/P_u^{th}	$P_u^{th}(k)$	P_u^{ex}/P_u^{th}
H1-T1	3.0	1,260	946	1.33	1,124	1.12
H1-T2	1.5	1,538	1,420	1.08	1,554	0.99
H2-T1	1.0	1,834	1,750	1.05	1,822	1.01
H2-T2	0.5	2,250	2,296	0.98	2,306	0.98

Bild 14: Vergleich theoret. u. experim. Traglasten von Vollwandträgern aus St E 70 mit unterschiedl. Quersteifenabständen (α), nach /13/

^{x)} Mit Plastizitätstheorie ist ein Berechnungsverfahren (Traglastbemessung) gemeint, das davon ausgeht, daß die einzelnen Tragglieder in der Lage sind, die ihrem Querschnitt entsprechenden vollplastischen Schnittgrößen aufzunehmen und während des weiteren Verformungsvorgangs konstant zu halten



Die Traglast P_{Kr}^{E47} für den Rahmenträger aus St E 47 war begrenzt durch Riß einer schlecht ausgeführten Schweißnaht. Sie hätte sonst vermutlich um 10 % höher gelegen. Britische Versuche an Stockwerkrahmen aus Stahl der Klassifikation BS 4360 Grade 50 (St 52) /15/ bestätigen die Anwendbarkeit der Plastizitätstheorie auch für höherfeste Baustähle und empfehlen ihre Zulassung /16/ bis Stahl nach BS 4360 Grade 55 (St E 47).

Bild 15: Vergleich theoret. u. experim. Traglasten von Rahmenträgern verschied. Baustahlfestigkeiten, nach /14/

3.4 Stabilität

3.4.1 Knicken

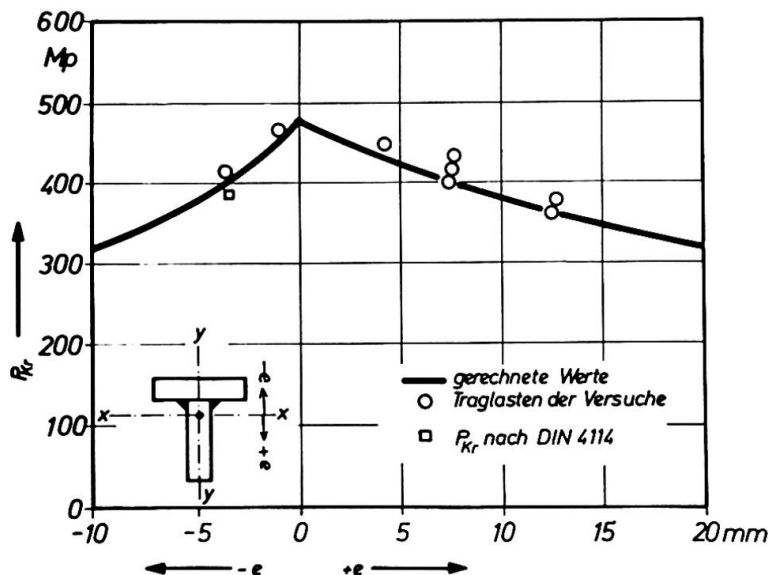


Bild 16: Traglasten für St E 70 in Abhängigkeit der Außermitteung e , nach /17/

Knickversuche im plastischen Bereich für St E 70 /17/ /18/ haben gezeigt, daß die Traglastbemessung auch für Druckstäbe aus höherfesten Stählen anwendbar ist. Bild 16 zeigt die gute Übereinstimmung zwischen Theorie und Experiment in Abhängigkeit verschiedener Außermitteigkeiten für St E 70. Ferner wurde gezeigt, daß die nach der deutschen Stabilitätsvorschrift DIN 4114 gewählte ungewollte Außermitteung

$$u = \frac{i}{20} + \frac{l}{500}$$

(die so groß angesetzt wurde, weil sie auch den Einfluß von Eigenspannungen mit abdecken sollte) ebenfalls gut mit dem ent-

sprechenden Versuchswert übereinstimmt. Überträgt man dieses Ergebnis auch auf die Stähle der Festigkeitsklasse St E 47, was wegen des gleichartigen

Druckspannungs-Stauchungsverhaltens wohl zulässig ist, dann lassen sich die kritischen Knickspannungslinien für die beiden hochfesten Stähle theoretisch ableiten (Bild 17).

Wenn nach Ausarbeitung der Europäischen Stabilitätsvorschriften in Zukunft für Knickstäbe die geometrische Imperfektion kleiner, nämlich $u = \frac{l}{1000}$ (für den beidseitig gelenkig gelagerten Einfeldstab der Länge l) gewählt wird (wobei man dann jedoch den Einfluß von Eigenspannungen gesondert berücksichtigt), und wenn außerdem die Querschnittsabhängigkeit durch mehrere profilabhängige Knickspannungslinien differenzierter zum Ausdruck kommt, so ändert dies nichts am Grundsätzlichen der bisher schon für das Knicken angewandten Plastizitätstheorie (auch nicht bei Ausdehnung auf hochfeste Stähle), sondern bedeutet lediglich eine weitere Zuschärfung. Die gute Übereinstimmung zwischen den von der "Europäischen Konvention für Stahlbau" (EKS, ECCS, CECM) vorgeschlagenen Knickspannungslinien, die vorzugsweise mit Versuchskörpern aus St 37 experimentell überprüft wurden, und einzelnen Versuchsergebnissen mit Prüfstäben aus höherfesten Stählen zeigen die Bilder 18 und 19.

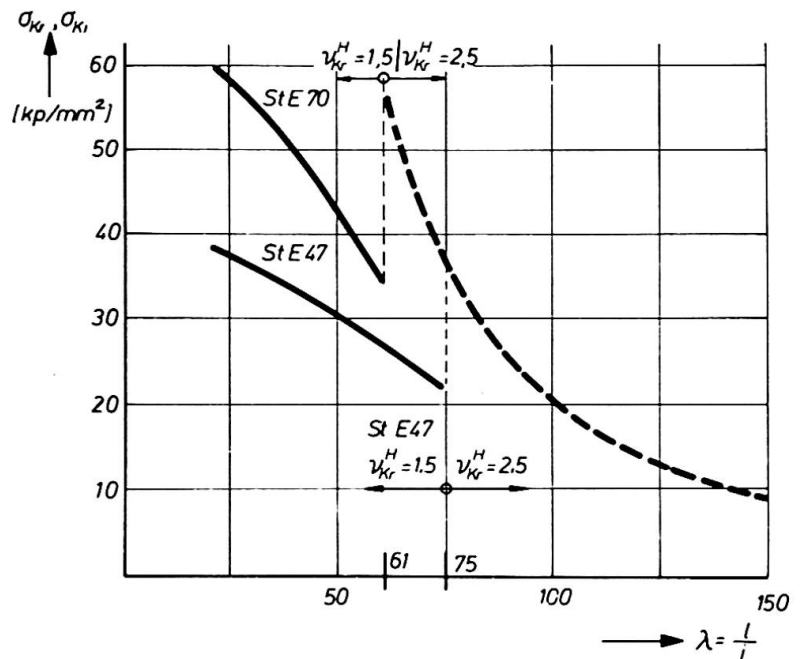


Bild 17: Knickspannungen σ_{Kr} (Traglast) für hochfeste Baustähle, nach /8/ u. DIN 4114

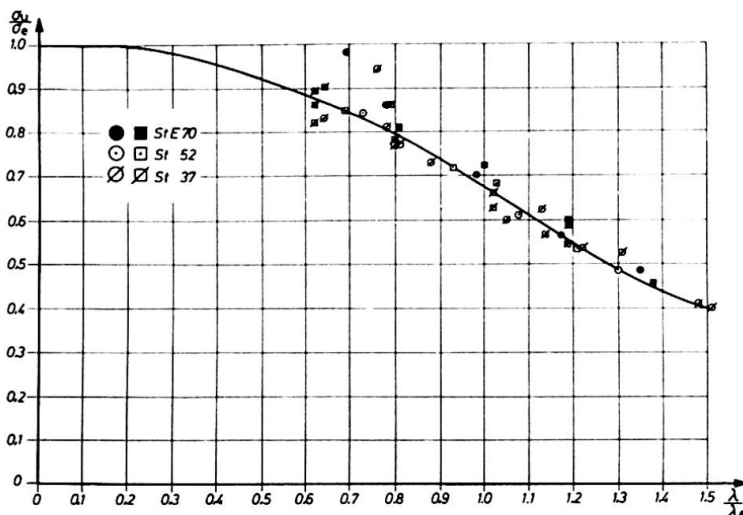


Bild 18: Vergleich der Europäischen Knickspannungslinie mit Versuchswerten (○ u. □ -Querschnitte), nach /19/

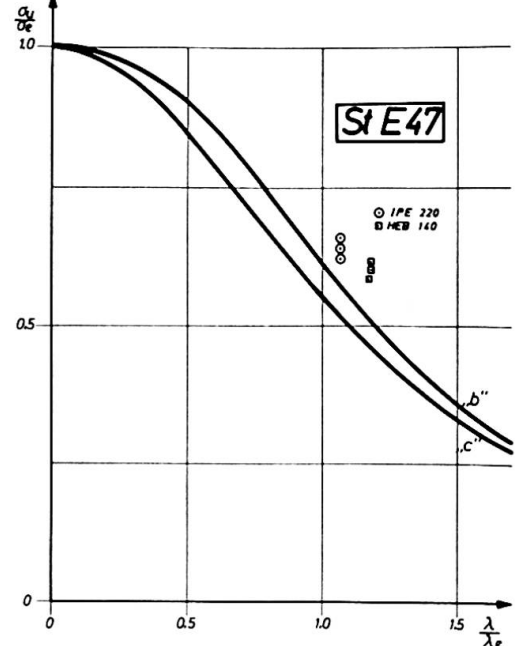
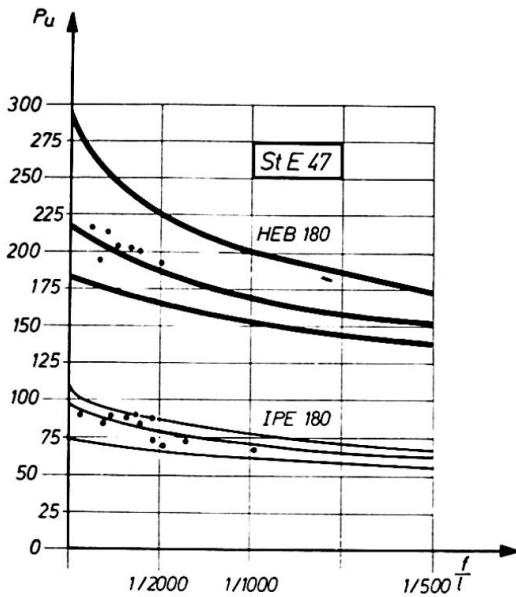


Bild 19: Vergleich d. Europäischen Knickspannungslinien für IPE- (Kurve "c") u. HEB-Profile (Kurve "b") mit Versuchswerten (Stahl St E 47), nach /20/



Die gute Erfassung geometrischer (Stabkrümmheit f/ℓ) und werkstofflicher Imperfektionen (Eigenspannungen) durch die den Europäischen Knickspannungslinien zugrundeliegenden Traglastberechnungen von Beer/Schulz auch bei hochfesten Stählen zeigt Bild 20. Die jeweils oberen Kurven gelten für das Verhältnis maximale Eigenspannung zu Streckgrenze $\bar{\sigma}_R = 0$, die mittleren für $\bar{\sigma}_R = 0,3$ und die unteren für $\bar{\sigma}_R = 0,5$. Hinsichtlich der Eigenspannungen ist ganz allgemein zu bemerken, daß diese für hochfeste Stähle, bezogen auf die Streckgrenze, im Schrifttum /13/ niedriger angegeben werden als bei den Baustählen St 37 und St 52 (was auch ohne weiteres einsehbar ist, da bei vergleichbaren Querschnitten infolge des Walz- oder Schweißvorgangs vergleichbare Temperaturverhältnisse und Abkühlbedingungen vorliegen).

Bild 20: Einfluß der Größe von σ_{vor} von Imperfektionen (Stabkrummheit f/l u. Eigenspannungen) auf die Knickspannungen u. Vergleich mit Versuchswerten (Stahl St E 43), nach [21]

3.4.2 Biegedrillknicken

Biegedrillknickversuche im elastischen und plastischen Bereich haben gezeigt, daß bezüglich des Stabilitätsverhaltens kein grundsätzlicher Unterschied zwischen den bisherigen Baustählen St 37, St 52 und dem hochfesten Stahl St E 47 (BS - Grade 55) besteht. Der Vergleich (Bild 21) der gemäß /22/ experimentell

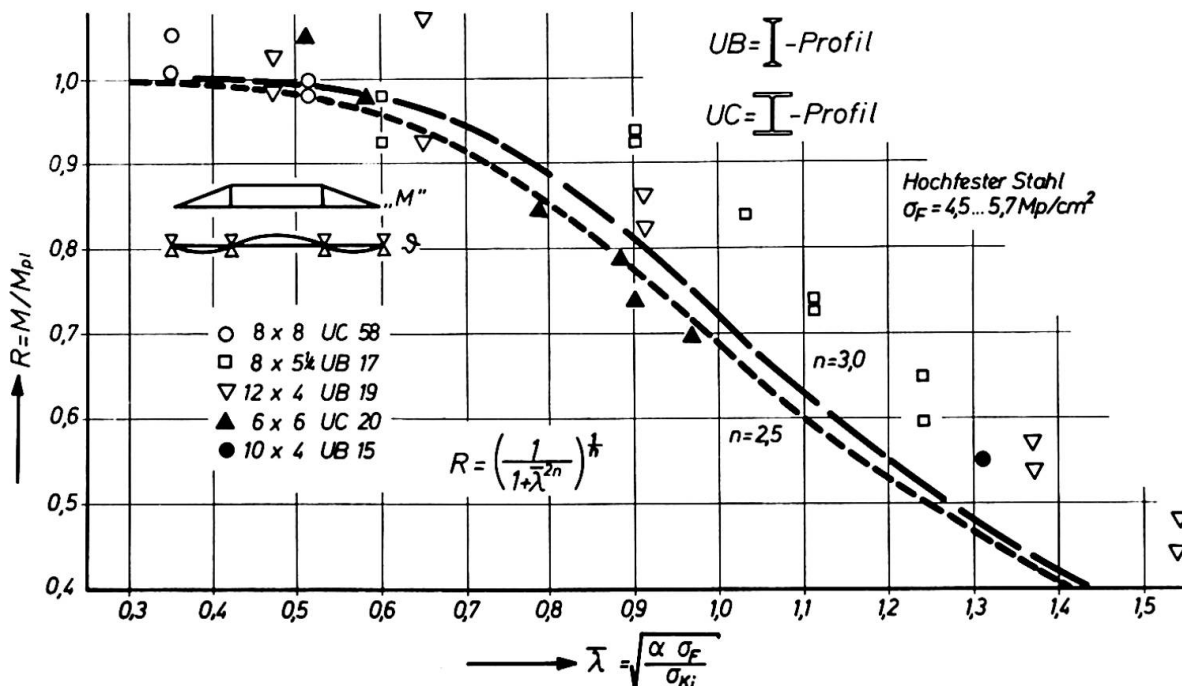


Bild 21: Vergleich zwischen der Näherungsformel für Biegedrillknicken (Empfehlungen der EKS) und den Versuchen, nach /22/

ermittelten Traglasten mit einer (z. Zt. für die Neubearbeitung der deutschen Stabilitätsnorm DIN 4114 und die Empfehlungen der EKS zur Diskussion stehenden) Näherungsformel zur Traglastberechnung für Biegedrillknickbeanspruchung, die anhand von Versuchen für St 37 und Kontrollrechnungen für St 37 und St 52 erarbeitet wurde, bestätigen diese Feststellung.

3.4.3 Beulen

Beulversuche mit hochfesten Baustählen im plastischen Bereich sind dem Verfasser bisher nicht bekannt geworden. Im Rahmen eines Ende 1974 beginnenden experimentellen Forschungsprogramms des Deutschen Ausschusses für Stahlbau, bei dem u. a. auch gezielt der Einfluß geometrischer Imperfektionen und Eigenspannungen auf das Tragverhalten berücksichtigt werden soll, sind auch Versuchskörper aus hochfesten Stählen vorgesehen. In den Fällen, in denen es erlaubt ist, bei Beulberechnungen die idealisierte Verzweigungstheorie anzuwenden (vgl. aber /23/), kann, solange allgemein keine bessere Bemessungsmöglichkeit besteht, die Abminderung für plastisches Beulen bei Verwendung hochfester Baustähle analog zu den für die niedriger festen Baustähle geltenden Abminderungen vorgenommen werden. Bild 22 zeigt die abgeminderten Beulspannungen im plastischen Bereich für die Stähle St E 47 und St E 70 unter hilfsweiser Heranziehung der Engeßer'schen Knickspannungen /8/.

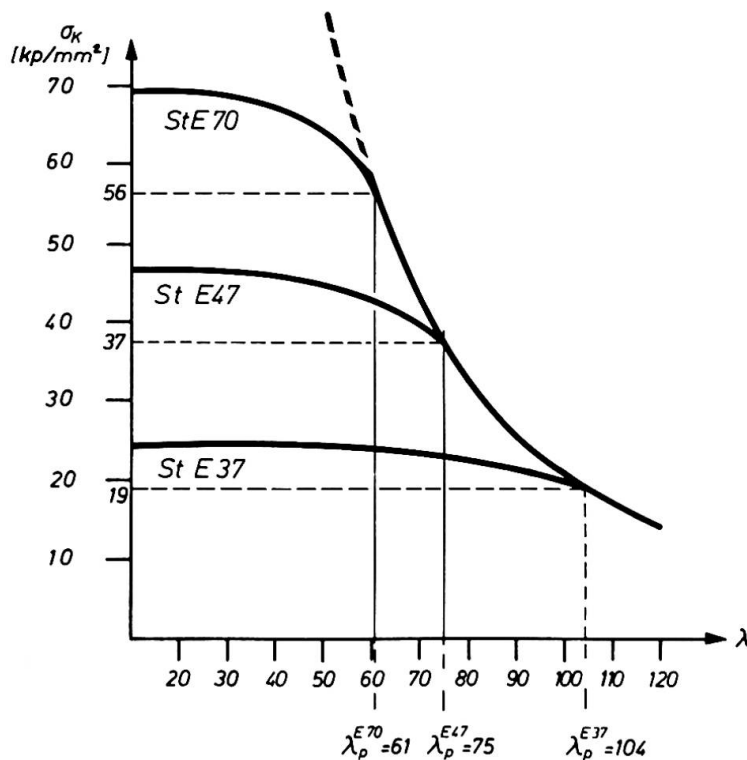


Bild 22: Abgeminderte Beulspannungen im plastischen Bereich für hochfeste Baustähle, nach /8/

3.5 Schwingfestigkeit

Die bisherigen Darlegungen haben gezeigt, daß sich die hohen Festigkeiten der hochfesten Baustähle unter den verschiedenen statischen Beanspruchungsarten im allgemeinen gut ausnützen lassen. Aus Dauerschwingversuchen mit Prüfkörper polierter Oberfläche (Bild 23) könnte man den Eindruck gewinnen, daß die hochfesten Baustähle demselben Dauerfestigkeitsgesetz (z. B. unter Wechselbeanspruchung

$$R = -1: \sigma_D^{\text{poliert}} = \frac{\sigma_B}{2}$$

unterliegen würden wie die Stähle niedrigerer Festigkeitsstufen. Leider zeigen aber Prüfstäbe mit Walzhaut (Bild 23) einen wesentlich geringeren Anstieg der Dauerfestigkeit in Abhängigkeit von der Zugfestigkeit /24/, was auf die höhere Kerbempfindlichkeit der hochfesten Stähle zurückzuführen ist. Dieser geringe Vorteil bei der Dauerfestigkeit hochfester Baustähle geht aber ganz verloren bei deren Verbindungen und Bauteilen /25/, so daß man vorschnell annehmen könnte, es sei unwirtschaftlich, hochfeste Stähle bei schwingender Beanspruchung einzusetzen. Bild 24 zeigt z. B. den starken Abfall der Dauerschwingfestigkeit ($N = 2 \cdot 10^6$ Lastspiele) unarbeiteter Stumpfnaht- und Kehlnahtschweißverbindungen aus St E 70 und deren Verbesserung durch besondere Oberflächenbearbeitung /26/.

Da die Konstruktionen des Hoch- und Brückenbaus aber in der Regel nicht im Dauerfestigkeitsbereich der Einstufenbeanspruchung des Wöhler-Versuchs unterliegen, kommt der Zeit- und Betriebsfestigkeit von Konstruktionsteilen aus hochfesten Baustählen weit mehr Bedeutung zu /27/. Allerdings ist dann eine bessere Kenntnis der Belastungsgeschichte eines Bauwerkes, die zu für die Betriebsfestigkeit maßgebenden Lastkollektiven ausgewertet werden kann, erforderlich. Bild 25 zeigt das wesentlich günstigere

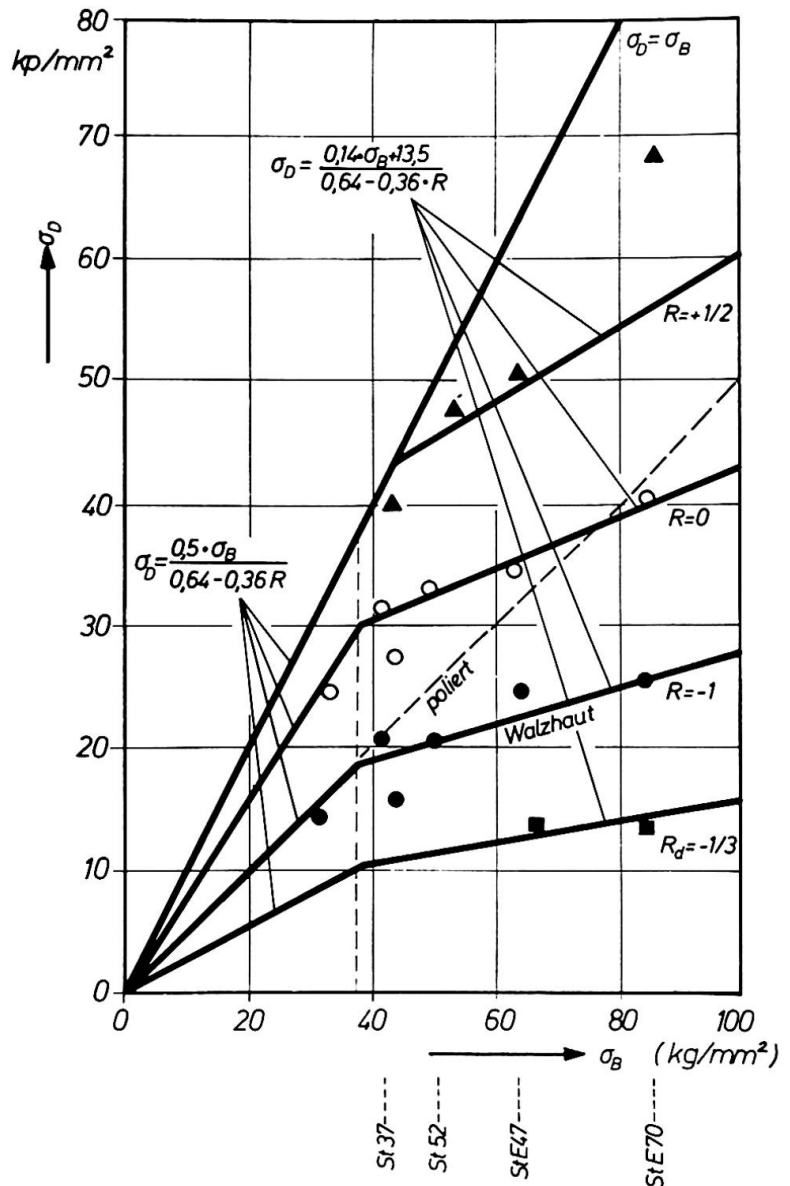


Bild 23: Dauerschwingfestigkeit σ_D (poliert und Walzhaut) abhängig von statischer Zugfestigkeit σ_B und Spannungsverhältnis R , nach /24/

Verhalten der hochfesten Stähle im Zeitfestigkeitsbereich ($N = 10^5$ Lastspiele).

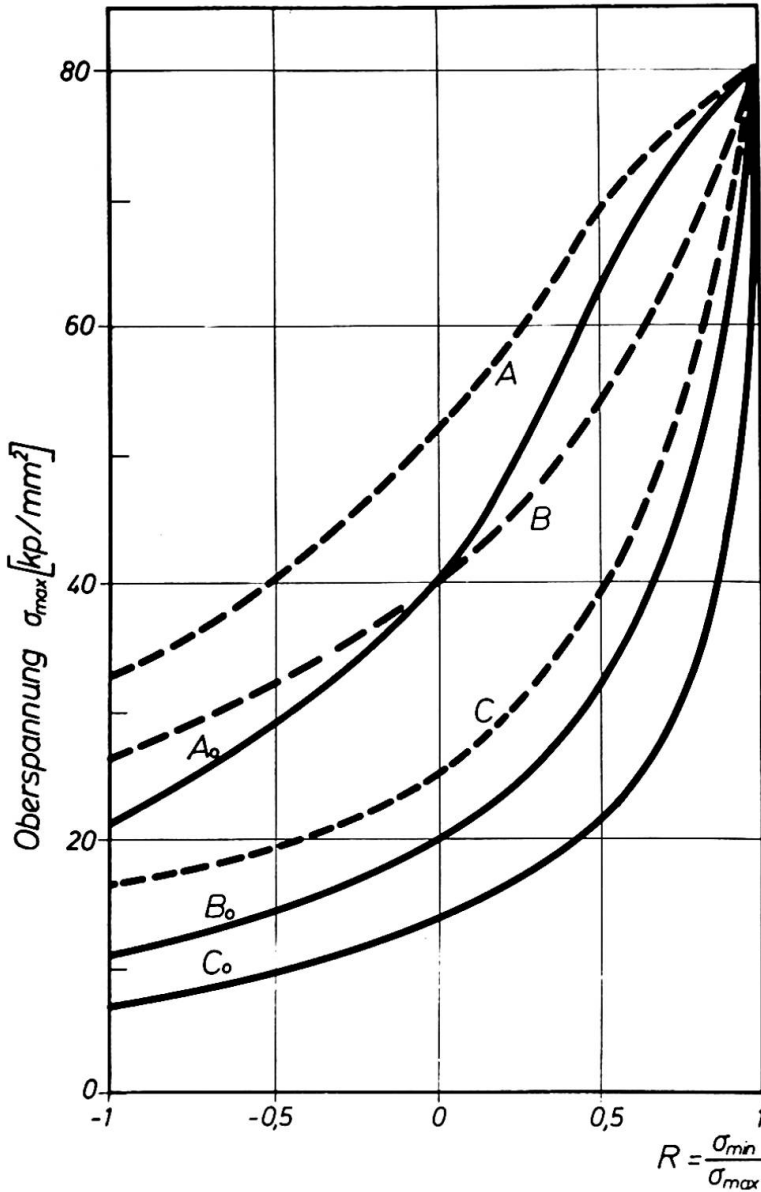


Bild 24:

Dauerschwingfestigkeit ($N = 2 \cdot 10^6$) des St E 70 abhängig von der Art der Schweißverbindung und deren Bearbeitung, nach /26/

- A_0 Grundwerkstoff, Walzhaut
- A Grundwerkstoff, feingeschliffen
- B_0 Schweißverbindung (V-Naht), unbearbeitet
- B Schweißverbindung (V-Naht), blechenbearbeitet
- C_0 beidseitig angeschweißte Quersteifen, Doppelkehlnaht, unbearbeitet
- C beidseitig angeschweißte Quersteifen, Doppelkehlnaht, bearbeitet

	Dauerfestigkeit $\sigma_{D,N=2 \cdot 10^6}$				Zeitfestigkeit $\sigma_{Z,N=10^5}$				Zugfestigkeit
$\kappa =$	-1/3	-1	0	+1/2	-1/3	-1	0	+1/2	+1
St37 normalisiert	-	16,5	26,5	40,0	-	21,5	33,0	-	43,0
St52 normalisiert	-	20,5	33,0	46,5	-	-	-	-	53,0
St E 47	-39,8	24,1	34,7	50,4	-56,0	33,0	47,3	58,7	63,8
St E 70	-41,0	25,6	40,1	68,7	-68,1	41,3	64,3	80,0	85,1

Bild 25: Vergleich zwischen Dauerfestigkeit, Zeitfestigkeit und Zugfestigkeit verschiedener Baustahlfestigkeiten, nach /24/

Die Auswertung dieser und weiterer Versuche führte schließlich zu Vorschlägen im Rahmen der Erweiterung der DASt-Richtlinie 011 /8/ für schwingende Beanspruchung unter Berücksichtigung verschiedener Lastkollektive. Im Sinne eines solchen Betriebsfestigkeitsnachweises werden künftig schwingbelastete Bauteile und Verbindungen aus hochfesten Stählen auf der Grundlage idealisierter bezogener Wöhler- und Lebensdauerlinien bemessen (Bild 26).

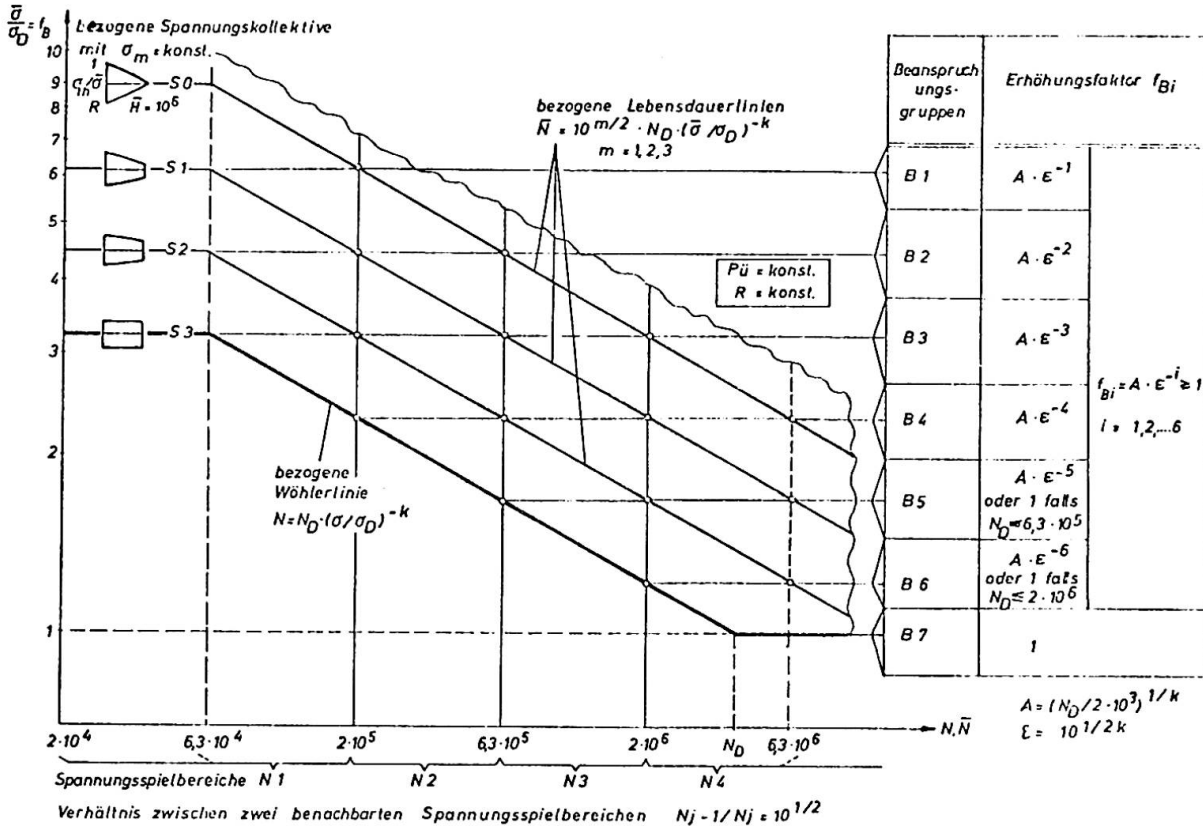


Bild 26: Betriebsfestigkeit nach Beanspruchungsgruppen B 1 bis B 7 abhängig von Spannungsspielbereichen R und Spannungskollektiven S (St E 70), nach /48/

Die Bauteile und Verbindungen sind in Kerbgruppen eingeteilt, für die in Abhängigkeit von der Stahlsorte jeweils einheitliche Grenzlasterlastspielzahlen N_D , Wöhlerlinienneigungen k und vom Spannungsverhältnis $R = \min \sigma / \max \sigma$ abhängige vorläufige Oberspannungsdauerfestigkeiten σ_D festgelegt sind.

Den in doppeltlogarithmischer Auftragsweise parallel zu den Wöhlerlinien und untereinander in gleichen Abständen verlaufenden Lebensdauerlinien (einschließlich der Wöhlerlinie vier Lebensdauerlinien) sind vier genormte Belastungskollektive S 0 bis S 4 mit unterschiedlichen Völligkeitsgraden zugeordnet. Aus Kombinationen der vier eingeführten Lastspielbereiche mit den vier genannten Belastungskollektiven entstehen sieben Beanspruchungsgruppen B 1 bis B 7 mit jeweils einheitlichen Dauerfestigkeits-Erhöpfungsfaktoren $f_B = \sigma / \sigma_D = \text{zul } \sigma / \text{zul } \sigma_D$, die Werte zwischen $f_B = 1$ und etwa $f_B = 8$ einnehmen können.

Aus diesem Bemessungsschema wird ersichtlich, daß die hochfesten Stähle den herkömmlichen Baustählen St 37 und St 52 dann überlegen sind, wenn hohe Oberspannungsdauerfestigkeiten σ_D , d. h. geringe Kerbwirkungen und hohe Spannungsverhältnisse R , und hohe Werte für die Erhöhungsfaktoren f_B ,

d.h. geringe Lastspielzahlen und geringe Völligkeitsgrade der Belastungskollektive, vorliegen. Diese Bedingungen führen für die hochfesten Stähle zu zulässigen Spannungen σ , die von den niedrigfesten Stählen wegen der oberen, durch die statische Belastung gegebenen und sich an der Streckgrenze orientierenden Grenzspannung σ_{stat} nicht mehr erreicht werden können.

4 Möglichkeiten und Grenzen der Anwendung im Hochbau

4.1 Entwurfsüberlegungen für den Geschoßbau

Für Geschoßbauten, in denen hohe Streckenlasten für Unterzüge und große Einzellasten für Stützen abzutragen sind, bestehen gute Einsatzmöglichkeiten für hochfeste Baustähle, sofern Instabilitäten im elastischen Bereich auszuschließen sind und die größere Verformungsnachgiebigkeit hochfester Stähle durch konstruktive Maßnahmen ausgeglichen werden kann. So wurde hochfester Stahl u. a. verwendet beim IBM-Building in Pittsburgh (USA) und beim World Trade Center in New York (USA).

4.1.1 Geschoßdecken

Aus Gründen der Sicherheit, der Gebrauchseignung und der Wirtschaftlichkeit sollten Geschoßdecken im modernen Stahlgeschoßbau in der Regel unter statischer Mitwirkung der Betondecke ausgeführt werden, wobei sowohl der Profilträger- als auch der Profilblechverbund in Betracht kommen. Vergleichsrechnungen haben gezeigt, daß im üblichen Bereich der Verkehrslasten ($p = 2 \text{ kN/m}^2$ bis 15 kN/m^2) und der Spannweiten ($l = 4,8 \text{ m}$ bis 12 m) unter Beachtung der für die Gebrauchseignung zweckmäßigen Durchbiegungsbegrenzung ($f = \frac{l}{300}$ bis $\frac{l}{500}$) die niedrigerfesten Stähle St 37 und St 52 im allgemeinen ausreichen. Da aus bauphysikalischen Gründen (Schall- und Brandschutz) die Dicke der Betonplatte im allgemeinen 10 cm nicht unterschreiten sollte, können die zulässigen Betondruckspannungen kaum ausgenutzt werden, so daß für Decken mit und ohne Trägerverbund sowohl höherfeste Betone als auch hochfeste Stähle nur selten in Frage kommen. Die bei Profilblechverbunddecken durch Walzprofilierung der Bleche erzielbare Kaltverfestigung mit Streckgrenzenwerten bis ca. 350 N/mm^2 sollte aber durchaus genutzt werden. Da der reine Haftverbund unter der Voraussetzung feiner Risseverteilung bis zur Traglast nicht zu gewährleisten ist, kommt der Forschung für eine wirtschaftliche Verbundsicherung mit mechanischen Verdübelungselementen für Profilbleche besondere Bedeutung zu.

4.1.2 Unterzüge

Da die Anzahl der Stützen aus Gründen einer guten Nettoflächenausnutzung in Geschoßbauten gering gehalten werden soll, ergeben sich oft relativ große Spannweiten für die Unterzüge. Geringe Unterzugbauhöhen aus Gründen der Wirtschaftlichkeit des Gesamtgebäudes, große Öffnungen im Träger für Deckeninstallationen und große Spannweiten mit hohen Belastungen sind aber sich gegenseitig nachteilig beeinflussende Entwurfsparameter. Von den drei Konstruktionsmöglichkeiten für Geschoßbauunterzüge:

- Vollwandträger mit Öffnungen
- Fachwerkträger
- und Rahmenträger (Vierendeel)

verdient der letztere besondere Aufmerksamkeit im Hinblick auf den zu suchenden Entwurfskompromiß. Eine Versuchsreihe des Deutschen Ausschusses für Stahlbau /14/ bewies den Vorteil der "plastischen" Bemessung nach der Fließgelenkhypothese gegenüber der elastizitätstheoretischen Berechnung und deren volle Wirksamkeit auch für die hochfesten Stähle St E 47 und St E 70 (Bild 15). Auch wenn anstelle der kaltverformten Bleche, auf deren Obergurte aus Hutprofilen montagegerecht Stahlprofilblechverbundplatten direkt aufgelagert werden können, Profilstahl z. B. Breitflanschprofile verwendet werden, stimmt die Traglastbemessung mit der im Versuch erzielten Versagensbelastung gut überein, vorausgesetzt, daß die Vertikalstäbe ausreichend biegesteif an die Gurte angeschlossen sind.

Hochfeste Stähle können für durchbrochene Unterzüge bei großen Spannweiten, niedrigen Bauhöhen und hohen Belastungen dann wirtschaftlich eingesetzt werden, wenn die Verbundwirkung der quer angeschlossenen Verbunddecken, die gleichzeitig die horizontale Stabilität der Unterzüge sichern, durch wirtschaftliche Verbundmittel (z. B. Kopfbolzendübel) auch in Unterzugrichtung miterwirkt wird. Dann ist auch bei Verwendung hochfester Stähle eine ausreichende Steifigkeit zu erwarten.

Im Kraftwerkbau müssen durch die laufende und noch nicht abgeschlossene Leistungssteigerung immer größere und schwerere Lasten aus den Dampferzeugern und Turbinen von den Stahlskeletten übernommen werden. Bild 27 zeigt die Deckenträger und Unterzüge eines Kesselgerüsts aus hochfestem Stahl St E 47, für die die niedrigerfesten Baustähle wegen ihrer geringeren zulässigen Spannungen nicht mehr ausgereicht hätten.

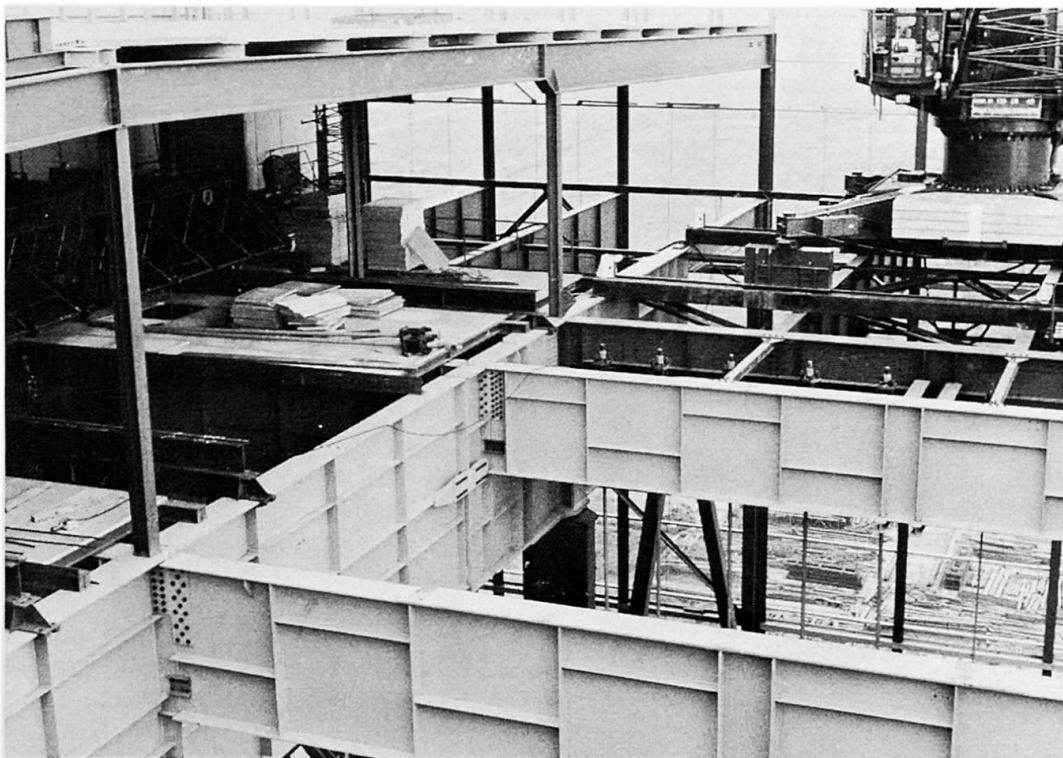


Bild 27: Unterzüge und Deckenträger eines Kraftwerk-Kesselgerüsts aus St E 47

4.1.3 Stützen

Bei Hochhäusern mit großer Druckbeanspruchung für die Stützen ist besonders der aus Blechen geschweißte Hohlkastenquerschnitt geeignet. Da aus Gründen des Nutzflächengewinns möglichst kleine Stützenquerschnitte erwünscht sind und die steifen Geschoßdecken-scheiben die Knicklänge auf die Geschoßhöhe begrenzen, liegt die Schlankheit der Stützen im plastischen Bemessungsbereich, wodurch der Einsatz hochfester Stähle voll zur Wirkung kommt. Bild 28 zeigt den

Vorteil hinsichtlich des Nutzflächengewinns hochfester Stähle gegenüber den herkömmlichen

Baustählen und vor allem gegenüber Stahlbetonstützen besonders bei hohen Stützenlasten, wobei ein Verhältnis Blechdicke zu Seitenlänge von 1 : 10 unterstellt ist. Aber wie bei den auf Biegung beanspruchten Deckenträgern und Unterzügen aus hochfesten Stählen besonders auf die Verformung zu achten ist, gilt dies auch für die relativ große Stauchung hochfester Stützen im Verhältnis zu den wesentlich geringeren Betonstauchungen massiver Hochhauskerne /28/. Außerdem ist nicht nur der Anfangszustand, sondern auch der zeitliche Verlauf der unterschiedlichen Normalkraftverformungen von Stahl- und Betontragwerken, die noch von der Verformung der Gründungsplatte überlagert werden, zu untersuchen. Der in Bild 29 dargestellte Stützenquerschnitt eines im Winter 1974/75 montierten Hochhauses in Frankfurt (Deutschland) zeigt den Nutzflächengewinn im Vergleich zu einem Stahlbetonquerschnitt gleicher Lastaufnahme.

Für Geschoßbauten gibt es bestimmte Bereiche häufig wiederkehrender Anforderungen aus

Raster
Spannweiten
Belastungen
Installationen
Brandschutz
Bauphysik usw.,

die man zu einem Anforderungsspektrum zusammenfassen kann. Die Vielzahl der Daten einer derart komplexen Bauaufgabe ist nur mit Hilfe eines Rechners im Hinblick auf die Optimierung des Entwurfs zu verarbeiten. Das Ergebnis einer solchen Entwurfsop-

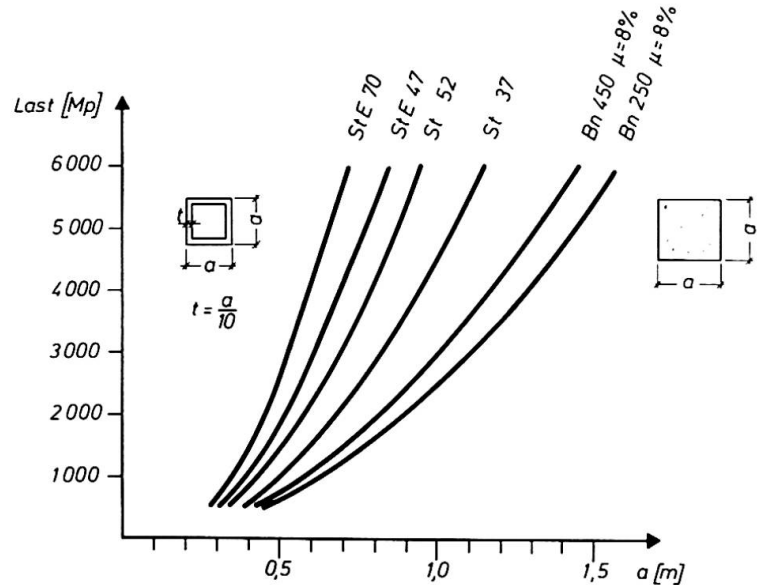
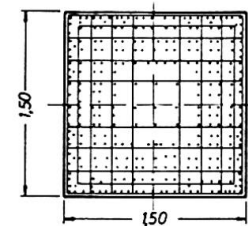


Bild 28: Stützenmaße abhängig von der Last bei verschiedenen Werkstoffen nach /28/

Stahlbetonstütze

Bn 450
294 # 28
8% Fe

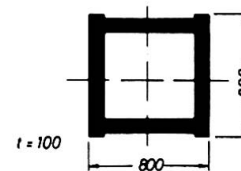


$Q_{zul} \approx 6700 \text{ Mp}$

Stahlstütze

St E 47

$\sigma_{zul} = 2800 \text{ kp/mm}^2$



$Q_{zul} \approx 6720 \text{ Mp}$

Bild 29: Querschnittsvergleich einer Stahlstütze aus St E 47 mit einer Stahlbetonstütze gleicher Lastaufnahme

mierung dient der Entwicklung von Konstruktionssystemen /29/.

$P_{zul} \hat{=}$	Stahlgüte			F	Nr.
MPa	St 37	St 52	St 43	cm ²	
200	I			204	1
250	I			226	2
300		I		210	3
350		I		226	4
400			I	218	5
450			I	256	6
500	I			450	7
550		I		358	8
600	I			511	9
650		I		397	10
700			I	369	11
750			I	367	12
800		I		511	13
900		I		552	14
1000			I	528	15
1100			I	552	16

HE - Sonderprofile

$h = 403 - 405 \text{ mm}$
 $b = 3015 - 336$



Bild 30: Stützensystem aus Breitflanschprofilen verschiedener Stahlfestigkeitsklassen

Für Hochhäuser bis etwa 20 Geschosse besteht eine gute Möglichkeit zur Entwicklung eines Stützensystems aus Breitflanschprofilen, die gegenüber Kastenquerschnitten keine Schweißarbeiten zur Profilverstellung benötigen und wegen ihres offenen Querschnitts einfache Anschlüsse für die Unterzüge ermöglichen. Unterstellt man einmal einen Stützenraster von 7,20 m x 7,20 m, dann ergibt sich bei einem Deckeneigengewicht von 5 kN/m² und einer Verkehrslast von ebenfalls 5 kN/m² eine Stützenlast je Geschos von ca. 500 kN. Bei 20 Geschossen erhält dann die unterste Stütze eine Last von 10 MN. Wählt man den Bereich der Profilhöhe $h = \text{ca. } 403 - 405 \text{ mm}$ und der Flanschbreite $b = \text{ca. } 300 - 340 \text{ mm}$ und läßt als Variation verschiedene Flansch- und Stegdicken sowie 3 Festigkeitsklassen zu, dann kann man unter diesen Bedingungen ein Stützensystem entwerfen (Bild 30), das bei nahezu konstantem Stützenumriß für 20-geschossige Hochhäuser dank der hochfesten Stähle geeignet ist. Die geringen Differenzen der Flanschbreite können mit der Befestigungs- und Brandschutzummantelung ausgeglichen werden.

4.2 Entwurfsüberlegungen für räumliche Systeme

Die Möglichkeit, mit Hilfe elektronischer Rechenautomaten fast jeden Beanspruchungszustand auch komplizierter statischer Systeme mindestens im Bereich der Elastizitätstheorie berechnen zu können, wirkt sich auch auf den konstruktiven Entwurf aus, um in gezielter Weise Kräfte durch Raumtragwerke besser verteilen und ableiten zu können. Die räumliche Kräfteverteilung führt zwar zu stark unterschiedlichen Schnittkraftbeanspruchungen. Andererseits verlangt aber nicht nur die architektonische Ästhetik räumlicher Tragwerke eine angenähert konstante Umrißgeometrie der Raumtragglieder, sondern auch die Wirtschaftlichkeit im Hinblick auf Fertigung und Montage.

Diesem Entwurfsparameter quasikonstanter Umrißgeometrie trotz unterschiedlicher Beanspruchung und vor allem auch im Hinblick auf standardisierte Kompaktanschlüsse kann durch zwei Entwurfsvariable Rechnung getragen werden: Durch die Kombination abgestufter Wanddicken und Festigkeiten. Man kann im Hinblick auf eine räumliche Tragwerkssystematik zwei Raumsysteme unterscheiden:

1. Das Normalkraft übertragende Raumfachwerk
2. Das biege- und torsionssteife Raumrostwerk

Raumfachwerke werden vorwiegend als Dachtragwerke verwendet, für die gegenüber Decken verhältnismäßig geringe Lasten vorgeschrieben,

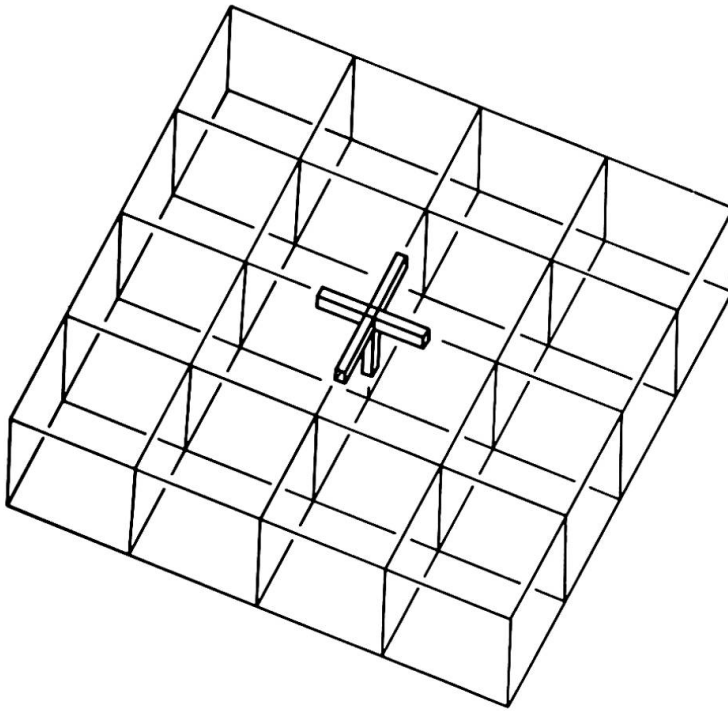
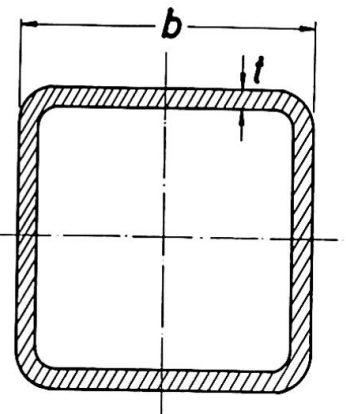


Bild 31: Raumrostwerk

Raumrostwerke, die nur orthogonal verlaufende Stäbe aufweisen, besitzen infolgedessen Knoten mit nur höchstens 6 Stabanschlüssen (Bild 31). Außer den Normalkräften müssen diese Knotenpunkte mit ihren Anschlüssen auch Querkkräfte, Biege- und Torsionsmomente übertragen können. Eine solche hochgradig statisch unbestimmte Konstruktion besitzt große Plastizitätsreserven, die bei einer "plastischen" Bemessung nach dem Traglastverfahren ausgenutzt werden können /30/. Bei hochbelasteten Geschoßdecken und relativ engem Raster können sich bei solchen Raumrostwerken durchaus Möglichkeiten für den Einsatz hochfester Stähle ergeben, insbesondere wenn aus wirtschaftlichen Gründen einer standardisierten Anschlußtechnik quasi-gleichbleibende Querschnitte gewählt werden sollen. In Bild 32 sind als Beispiel die möglichen Spannungsextremwerte aus der Überlagerung von N , Q_x , Q_y , M_x , M_y , und M_T dargestellt und verschiedenen Stahlfestigkeitsklassen zugeordnet. Bei der vorteilhaften Verwendung von Rechteckhohlprofilen empfiehlt sich das in Bild 33 angegebene Seitenlänge-Dickenverhältnis, um vorzeitiges Ausbeulen bei Anwendung des Traglastverfahrens auszuschließen.



St 37 $b/t = 34$
St 52 $b/t = 28$
StE47 $b/t = 22$
StE 70 $b/t = 18$

Bild 32: Seitenlänge-Dickenverhältnis für Rechteckhohlprofile

aber meist ausreichende Bauhöhen selbst bei großen Spannweiten zulässig sind. In der Regel reichen deshalb die niedrigeren Festigkeitsklassen des St 37 und St 52 für die Fachwerkstäbe aus, zumal die Druckstäbe meist im höheren Schlankheitsbereich liegen, für den ohnehin der Elastizitätsmodul und nicht die Festigkeit für die Bemessung maßgebend ist. In Sonderfällen kann es aber aus architektonischen Gründen vorteilhaft sein, an Knoten mit größeren Stabkraftunterschieden die höher beanspruchten Stäbe aus hochfestem Stahl herzustellen, um eine ausgeglichene Umrißgeometrie der Einzelstäbe zu erreichen.

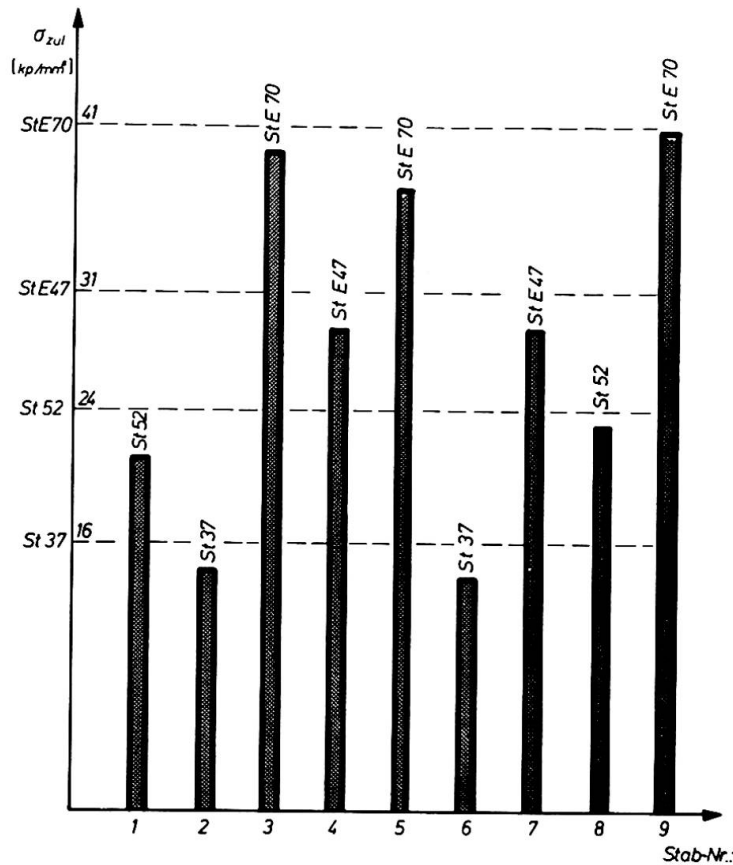


Bild 33: Geometrisch ähnliche Bemessung durch Variation der Festigkeit

4.3 Entwurfsüberlegungen für weitgespannte Dachtragwerke

Weitestgespannte Brücken- und Dachtragwerke sind nur unter Ausnutzung der allgemein technisch herstellbaren höchsten Zugfestigkeit, also durch Kaltverfestigung des Stahls möglich /31/. Die konstruktive Verwendung dieser hohen Zugfestigkeit wird durch Seile und Seilnetze mit Festigkeiten bis 1400 1800 N/mm² ermöglicht. Während des IVBH-Kongresses 1972 in Amsterdam ist ausführlich über Seilnetzkonstruktionen insbesondere über das Olympiaseildach von München berichtet worden /32/. Die Ergänzung der Anwendung hochfester Stähle im Hochbau durch die kaltverfestigten hochzugfesten Seiltragwerke soll, da die grundlegenden geometrischen, statischen, dynamischen und detailkonstruktiven Parameter vorerst als weitgehend geklärt angesehen werden können, das Augenmerk hier auf zwei bisherige Handicaps richten:

1. Gibt es überhaupt wirtschaftliche, d.h. kostengünstige Entwürfe für Seildachkonstruktionen?
2. Welches sind geeignete kostengünstige Dachhäute für Seilnetze?

Der Verfasser glaubt, daß ohne eine weitergehende Untersuchung in dieser Richtung - also der Kostensenkung - die Seilnetztechnik mit Hilfe hochzugfester Stähle auf wenige Einzelfälle für Ausstellungen und Sportmassenveranstaltungen beschränkt bleiben und kaum eine breitere Anwendung finden wird.

Welches wären die Möglichkeiten?

1. Verzicht auf die Unsymmetrie der "Zeltlandschaft" und Rückbesinnung auf die Wirtschaftlichkeit der Symmetrie (z. B. System Jawerth)
2. Kombination von Seilnetzen und biege- und torsionssteifen

Stabwerken (z. B. US-Pavillon 1958 Brüssel)

3. Verminderung der Fugen bei Verwendung von Dachtafeln durch abrollbare Bahnen mit wenigen und einfachen Dichtungen.

Diese Hinweise versteht der Verfasser lediglich als Denkanstöße in Richtung kostengünstigerer Entwürfe, die mehr die Handschrift des Ingenieurs als des Architekten verraten sollten, um eine Verbreiterung des Anwendungsbereiches hochzugfester, vorgespannter Seilnetzkonstruktionen zu ermöglichen.

5 Möglichkeiten und Grenzen der Anwendung im Brückenbau

5.1 Entwurfskonzepte für weitgespannte Balkenbrücken

In den USA sind schon Ende der 50er/Anfang der 60er Jahre hochfeste wasservergütete Baustähle für Großbrücken eingesetzt worden, so z. B. für die Carquinez-Strait-Brücke (Fachwerkbalken), Louisville - New Albany-Brücke (Fachwerkbogen mit Zugband), Benicia-Martinez- und die Whiskey-Creek-Brücke (Vollwandbalken) /33/ /34/. Bei der 1974 errichteten Osaka Port Brücke (Japan) mit 510 m eine der weitestgespannten Fachwerkbrücken der Welt wurden für die Gurte, die aus geschweißten Kastenquerschnitten 1400 mm breit und 1800 mm hoch mit Blechdicken zwischen 30 und 75 mm bestehen, die beiden hochfesten vergüteten Baustähle HT 70 und HT 80 mit Streckgrenzen von 600 bis 700 N/mm² verwendet /35/. Der Vorteil des Einsatzes hochfester Stähle darf allerdings nicht nur vorwiegend in der Gewichtsverminderung gesehen werden, sondern im Zusammenwirken mit weiteren Entwurfsparametern, z. B. :

1. Beibehaltung quasi-gleichbleibender Querschnittsabmessungen trotz wachsender Beanspruchung durch Übergang zu höherer Werkstoff-Festigkeit
2. Reduzierung der Schraub- und Nietanschlüsse durch Anschweißen von Stabenden aus höherfestem Stahl - Stoßlaschen auch aus hochfestem Stahl
3. Verminderung der Untergurtlamellen bei offenen Hutquerschnitten mit voll mitwirkender Stahlrippen- oder Stahlbetonfahrbahn
4. Erhöhung der Beulstabilität bei gedrungener Blechschlankheit.

Bild 34 zeigt die Ansicht und den Querschnitt der 1962 fertiggestellten Rheinbrücke Wiesbaden-Schierstein, die in St 52 ausgeführt wurde und drei Vergleiche für die Ausführung des Untergurtbereichs in St 52 oder in St 52/St E 47 oder in St 52/St E 70. Das Wesentliche des Vergleichs ist nicht die Gewichtsersparnis, sondern die kostengünstigere Querschnittsverminderung des Untergurtes. Eine Stahl-Verbundbrücke (Bild 35) mit Hybridträgern, deren Untergurt aus hochfestem Stahl besteht, ist für mittlere Spannweiten-



Bild 35: Stahlverbundbrücke mit Hybridträgern aus hochfestem Baustahl

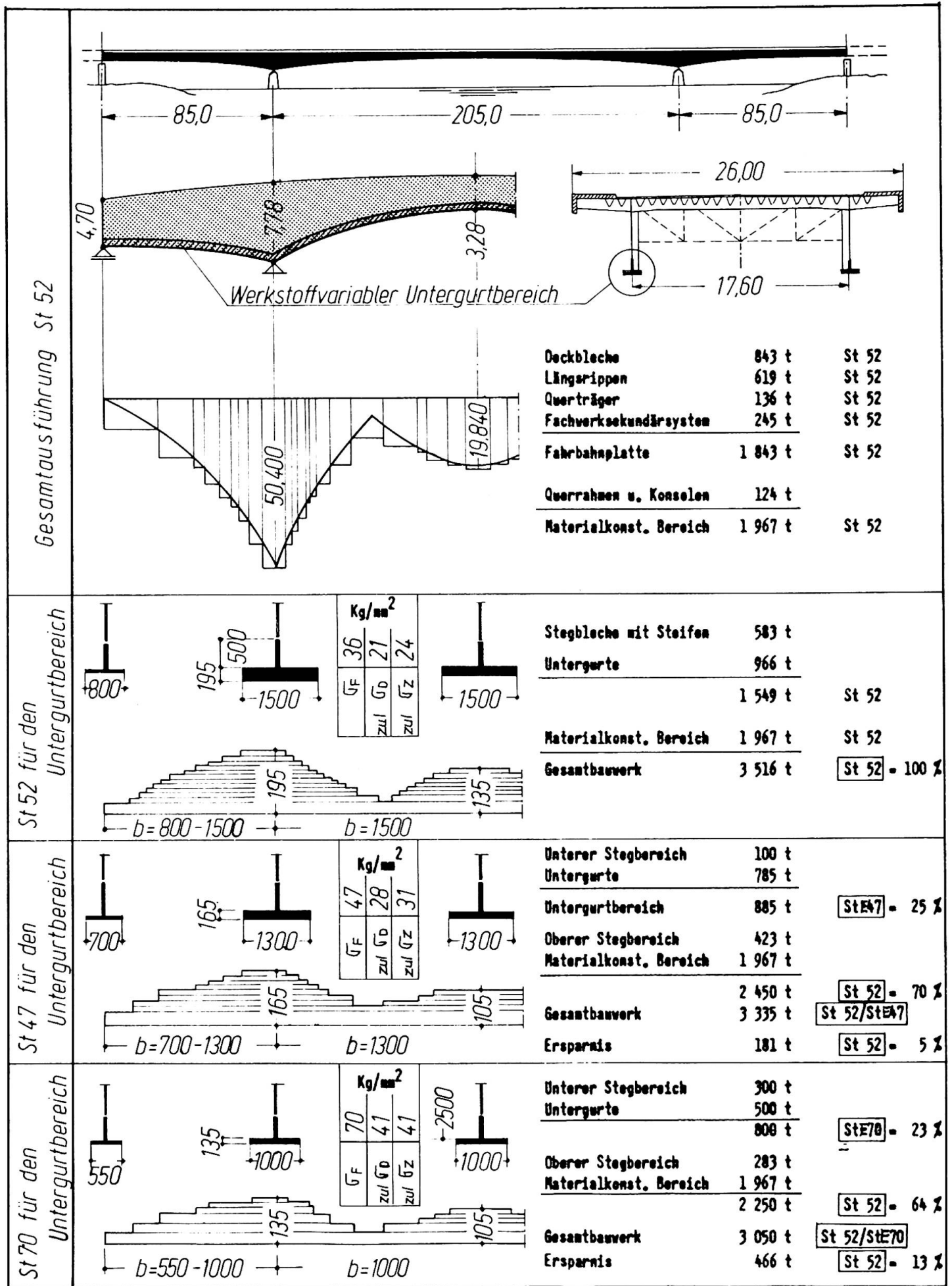


Bild 34: Vergleichende Untersuchung über die Anwendung hochfester Baustähle am Beispiel der Rheinbrücke Wiesbaden-Schierstein

bereiche (50-60 m) konzipiert /36/. Die Hybridträger bestehen aus miteinander längsverschweißten leichten (St 37) und schweren (St E 43) halbierten Breitflanschträgern. Die so gebildeten schmalen dünnwandigen Obergurte niedriger Festigkeit haben lediglich die Aufgabe, die Kopfbolzendübel zur Sicherung der schubfesten Verbindung aufzunehmen, während die breiten dickflanschartigen Untergurte höherer Festigkeit eine bessere Ausnutzung der Druckspannungen in der Betonfahrbahnplatte erlauben sollen. Die durch Verwendung hochfester Stähle ermöglichte niedrige Bauhöhe führt zwar zu vergleichsweise größeren Durchbiegungen, die aber im Brückenbau für das Eigengewicht und einen Teil der Verkehrslast durch Überhöhung teilweise ausgeglichen werden können.

Ein weiterer interessanter Entwurfsvorschlag /37/ für die Neubaustrecken der Deutschen Bundesbahn sieht eine Verbundkastenträgerbrücke (Bild 36) vor,

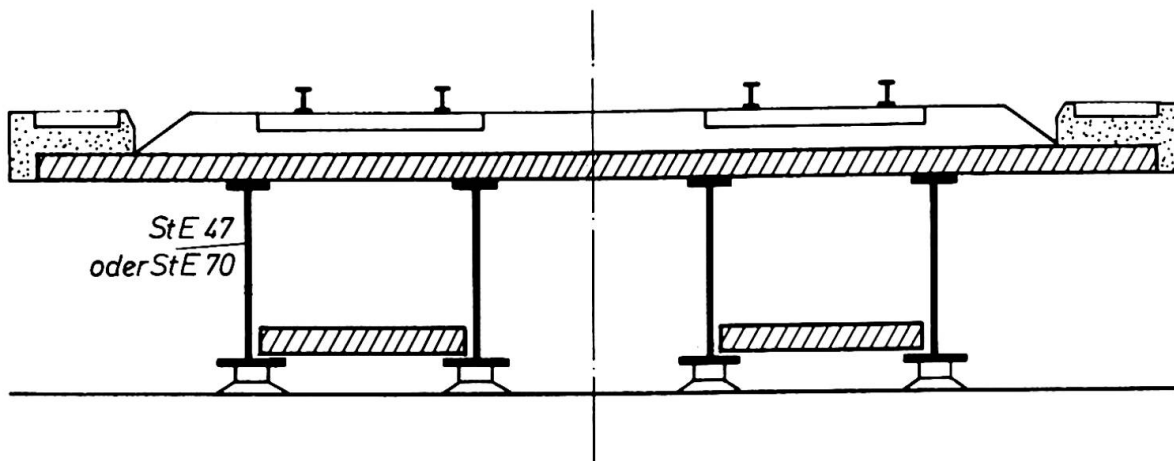


Bild 36: Vorgespannte Kastenträgerbrücke aus hochfestem Stahl für Eisenbahn-Schnellverkehr mit Ober- und Untergurtverbund, nach /36/

deren Ober- und Untergurt aus Betonplatten, die bei der Montage vorgespannt werden, und deren Hauptträger aus hochfestem Stahl bestehen. Die Vorspannung des Obergurts erfolgt durch Absenken nach Demontage der Hilfsstützen, die des Untergurts durch Auseinanderdrücken der zunächst zweigeteilten endverdübelt Untergurtplatte mittels Pressen und anschließende HV-Verschraubung mit den Stahlträgerunterflanschen (Bild 37). Dabei kommen die Stahlträger als "Spannbett" für die Vorspannung umso besser zur Wirkung je höher ihre Festigkeit ist. Die Vorteile des Systems liegen in der sehr hohen Steifigkeit infolge des doppelten Verbundes ($\text{Durchbiegung} = l/2000$), in den geringen Kriechverformungen des Betons, da beide Betongurte etwa gleichmäßig kriechen, in der hohen Dauerfestigkeit, da die Stahlspannungen infolge Verkehrslast durch die Betonmitwirkung gering sind und im Wegfall mindestens der Längssteifen, da die Stahlträger über die gesamte Länge unter Zugspannung stehen.

Ganz allgemein wird man Entwurfslösungen suchen müssen, die mehr bieten als Gewichtseinsparungen im Verhältnis der Streckgrenzen, so in der

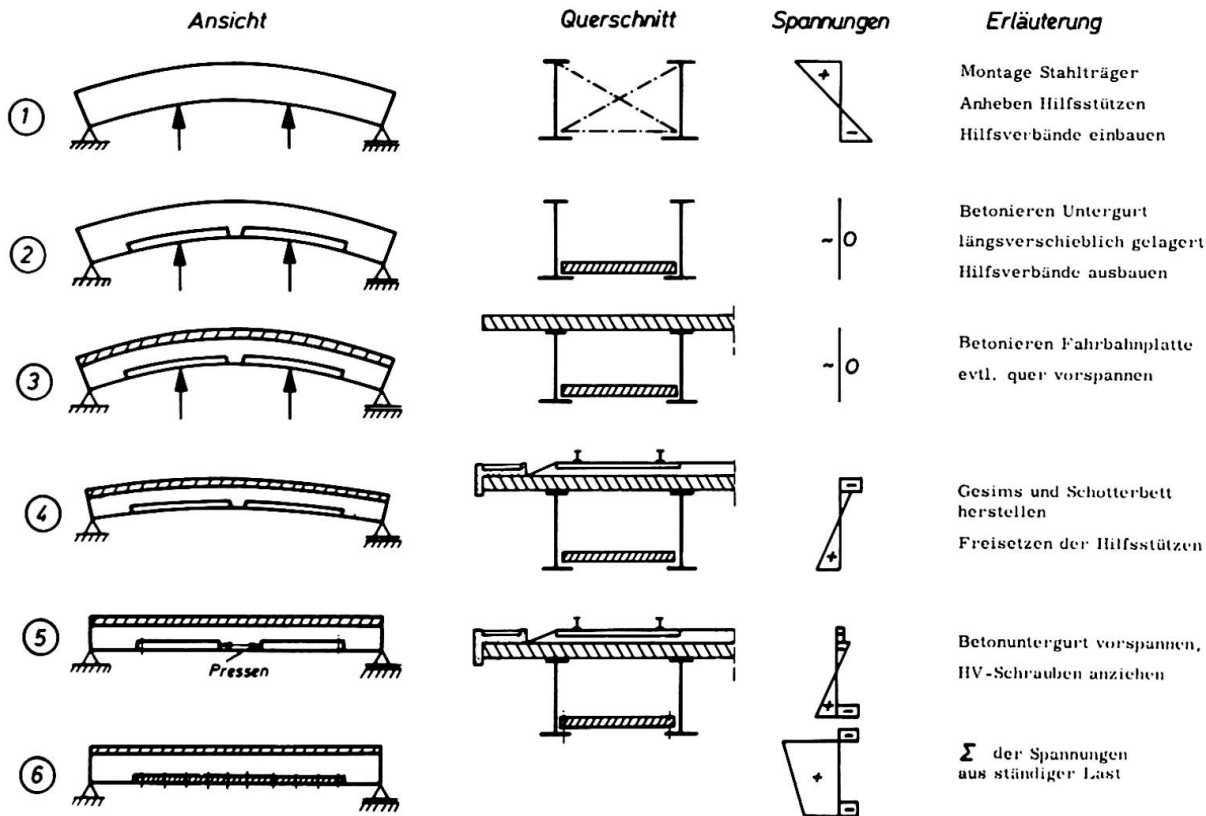


Bild 37: Montagevorgänge und Spannungszustände zu Bild 36

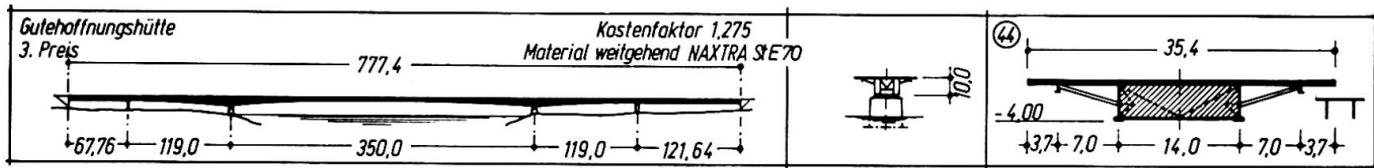


Bild 38: Entwurf einer Balkenbrücke über den Rhein bei Duisburg

Fertigung durch Dickenverminderung von Schweißnähten oder in Bereichen, für die Stähle mit niedrigerer Festigkeit nicht mehr ausreichen. Bild 38 zeigt den nicht ausgeführten Entwurf einer Balkenbrücke über den Rhein bei Duisburg von 350 m Spannweite, für den weitgehend St E 70 vorgesehen war. In den Kalkulationskosten lag er um 25 % über dem ausgeführten Schrägseilentwurf, dessen Pylone aus St E 70 bestehen. In St 52 wäre der Balkenentwurf allerdings nicht mehr möglich gewesen. Für die derzeit weitestgespannte Balkenbrücke der Welt (Niteroi-Brücke Brasilien) wurde vorwiegend Stahl BS 4360 Grade 55 (St E 47) verwendet /38/.

5.2 Seile

Die technische Ausnutzung des Verfestigungsmechanismus der Kaltverformung, mit der praktikable Zugfestigkeiten bis etwa 2000 N/mm^2 zu erreichen sind, erlaubt es mit Hilfe der Seiltechnik, wenn auch unter Verzicht auf Schweißseignung, Spannweiten von mehreren Kilometern Länge zu überbrücken (z. B. Hängebrückenentwurf von Musmeci u. Partnern über die Straße von Messina mit 3 km Spannweite). Das wiederentdeckte und in den letzten beiden Jahrzehnten kreativ entwickelte Schrägseilbrückensystem wäre ohne die Ausnutzung der hohen Stahlfestigkeit nicht möglich. Man unterscheidet im Hänge- und Schrägseilbrückenbau drei Seilarten:

Beispiele:

- | | |
|-------------------------|--|
| 1. Spiraldrahtseile: | Wye-Brücke, England 1966
Papineau-Leblanc-Brücke, Kanada 1969
Lower-Yarra-Brücke, Australien 1974
($\beta_z = 160/180$) |
| 2. Verschlossene Seile: | Massena-Brücke, Frankreich 1969 ($\beta_z = 135/182$)
Donaubrücke, CSSR 1972 ($\beta_z = 135/160$)
Köhlbrand-Brücke, BRD 1974 ($\beta_z = 120/160$) |
| 3. Paralleldrahtseile: | Toyosato-Brücke, Japan 1970 ($\beta_z = 165/185$)
Rheinbrücke Mannheim, BRD 1972 ($\beta_z = 140/160$)
Paranabrücke Argentinien 1975 ($\beta_z = 150/170$) |

Da das aufwendige Luftspinnverfahren in Zukunft auf Hängebrücken über 1000 m Spannweite beschränkt sein wird, steht z. Zt. besonders der Vergleich der technischen und wirtschaftlichen Leistungsfähigkeit zwischen dem verschlossenen Seil und dem Paralleldrahtbündel im Mittelpunkt der internationalen Diskussion. Nachdem in Japan die fabrikmäßige Herstellung des Paralleldrahtseiles (factory-bundled parallel wire strand) Anfang der 70er Jahre gelungen ist, scheint das wichtigste Handicap des technisch überlegenen Paralleldrahtseils beseitigt zu sein. Es wird darüber hinaus interessant sein, die Erfahrungen mit den Paralleldrahtseilen in einer Kunststoffhülle, die nach der Montage mit Mörtel ausgepreßt wird (Paranabrücke Argentinien 1975) abzuwarten. Denn der wirtschaftliche Korrosionsschutz ist die zweite vordringliche Entwicklungsaufgabe für Paralleldrahtbündel.

Der Vorteil des Paralleldrahtbündels gegenüber dem verschlossenen Seil liegt in folgenden hauptsächlichen Gesichtspunkten:

1. Höhere Zugfestigkeit
2. bessere Dauerschwingfestigkeit
3. höherer Elastizitätsmodul
4. größerer Seildurchmesser
5. geringeres spezifisches Gewicht

Bild 39 zeigt einen Wirtschaftlichkeitsvergleich /39/ für die 1972 fertiggestellte Rheinbrücke Mannheim-Ludwigshafen /40/, der mit einer Gewichtsersparnis von ca. 16 % zugunsten des allerdings dort auf der Baustelle hergestellten Paralleldrahtbündels abschließt.

Vor allem für größere Spannweiten im Schrägseilbrückenbau ($\ell = 400$ m bis 1000 m) wegen des infolge Durchhang reduzierten E-Moduls (Bild 40) und bei Beanspruchung durch Eisenbahnverkehr wegen der größeren Schwingweiten wird das fabrikmäßig hergestellte Paralleldrahtbündel seine Überlegenheit ausspielen können /42/.

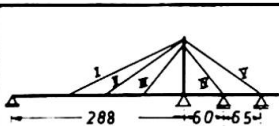
				Paralleldrahtseile										Patentverschlossene Seile										Vergleich	
				St 140/160 $E = 2,05 \cdot 10^7 \text{ Np/m}^2$ zul $\sigma = 0,45 \cdot 160 = 72 \text{ Kp/mm}^2$ zul $\Delta \sigma = 20 \text{ Kp/mm}^2$										St 150 $E = E_p = 1,65 \cdot 10^7 \text{ Np/m}^2$ zul $\sigma = 0,42 \cdot 150 = 63 \text{ Kp/mm}^2$ zul $\Delta \sigma = 15 \text{ Kp/mm}^2$											
Seil- gruppe	Länge m	max S^H Np	max ΔS (40% Verkehr) Np	Dimensio- nierung	F cm ²	max G^H Kp/mm ²	max ΔG Kp/mm ²	E, F · 10 ⁻³ Np	g t/m	G t	Dimensio- nierung	F cm ²	max G^H Kp/mm ²	max ΔG Kp/mm ²	E, F · 10 ⁻³ Np	g t/m	G t	$\frac{\Delta F}{F}$ %	$\frac{\Delta G}{G}$ %						
I	212	9.849	1.360	12 x	1.362 (1.470)	72,3 (67,0)	10,0	2.792 (3.014)	1.069 (1.154)	227 (245)	15 # 123	1.560	63,1	8,7	2.574	1.326	281	7,8	7,0						
II	143	5.696	1.535	8 x	908	62,7	16,9	1.861	0.713	102	12 # 110	1.004	56,7	15,3	1.657	0.853	122	11,0	2,4						
III	91	2.378	619	4 x	454	52,4	13,6	931	0,357	33	6 # 100	410	58,0	15,1	677	0,348	32	27,3	0						
IV	51	2.773	634	4 x	454	61,1	14,0	931	0,357	33	6 # 103	436	63,6	14,5	719	0,371	34	22,8	0						
V	144	15.650	2.690	20 x	2.271 (2.336)	68,9 (67,0)	11,8	4.656 (4.789)	1.783 (1.834)	257 (264)	24 # 123	2.496	62,7	10,8	4.118	2,122	306	11,6	4,3						
Bemessung nach günstigster Annahme: zul $\sigma = 0,45 \beta_H = 72 \text{ Kp/mm}^2$										652	Bemessung nach DIN 1073 Entw. 1969: zul $\sigma = 0,42 \beta_H = 67 \text{ Kp/mm}^2$										(677)	775			
Bemessung für Verwerruß : zul $\Delta \sigma = 20 \text{ Kp/mm}^2$										[690]	Bemessung für Verwerruß : zul $\Delta \sigma = 15 \text{ Kp/mm}^2$														
Bei voller Ausnutzung von zul σ haben beide Seilarten etwa die gleiche Steifigkeit											Gesamtgewichtsdifferenz $775^t - 652^t = 123^t = 15,9 \%$										bestehend aus folgenden Differenzanteilen				
$f^p = \frac{S}{\text{zul } \sigma^p}; f = \frac{S}{\text{zul } \sigma}; \frac{f^p}{f} = \frac{\text{zul } \sigma}{\text{zul } \sigma^p}$											1.) spez. Gewicht (7,85 zu 8,50) = 53% = 6,8 %										2.) Drahtfestigkeit (160 zu 150) = 32% = 4,2 %				
$\frac{f^p}{f} = \frac{f^p \cdot \text{zul } \sigma}{f \cdot \text{zul } \sigma^p} = \frac{2,05 \cdot 63}{1,65 \cdot 72} = 1,007 \approx 1,0$											3.) Sicherheitskoeff. (0,45 zu 0,42) = 25% = 3,2 %										4.) zul $\Delta \sigma$ (20 zu 15) = 13% = 1,7 %				

Bild 39: Vergleich Paralleldraht- und verschlossenes Seil bei einer Schrägseilbrücke über den Rhein, nach /38/

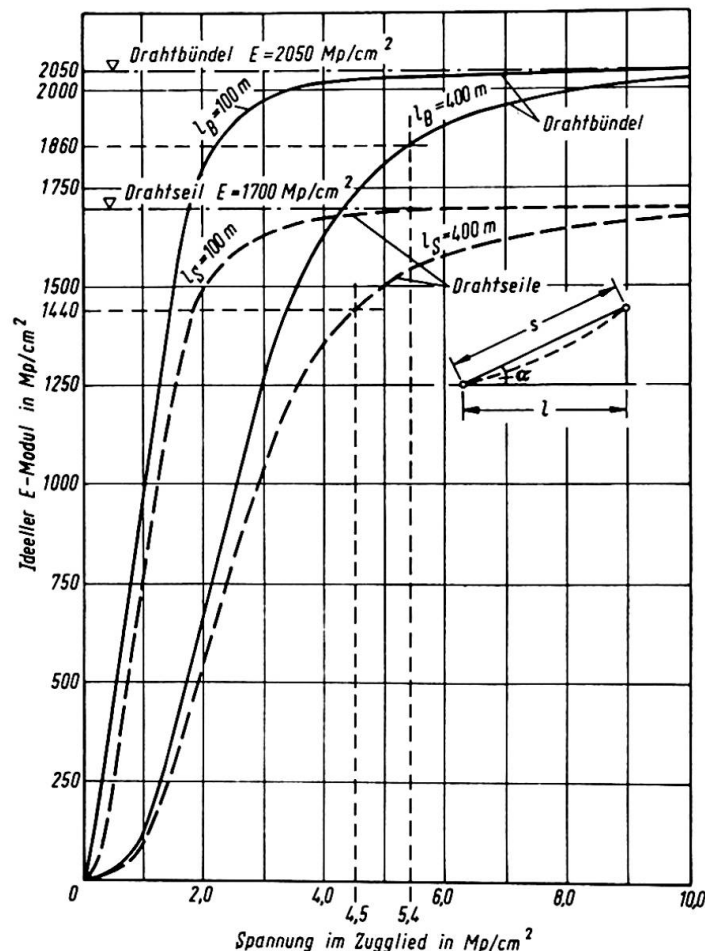
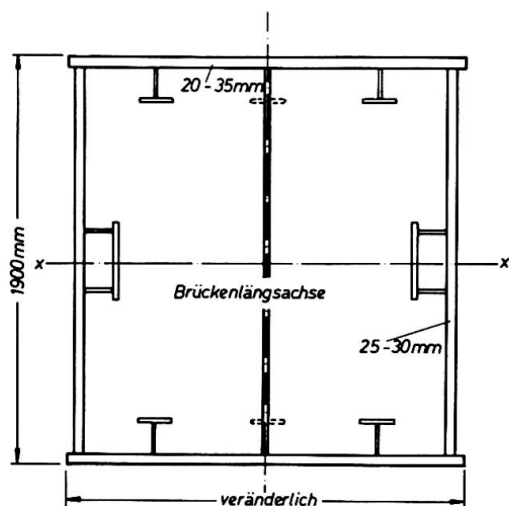


Bild 40: Ideeller E-Modul infolge Seildurchhang bei Paralleldraht- und verschlossenen Seilen, nach/40/

5.3 Pylone für seilverspannte Brücken

Die wichtigsten Konstruktionsglieder, die das Kräftespiel der Überspannungen aus hochfesten Zuggliedern mit dem auf Biegung und Druck beanspruchten Brückenbalken (Versteifungsträger) verbinden, sind die Pylone. Die Ausnutzung der im Bereich der Streckgrenze liegenden Stauchgrenze höherfester Stähle wird für diese vom Verhältnis der Anteile von Druck und Biegung der Pylone bestimmt. Vom Entwurf her gesehen sollen die Pylone biegeweich, um der Bewegung der Kabel wenig Widerstand entgegenzusetzen, aber hinreichend knicksteif zur Aufnahme der hohen Druckkräfte ausgebildet werden. Hier ist der nicht "hochlegierbare", sondern gleichbleibende E-Modul, der bei hochfesten Stählen zu relativ größerer Verformung führt, von Vorteil, weil er das Kräftespiel von Biegung zu Druck verlagert und damit die hochliegende Stauchgrenze besser zur Wirkung bringt /43/. Bei den Pylonen der Schrägseilbrücke Duisburg wurde für die Pylone wasservergüteter Baustahl der Festigkeitsklasse St E 70 eingesetzt (Bild 41), was zu erheblicher Reduzierung der Wand-



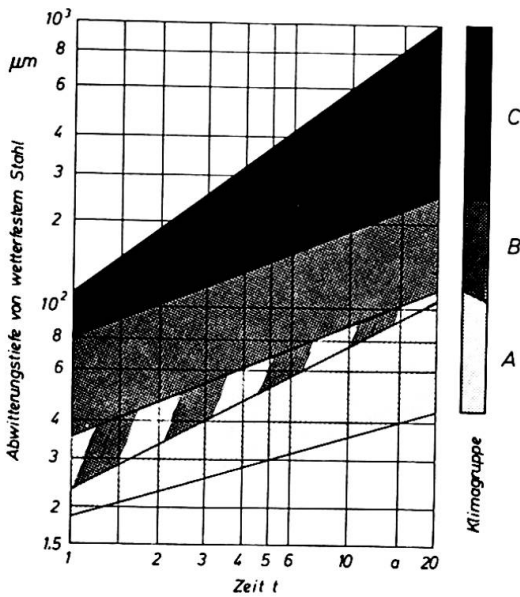
dicke führte und sich somit besonders schweißtechnisch für die Stützenstöße günstig auswirkte. Die zulässige Druckspannung wurde hier mit $\text{zul}\sigma = 41 \text{ kN/cm}^2$ festgelegt. Die Gewichtspersparnis betrug gegenüber Stahl St 52 ca. 41 %, wovon 34 % aus der höheren Stauchgrenze ($\sigma_F = 70 \text{ kN/cm}^2$ gegenüber 36) und 7 % aus der größeren Biegeweichheit resultierten /44/.

Bild 41: Querschnitt der Pylone der Schrägseilbrücke Duisburg aus St E 70

6 Besondere Entwurfshinweise bei Anwendung wetterfester Stähle

6.1 Mechanismus der wetterfesten Deckschichtbildung

Die Zulegierung von geringen Mengen Kupfer, Phosphor, Chrom und Nickel führte nach bemerkenswerten Anfängen in der 20er Jahren (Patina-stahl in Deutschland) zur systematischen Entwicklung wetterfester Stähle in den 50er Jahren, deren bekannteste Analyse als Corten-Stahl bezeichnet wird. Die genannten Elemente bilden mit der Zeit auf der Stahloberfläche basische Sulfate, Hydroxyde, Karbonate, Phosphate und Silikate, die in Wasser und Schwefeldioxyd unlöslich sind und die Stahloberfläche gegen aggressive Atmosphären abdichten. Die sich bei den wetterfesten Stählen je nach den atmosphärischen Bedingungen nach etwa 1 bis 3 Jahren bildende feste Deckschicht mit braunvioletter Färbung bewirkt in nichtchloridhaltiger Atmosphäre eine nur noch geringfügige Abwitterung, die z. B. in 20 Jahren nur 200μ beträgt (Bild 42). Zur Erhaltung der Schweißbarkeit muß der Kohlenstoffgehalt wie bei den höherfesten Stählen abgesenkt werden, und zwar hier auf Werte zwischen 0,10 - 0,15 % (Bild 43). Meist wird wetterfester Stahl in der Festig-



chemische Zusammensetzung									
Stahlmarke	C %	Si %	Mn %	P %	S %	Cr %	Cu %	Ni %	V %
WT St 52-3A	0,12	0,30 0,75	0,20 0,50	0,070 0,150	≤0,035	0,50 1,25	0,30 0,55	0,65	-
WT St 52-3B	0,15	0,10 0,40	0,90 1,30	≤0,045	≤0,035	0,50 0,80	0,30 0,50	0,40	0,02 0,10
WT RSt 37-2	0,15	0,10 0,40	0,20 0,50	≤0,050	≤0,035	0,50 0,80	0,30 0,50	0,40	-
mechanische Eigenschaften									
Stahlmarke	Blechdicke mm	Streckgrenze kp/mm²	Zugfestigkeit kp/mm²	Bruchdehnung %	Kerbschlagzähigkeit (ISO-V-Probe, längs)				
					-20 °C	+10 °C			
					mcp/cm²	mcp/cm²			
WT St 52-3A	bis 12	≥36	52 ÷ 62	≥22	≥3,5	-			
WT St 52-3B	bis 100	≥36	52 ÷ 62	≥22	≥3,5	-			
WT RSt 37-2	bis 100	≥24	37 ÷ 50	≥25	-	≥3,5			

Bild 42: Zeitliche Abwitterungstiefe wetterfester Stähle in verschiedenen Klimazonen, nach /44/

Bild 43: Typische chemische Zusammensetzung wetterfester Stähle, nach /45/

keitsklasse des St 52 mit ca. 360 N/mm^2 Streckgrenze erschmolzen, in einigen Ländern auch als St 37 mit einer Streckgrenze von 240 N/mm^2 bei geringerem Manganhalt.

6.2 Besondere Entwurfshinweise

Da die wetterfesten Stähle in ihren mechanischen Eigenschaften und Verarbeitungsmöglichkeiten den anderen Stählen der Festigkeitsklasse St 37 und St 52 entsprechen, kann auf deren Erfahrungsgut und Bemessungsregeln zurückgegriffen werden unter Beachtung besonderer Zusatzmaßnahmen, die zum Teil behördlich geregelt sind /46/. Die bei Grundmaterial-Prüfstäben aus wetterfestem Stahl festgestellte geringere Dauerfestigkeit infolge der stärkeren Naturkerbigkeit der Oberfläche kommt in den Konstruktionen des Stahlbaus kaum zur Wirkung, weil in der Regel die konstruktiven "Kerben" der Verbindungen für die Bemessung maßgebend werden.

Folgende Entwurfshinweise gelten für wetterfeste Stähle, beanspruchen aber keine Vollständigkeit:

1. Die Deckschicht bildet sich nicht:
 - in Meeresnähe
 - bei ständiger Wasserbenetzung
 - in chemikalienhaltiger Luft (z. B. Chloride)
2. direkte Verbindung mit anderen Metallen vermeiden wegen elektronischer Lokalelementbildung
3. Regenwasser, das durch Eisenhydroxyd und -sulfat besonders bei Beginn der Deckschichtbildung rotbraun gefärbt wird, nicht von Konstruktionen abtropfen lassen, sondern planmäßig durch Rinnen und Rohre ableiten.

4. Als Nachbarstoffe Materialien mit glatter und dichter Oberfläche verwenden, z. B. Glas und Emaille
5. Farblose, dichtende Schutzanstriche schützen poröse Nachbarstoffe (Beton) für die Zeit der Deckschichtbildung
6. Beim Übergang zum Erdreich empfehlen sich Steinschüttungen (grobe Kiesel) in direkter Nachbarschaft
7. Möglichst glatt konstruieren, einspringende Ecken und Versatz vermeiden
8. Für Verbindungen Elektroden und Schrauben ebenfalls aus wetterfestem Stahl verwenden
9. Entzundern fördert die Gleichmäßigkeit der Deckschichtbildung
10. Die geringe Abwitterung von z. B. 200μ in 20 Jahren bei Industrieklima ist bei der Bemessung dünnwandiger Konstruktionen zu beachten, z. B. durch einen Abwitterungszuschlag.

Von der hervorragenden Eignung wetterfester Stähle besonders für hohe und weitgespannte Tragwerke wurde in den letzten Jahren bei zahlreichen Bauwerken der Welt Gebrauch gemacht, so z. B. beim Verwaltungsgebäude der US Steel Corp. in Pittsburgh, bei der Onomichi-Schrägseilbrücke in Japan, beim Gerichtshof der Europäischen Gemeinschaft in Luxemburg und beim Verwaltungsgebäude der Imperial Tobacco Group in Hartcliffe, Großbritannien.

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ZUSAMMENFASSUNG

Verschiedene Verfestigungsmechanismen sind von Einfluss auf die Eigenschaften höherfester Stähle, für die neuerdings in Normen bestimmte Festigkeitsklassen und Lieferformen festgelegt werden. Die Eigenschaften: Zug- und Druckfestigkeit, Elastizität, Plastizität, Stabilität, Schwingfestigkeit beeinflussen den Ingenieur-entwurf. In Abhängigkeit dieser Eigenschaften werden die Möglichkeiten und Grenzen der Anwendung hochfester und wetterfester Stähle im Hoch- und Brückenbau erörtert.

SUMMARY

Several mechanisms of hardening influence the characteristics of high tensile steel, for which recently standards of classifications of strength and specifications of delivery have been worked out. The characteristics: ultimate tensile and compressive strength, modulus of elasticity, yield strength, stability, fatigue strength influence the engineer's design. Referring to these characteristics possibilities and bounds of application of high tensile steel in structural and bridge engineering are shown.

RESUME

Les propriétés des aciers à haute résistance, dont les normes de classe de résistance et de façonnage ont été déterminés récemment, sont influencés par différents mécanismes d'écrouissage. Ces propriétés: résistance à la traction et compression, élasticité, plasticité, stabilité, résistance à la fatigue ont une influence sur les projets des ingénieurs. Les possibilités et les limites d'application des aciers à haute résistance dépendent de ces propriétés, et sont présentées pour diverses constructions de bâtiments et de ponts.

Vc

Problèmes de fabrication et de montage

Herstellungs- und Montageprobleme

Fabrication and Erection Problems

*Le rapport introductif n'est pas parvenu à temps pour l'impression.
Des contributions sont cependant bienvenues pour le Rapport Préliminaire.*

*Der Einführungsbericht hat uns nicht rechtzeitig erreicht und konnte deshalb hier
nicht abgedruckt werden.
Beiträge für den Vorbericht sind jedoch willkommen.*

*The Introductory Report has not reached us on time for being printed.
Contributions for the Preliminary Report however are welcome.*

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VI

Constructions en béton préfabriqué

Vorfabrizierte Bauwerke

Precast Structures

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Sicherheit und Stabilität von Elementen und Bauwerken

Safety and Stability of Elements and Structures

Sécurité et stabilité des éléments et des constructions

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1. Zum Begriff der Sicherheit und Stabilität

Das enorme Tempo, das in der Entwicklung der Bautechnik eingetreten ist, hat das Problem der Sicherheit der Bauwerke neu gestellt. Die alte empirische Anschauung von der "absoluten Sicherheit" muß durch eine wissenschaftlich begründete Sicherheitstheorie abgelöst werden.

Über diese technisch-philosophische Grundfrage des Bauwesens ist in den vergangenen Jahren bereits eine umfangreiche Arbeit geleistet worden, wie viele dazu durchgeführte wissenschaftliche Veranstaltungen und das starke Anwachsen der Zahl von Aufsätzen zu dieser Problematik in der Literatur beweisen. Die IVBH hat daran entsprechend ihrer Aufgabenstellung einen großen Anteil, wie dies u.a. das bedeutende Thema "Sicherheit" des VIII. Kongresses /1/, das umfangreiche Symposium "Über neue Aspekte der Tragsicherheit und ihre Berücksichtigung in der Bemessung" /2/ und das kritische Podiumsgespräch im Rahmen des Symposiums "Bemessung und Sicherheit von Stahlbeton-Druckgliedern" /3/ veranschaulichen. Erwartungsgemäß konnte diese entscheidende Frage noch nicht durch eine allseits abgeklärte, einheitliche neue Sicherheitstheorie ausreichend anwendungsreif beantwortet werden. Dies hat neben den objektiven Schwierigkeiten, die insbesondere in der wegen des stochastischen Charakters komplizierten Bereitstellung exakter Angaben für die strenge mathematische Behandlung zu suchen sind, Ursachen in der Vielfalt der praktischen Auswirkungen, weil sich dafür nicht nur die Bauschaffenden, sondern die gesamte Gesellschaft verantwortlich interessiert /4/.

Deutlich erkennbar ist, daß die Sicherheit der Bauwerke mehr und mehr mit Hilfe der wirklichkeitsnahen stochastische Einflüsse berücksichtigende Methoden errechnet wird und die bisher üblichen nur deterministisch orientierten Betrachtungsweisen zurückgedrängt werden. Der bereits von vielen Ländern angenommene ISO-Standard /5/, an dessen Entwurf auch die IVBH mitgewirkt hat, liefert dafür

einen Beweis. Von einer umfassenden Anwendung der Zuverlässigkeitstheorie für die Vorausbestimmung der Sicherheit der Bauwerke sind wir heute jedoch weit entfernt, weil diese Theorie zum Nachweis einer festgelegten Bestands- bzw. Versagenswahrscheinlichkeit eine sehr komplizierte und aufwendige mathematische Behandlung mit umfangreichen teilweise kaum zu bewältigenden statistischen Erfassungen erfordert. Die weitgehendste Annäherung an diese Theorie, wie sie u.a. in der heute bereits angewandten teilprobabilistischen Methode der Berechnung nach Grenzzuständen zum Ausdruck kommt, bleibt jedoch das Hauptziel.

Im Vergleich zu dem Begriff der Sicherheit, wo eine Vielzahl von objektiven und subjektiven technischen, ökonomischen und philosophischen Faktoren mitwirken, wird mit dem Begriff der Stabilität nur ein rein technisches Teilgebiet der Sicherheit umrissen. Seine besondere Herausstellung kann mit der für die Sicherheit eines Bauwerkes erstrangigen Frage nach der Standsicherheit der Konstruktion begründet werden. Stabilität bedeutet, daß das Element oder Bauwerk unter allen möglichen Beanspruchungen, die im Verlauf der Herstellung, des Transportes, der Montage und der Nutzung einwirken "stabil", d.h. standfest bleiben muß.

Im Rahmen des Unterthemas VIa sollen nicht die allgemeinen Fragen und Aspekte der Sicherheit vordergründig behandelt werden. Entsprechend dem Hauptthema wird erwartet, daß zu den speziellen Problemen, die sich mit der Anwendung der Betonfertigteil-Bauweise ergeben (im Folgenden teilweise angedeutet), Untersuchungsergebnisse, Erfahrungen und Lösungen mitgeteilt und diskutiert werden. Die Methode der Berechnung nach Grenzzuständen mit aufgeteilten Faktoren ermöglicht es, diese Besonderheiten explizit zu erfassen und in die Sicherheitsbetrachtung direkt mit einzubeziehen. Damit wird auch ein Beitrag zur Schaffung einer neuen Sicherheitstheorie geleistet.

2. Sicherheit und Stabilität von Betonfertigteilen

Die Frage nach der Sicherheit und Stabilität von bewehrten Betonelementen wird aus folgenden Gründen in Erweiterung der Fragestellung für das monolithische Bauwerk aufgeworfen: Während des Herstellungsprozesses, des Transportes und des Einbaues der Betonelemente dürfen

- keine Menschenleben gefährdet werden;
- keine Ausrüstungen, wie Fertigungsaggregate, Formen, Transportfahrzeuge, Hebmittel u.a. zerstört oder beschädigt werden;
- keine Beschädigungen, ungewollte bleibende Verformungen oder Brüche der Fertigteile eintreten, die einen Einbau ausschließen.

Die ausreichend fundierte Beantwortung dieser Frage ist mit den bisherigen allgemeingültigen Ergebnissen über Sicherheit und Stabilität von Baukonstruktionen nicht allein möglich. Weitgehendere Überlegungen über die auftretenden und anzunehmenden Beanspruchungen und die aus den unterschiedlich zu bewertenden Auswirkungen auch unterschiedlich festzulegenden Sicherheiten gegen eine ungewollte Gefahr sind dazu notwendig.

2.1. Bei der Herstellung

Für die Sicherheit und Stabilität des aus Fertigteilen errichteten Bauwerkes ist es notwendig, daß das Betonelement mit seinen Baustoffeigenschaften und seiner inneren und äußeren Geometrie dem im Projekt geforderten Werten innerhalb eines Toleranzbereiches entsprechen muß, um die volle Funktionstüchtigkeit im Bauwerk zu gewährleisten.

Änderungen von bewehrten Betonelementen auf der Baustelle sind gar nicht oder nur bedingt mit erheblichem Aufwand möglich. Die Prüfung aller Eigenschaften auf der Baustelle ist nicht nur sehr beschwerlich, sondern vielfach technisch-ökonomisch nicht möglich. Die zerstörungsfreien Prüfmethode zur Messung der Betoneigenschaften und zur Fixierung der Lage der Bewehrung sind noch sehr ungenau bzw. sehr zeit- und kostenaufwendig. Die Überprüfung der Vorspannung ist fast überhaupt nicht möglich. Selbst die genaue Ermittlung der äußeren wesentlichen Abmessungen bereitet oft erhebliche unliebsame Erschwernisse auf der Baustelle.

Ein beachtlicher Vorteil der Fertigteilbauweise ist es aber, daß in einer stationären industriell organisierten Fertigungsstätte durch Anwendung eines geschlossenen Qualitätssicherungssystems die Toleranzen in den geometrischen Abmessungen und die Streuungen der Betoneigenschaften soweit eingegrenzt werden können, damit diese günstigen Aspekte bei der Bewertung der Sicherheit in einer Form einfließen, die direkt zur Verbesserung der Ökonomie der Bauaufgabe führt. Das Qualitätssicherungssystem schließt alle Maßnahmen auf technischem, organisatorischem, kadernmäßigem und ökonomischem Gebiet ein, die der Beherrschung des Produktionsprozesses zur qualitätsgerechten Produktion dienen. Dabei bilden die Qualitätsentwicklung, die Qualitätssicherung und die Qualitätskontrolle eine Einheit. Grundlage bilden hierbei staatliche Vorschriften, wie z.B. in der DDR die ASMW-VW 1184 /6/ über deren Grundprinzipien Wahl /7/ berichtet oder in der BRD die DIN 1084 /8/, die Grimm /9/ erläutert.

In der DDR ist seit 1970 eine Vorschrift über den Nachweis der Betondruckfestigkeit mit Hilfe statistischer Qualitätskontrolle /10/ gültig, die es den Betonwerken erlaubt in Abhängigkeit von der in ihrer Mischstation nachgewiesenen Standardabweichung die anzustrebende mittlere Betonfestigkeit für die im Projekt geforderte Rechenfestigkeit selbst zu bestimmen. Der wirtschaftliche Vorteil, durch eine geringere Streuung den Zementein-satz reduzieren zu können, liegt hier offen auf der Hand, wie in Bild 1 (nächste Seite) erkennbar.

Zur Kontrolle der äußeren Geometrie der Betonfertigteile besteht die Tendenz zur Entwicklung spezieller z.T. automatisierter Meßplätze. Entwicklungen der UdSSR stellt Neumann /11/ vor. Eine automatisierte Anlage zur Kontrolle der Abmessungen beliebig im Raum angeordneter Fertigteile befindet sich im Institut für Stahlbeton Dresden in der Entwicklung. Mittels eines Tastfühlers werden die Umriss des Fertigteiltes festgestellt, mit den vorprogrammierten Solldaten verglichen und das Ergebnis auf einer Kontrollkarte ausgedruckt. Mit der Auswertung dieser laufenden Kontrollmessungen werden statistische Angaben über Toleranzen gewonnen, die eine weitere genauere Aussage über wirklich vorhandene Querschnitte, Eigengewichte, Auflagerflächen usw. erlauben, um sie künftig in die Sicherheitsbetrachtungen direkt mit einzubeziehen.

Als ein erstes Problem zur Diskussion im Rahmen dieses Themas stellt sich damit folgende Frage:

Wie können die erhöhten Qualitätssicherungen bei der Herstellung der Betonfertigteile in die Sicherheitskonzeptionen eingebaut werden, damit wirtschaftliche Vorteile ohne Herabsetzung der Gesamtsicherheit des aus Fertigteilen errichteten Bauwerkes eintreten ?

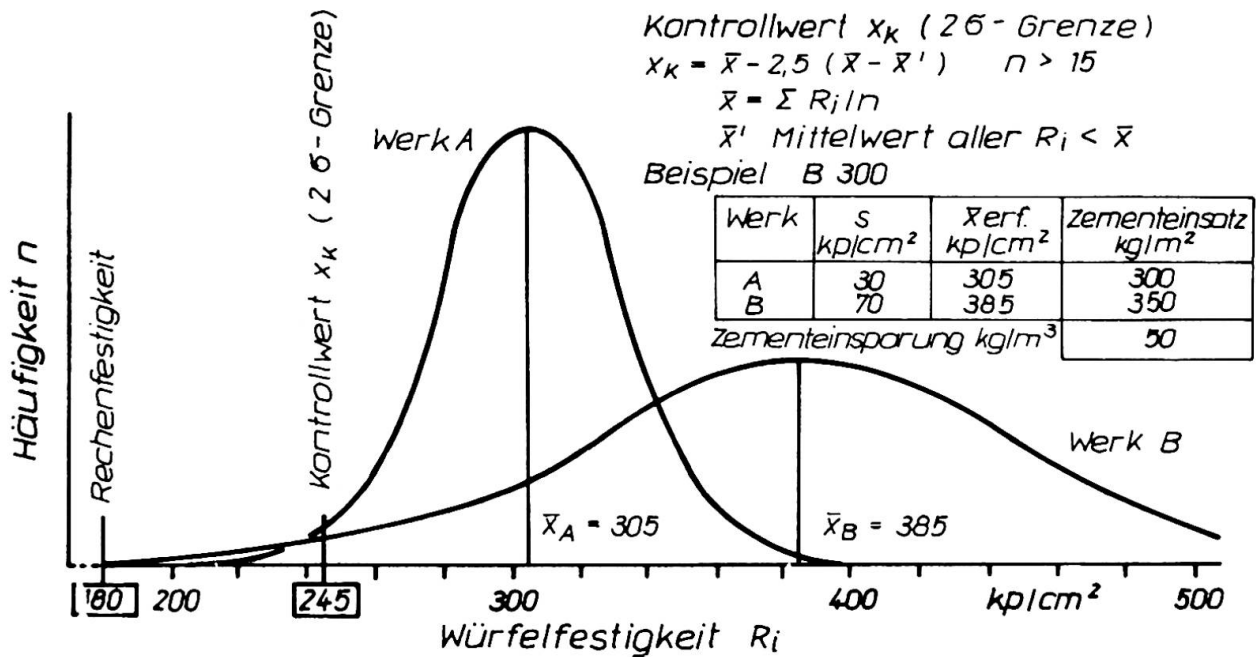


Bild 1 Erforderlicher Mittelwert der Betonfestigkeit B 300 zur Einhaltung der Kontrollfestigkeit nach DAMW-VW 968 /10/ bei Standardabweichungen $s = 30 \text{ kp/cm}^2$ und 70 kp/cm^2

Im Herstellungsprozeß wird das Betonfertigteil bereits vielfältigen Beanspruchungen unterworfen, obwohl die Betonfestigkeit als Funktion der Erhärtungszeit und des Erhärtungsregimes noch nicht den 28-Tage-Normwert erreicht hat. So wirken beim Ausschalen, beim Abheben von der Form, bei der Einleitung der Vorspannung und durch Temperatur- oder Schwindverformungen Kräfte, deren genauere Kenntnis ebenso wie die zum Belastungszeitpunkt vorhandene Betonfestigkeit die Grundlage für eine wirtschaftlich optimale Sicherheit darstellen.

Die vorliegenden umfangreichen Untersuchungen zur Verteilungsfunktion der Betonfestigkeit gehen von den 28 Tagefestigkeiten aus. Es ist jedoch anzunehmen, daß die Frühfestigkeiten nicht den gleichen statistischen Verteilungsgesetzen und insbesondere die Beziehungen der Druckfestigkeit zur Rißfestigkeit, Verbundfestigkeit und dem Elastizitätsmodul nicht mit den 28 Tagewerten korrespondieren. Diese Beziehungen der Frühfestigkeiten werden gleichzeitig durch die Methoden der Erhärtungsbeschleunigung z.B. Warmbehandlung bestimmt. Im Institut für Stahlbeton Dresden wurde z.B. für die 1 Tagefestigkeit warmbehandelter Betone eine Standardabweichung der Betonfestigkeit von 46 bis 72 % der Standardabweichung der 28 Tagefestigkeit festgestellt.

Grundsätzlich unterschiedliche Auffassungen gibt es auch zur erforderlichen Betonfestigkeit zum Zeitpunkt der Eintragung der Vorspannung bei Fertigteilen mit sofortigem Verbund. Die Vorschriften /12/ /13/ fordern generell 80 % der 28 Tagefestigkeit als Mindestwert, die Vorschrift der UdSSR /14/ verlangt ausgehend von der Sicherheit des Verbundes in Abhängigkeit von den Stahlarten und der Betonklasse eine Mindestfestigkeit von 140 bis 300 kp/cm² und die Vorschrift der USA /15/ schreibt die erforderliche Umspannfestig-

keit indirekt vor, indem zulässige Zug- und Druckbeanspruchungen als Funktion der tatsächlich vorhandene Umspannfestigkeit vorgegeben werden. Die zulässige Druckbeanspruchung ist dabei mit 0,6 Rp der Umspannfestigkeit relativ höher als die nach Aufbringen aller Lasten mit 0,45 Rp. Ausgehend von den zulässigen Spannungen in /12/ und der Beachtung der Festigkeitsstreuung des Betons, ist in der DDR eine Umspannfestigkeit von 0,62 bis 0,70 R₂₈ als Mindestwert bei statistischer Qualitätskontrolle zugelassen.

Damit ergeben sich folgende diskussionswerte Probleme:

Auf welches Minimum kann der Sicherheitsabstand zwischen den speziellen Beanspruchungen der Betonfertigteile im Herstellungsprozeß und der von den zeit- und prozeßabhängigen Betonfestigkeiten bestimmten Tragfähigkeit reduziert werden, um damit einen wirtschaftlich optimalen Fertigungsprozeß zu gestalten ?

Welche Umspannfestigkeiten sind für Betonfertigteile aus sicherheitstheoretischer und wirtschaftlicher Sicht optimal ?

2.2. Beim Transport und der Lagerung

Die Beanspruchung, die Betonfertigteile während des Transportes oder bei der Lagerung erfahren, sind durch folgende Besonderheiten charakterisiert:

- Die Betonfestigkeit hat vielfach noch nicht den 28-Tage-Normwert erreicht.
- Der Angriff der Einzelkräfte aus den Anschlagmitteln erfolgt an anderen Stellen und in einer anderen Art wie die Stütz- und Verbindungskräfte im eingebauten Zustand des Fertigteiles.
- Die Lage der Fertigteile und damit die Wirkung der Eigengewichtslasten ist häufig stark abweichend vom Einbauzustand ebenso wie die Stützkkräfte der Transportfahrzeuge und bei der Zwischenlagerung.
- Der Beanspruchungszustand ist stets relativ kurz aber durch dynamische Kräfte beeinflusst.

Diesen besonderen Beanspruchungen muß bei ausreichender Sicherheit unter Beachtung der folgenden Prämisse entsprochen werden:

- Frühestmöglicher Transport, um die Formen und Fertigungslinien optimal auszunutzen.
- Möglichst keine zusätzlichen Bewehrungen vorsehen, die nur für den kurzfristigen Transport und Lagerprozeß erforderlich sind.

Das Anschlagen der Betonfertigteile und die Form der Anschlagmittel sind ganz entscheidend für die Beanspruchung bei bestimmten Transportvorgängen. Bekannt sind einbetonierte Transportösen, speziell angeordnete Aussparungen für den Transport, einbetonierte Schraubhülsen oder andere Verbindungsmittel und das Anschlagen mit Klemmzangen, Vakuumhebern und anderen Transportmitteln, die keinerlei zusätzliche konstruktive Ausbildungen für den Transport am Betonfertigteile erforderlich machen. Greifen wir als Beispiel die Tragöse heraus. Wie muß eine solche Tragöse ausgebildet sein und nach welchen Gesichtspunkten kann sie berechnet werden. In der DDR wurden als Grundlage der Vorschrift über Tragösen /16/ Versuche entsprechend Bild 2 durchgeführt und als Kriterium zur Festlegung der zulässigen Last einschließlich eines Massenkraftfaktors die bleibende Verformung des Schenkelabstandes gewählt.

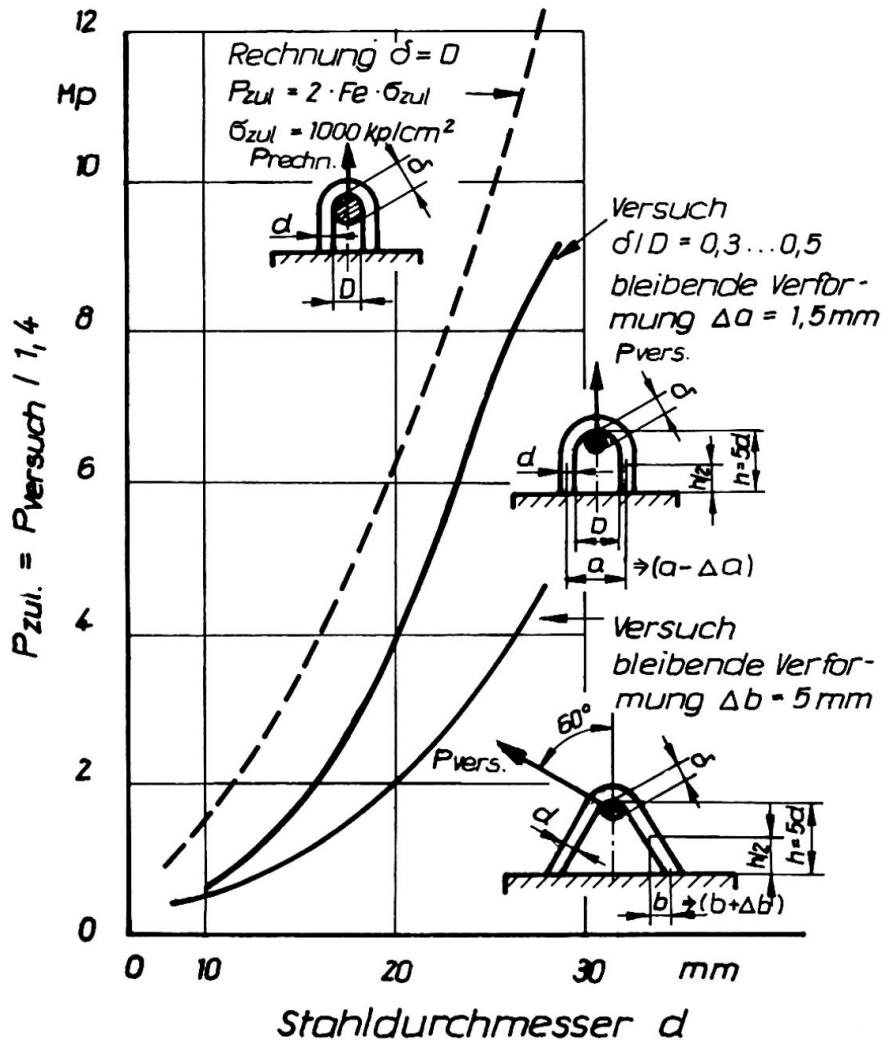


Bild 2

Zulässige
 Traglasten
 für Tragösen
 nach /16/ als
 Funktion der
 bleibenden
 Verformung
 der Schenkel
 (Versuchsergebnisse)
 St A I mit
 $\sigma_s = 2400 \text{ kp/cm}^2$

Ein weiteres Problem ist somit:

Welche Forderungen hinsichtlich konstruktiver Durchbildung und notwendiger Sicherheiten sind an die Einbauteile zu stellen, die speziell für den Transport vorgesehen sind?

Von Bedeutung ist dabei, ob z.B. mittels einer Traverse nur vertikale Kräfte angreifen oder ob mittels Schrägseilzug auch horizontale Kräfte das Betonfertigteile beanspruchen, wie überhaupt Angriffspunkt und -richtung die Sicherheit entscheidend beeinflussen. Untersuchungen über den Stabilitätszustand von Betonfertigteilen im Transportzustand sind deshalb für viele Fälle unbedingt erforderlich. So ist das Problem des Knickens, Kippens und Beulens von schlanken, hohen oder großflächigen Fertigteilen gerade für die Transportvorgänge zur Beurteilung der Sicherheit von allergrößtem Interesse. Der Bereitstellung von Berechnungsmethoden zur Seitenstabilität von stabförmigen Baugliedern wird daher in letzter Zeit erhöhte Aufmerksamkeit geschenkt /17/ /18/ /19/ /20/ /21/ /22/. Dabei besteht auch hier die Problematik in der Festlegung der erforderlichen Sicherheit bei fortschreitender Kenntnis des Werkstoffverhaltens. Gleichzeitig werden Maßnahmen z.B. Montageverspannung /23/ und Hilfstraversen /24/ zur Beherrschung dieses kritischen Zustandes ohne zusätzliche Aufwendungen am Bauelement vorgeschlagen.

Hieraus läßt sich folgende Frage ableiten:

Welche Festlegungen und Sicherheitsforderungen sind auf Grund welcher Untersuchungen zum Knicken, Kippen und Beulen von Betonfertigteilen im Transportzustand in die Bauvorschriften aufgenommen worden oder welche theoretischen oder Versuchsergebnisse können in einer für die praktische Anwendung aufbereiteten Form dazu bereitgestellt werden ?

Der rollende Transport auf Schiene oder Straße erzeugt Schwingungen und Stöße, die insbesondere beim Rangieren oder auf der unebenen Baustraße beachtliche Größen erreichen. Ledderboge und Danilow /25/ geben bei loser Zweipunktauflagerung von Balken eine Erhöhung des Biegemomentes um das 3,5-fache an. In der Praxis wird daher die dynamische Transportbeanspruchung durch elastische Zwischenlagen, Verspannungen oder spezielle Pufferkonstruktionen /26/ reduziert. Allgemein werden diese Beanspruchungen im Entwurf nur grob abgeschätzt und nach Durchführung des Transportes erfolgt eine augenscheinliche Beurteilung hinsichtlich aufgetretener Risse oder anderer ungewollter Beschädigungen /27/.

Es wird deshalb die Frage nach quantitativen Meßwerten über derartige Transportbeanspruchungen, die als Grundlage einer genaueren Bemessung dienen können, gestellt.

2.3. Bei der Montage

Der Montagezustand verlangt eine besonders sorgfältige Durchdringung aller möglichen Beanspruchungszustände unter Beachtung einer ausreichenden Standsicherheit der einzelnen Abschnitte bis zur Fertigstellung des Gesamtbauwerkes. Im Gegensatz zur monolithischen Bauweise, mit ihren kontinuierlichen festen Verbindungen der Bauwerksteile, ist die Fertigteilbauweise durch Verbindungen der Einzelelemente charakterisiert, die im Montagezustand oft anders wirken als im späteren Gebrauchszustand oder die prinzipiell beweglich ausgebildet sind. Hierbei ist zu beachten, daß die Beanspruchungen infolge Wind und die Wirkung nur der Eigenmasse zu ungünstigen Lastkombinationen führen können, worauf Tümler /28/ hinweist. Grundsätzlich ist der Montageaussteifung besondere Aufmerksamkeit zu schenken.

Unter der Voraussetzung, daß das gewählte statische System und die angenommenen einwirkenden Kräfte während der einzelnen Montageabschnitte zutreffend sind, ist die Frage zu stellen, ob die rechnerische Sicherheit, konkret das Verhältnis des unteren Grenzwertes des Widerstandes zum oberen Grenzwert der Beanspruchung, kleiner als im Gebrauchszustand sein kann, da der Montagezustand relativ kurz (keine Einflüsse aus der Dauerbeständigkeit) und die Auswirkungen einer unvorhergesehenen Unterschreitung dieser Sicherheit in der Regel geringer als im Gebrauchszustand sind. Durch früher erkennbare größere Verformungen oder Rißbildungen können mögliche Bruchgefahren beseitigt, einzelne Fertigteile noch ausgewechselt oder verstärkt werden. Dies ist für die wirtschaftliche Durchführung der Montage z.B. mit besonderen Montageaussteifungen u.a. vorübergehender Maßnahmen von großer Wichtigkeit. Vielfach wird diese Frage von vornherein verneint mit der Begründung, daß im Bau- und Gebrauchszustand grundsätzlich die gleichen Sicherheiten gegen eine Gefährdung von Menschenleben gelten müssen. In einigen Vorschriften sind solche Festlegungen getroffen, wie z.B. in der DIN 1045 /29/, wo der Sicherheitsbei-

wert γ_m für den Montagezustand bei Biegung und Biegung mit Längskraft auf 1,3 abgemindert werden kann. Die internationalen Richtlinien zur Berechnung und Ausführung von Bauwerken des CEB-FIP /30/ sehen prinzipiell auch diese Möglichkeit vor.

Es soll deshalb hier die folgende Frage zur Diskussion gestellt werden:

Können die rechnerischen Sicherheiten im Montagezustand für einzelne Elemente, Abschnitte und das gesamte Bauwerk geringer angesetzt werden als für den späteren Gebrauchszustand?

Die Verbindungen der Fertigteile erfordern bereits im Montageablauf eine sorgfältige Einschätzung ihrer Wirkungsweise, weil häufig nur eine stufenweise Herstellung dieser erst im Gebrauchszustand voll funktionierenden Verbindungen erfolgt. Bekannt sind die übergreifenden Anschlüsseisen, die erst mittels Ortbeton eine kraftschlüssige Verbindung herstellen; Schweißverbindungen mit speziell einbetonierten Anschlußteilen oder Verschraubungen, die sofort voll oder teilweise tragfähig ausgeführt werden können; Verbindungen mittels Spanngliedern und andere, eine Übersicht hierzu gibt Rehm /31/.

Will man die Tragfähigkeit der Betonfertigteile im Bau voll ausschöpfen, so dürfen die Verbindungen nicht als "schwache Stellen" der Konstruktion diesem Bestreben entgegenwirken. Es folgt daraus, daß die Sicherheit und Wirtschaftlichkeit eines Bauwerkes aus Betonfertigteilen sehr entscheidend von der zweckmäßigsten Konstruktion dieser Verbindungen und ihrer unkomplizierten Ausführbarkeit bei der Montage abhängig sind. Die Einflüsse auf die Widerstandsfähigkeit dieser Fertigteilverbindungen während der Montage sind vielfältig. Es sind dies die Produktions- und Montageungenauigkeiten, die Festigkeiten des Vergußbetons oder anderer Vergußmassen, die Qualität der Schweiß- und anderer sofort kraftschlüssiger Verbindungen und die Witterungseinflüsse. Gleichzeitig gewinnen Verbindungslösungen bei der Herstellung von Fertigteilen aus verschiedenen Baustoffen wie z.B. Mehrschichtwandplatten an Bedeutung /32/ /33/. Für die Verbindungen gibt es in den einzelnen Ländern auf der Grundlage umfangreicher experimenteller Arbeiten z.B. /34/ /35/ /36/ /37/ (siehe auch Bild 3) eine Vielzahl von Zulassungen und Vorschriften z.B. /38/. In den meisten Fällen werden die experimentellen Ergebnisse durch Division mit einem Sicherheitsbeiwert zwischen $\gamma = 3$ und 5 für die Praxis in Form zulässiger Werte aufbereitet /39/.

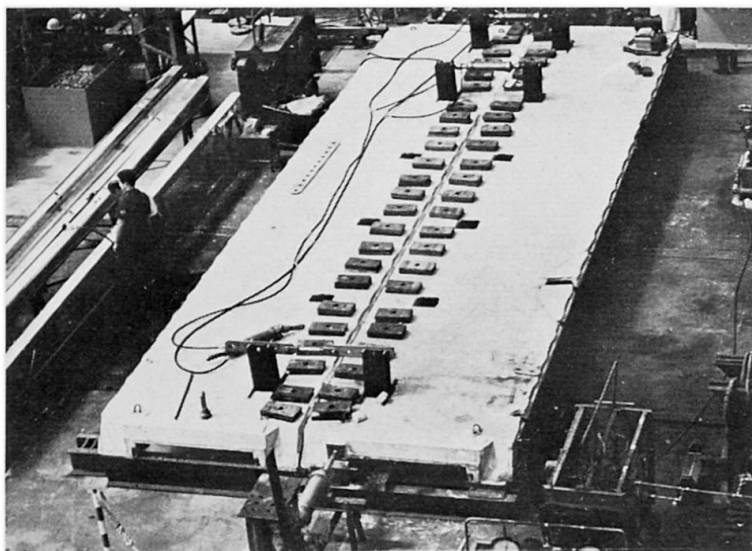


Bild 3

Erprobung der Scherfestigkeit der Fugen von 2,25 x 12 m π -Dachplatten /40/ zwecks Anwendung als schubsteife Dach-scheibe /41/

Für die Weiterentwicklung der Verbindungen im Hinblick auf die Sicherheit des Gesamtbauwerkes erscheint es notwendig, daß auch für dieses wichtige Konstruktionsdetail auf statistischer Grundlage beruhende Sicherheitsbetrachtungen (z.B. Beck /42/ und Struck /43/) angestellt werden.

Folgende Frage wird deshalb gestellt:

Welche Untersuchungsergebnisse an ausgeführten Bauwerken zur Wirkungsweise im Montage- und Gebrauchszustand und zur Dauerbeständigkeit der Fertigteilverbindungen können zur Qualifizierung der Sicherheitsbeurteilung mitgeteilt werden?

3. Sicherheit und Stabilität von Bauwerken aus Betonfertigteilen

Die in den letzten Jahrzehnten mit dem Bauen von Betonfertigteilen gewonnenen Erkenntnisse erlauben die Feststellung, daß die Bauwerke aus Betonfertigteilen bei Nutzung dieser vorliegenden Erfahrungen in gleicher Weise sicher hergestellt werden können, wie Ortbetonkonstruktionen. Man hat gelernt, die Bauelemente derart zu einem Ganzen zusammenzufügen, daß die anfänglichen Befürchtungen, die "schwachen Verbindungen" könnten zu einer Stabilitätsminderung führen, die eine größere Unsicherheit hervorruft, nahezu ausgeräumt werden konnten. Für verschiedene Bauwerksarten haben sich bestimmte in der Praxis bewährte statisch-konstruktive Systeme herausgebildet, die erfahrungsgemäß eine ausreichende Sicherheit gewährleisten.

Über die mit diesen Systemen gesammelten verallgemeinerungsfähigen positiven und negativen Erfahrungen, wie sie beispielsweise für eine Auswahl in den folgenden Abschnitten angedeutet werden, soll im Rahmen dieses Unterthemas berichtet und beraten werden.

3.1. Eingeschossige Gebäude

Die Hauptelemente, die den Hallenkonstruktionen die Standsicherheit gewährleisten, sind in der Regel: in Hülsefundamente eingespannte Stützen in Verbindung mit gelenkig aufgelagerten Bindern, Dach- und Wandscheiben sowie gesondert angeordnete Portale zur Längsaussteifung insbesondere bei Kranbahnen in Industriehallen.

Die Ausbildung einer schubsteifen Dachscheibe, wie dies in einfacher Weise mit großflächigen mit den Obergurten der Binder verbundenen Dachplatten möglich ist, verleiht diesen Konstruktionen eine relativ große Stabilität. Die Einbindung der Außenwände in die Längsaussteifung ist nicht so einfach, da die von den einzelnen Wandelementen infolge Temperaturänderungen hervorgerufenen relativen Bewegungen eine starre Verbindung mit den Stützen ausschließen. Die diesen Fertigteilkonstruktionen innewohnenden Systemreserven werden zur Verfeinerung der Berechnung immer besser genutzt. Auf der Basis der Arbeit von Benkert /44/ kann nach der DDR-Vorschrift TGL O-1045 /45/ die Knicklänge von eingespannten Hallenstützen entsprechend Bild 4 um 25 % verringert werden, wenn die Dachbinder mittels Mörtelfuge ≤ 25 mm Dicke auf den Stützenköpfen verlegt sind.

Die Abweichung der Lagerungsbedingungen von den Annahmen der Statik hinsichtlich der Lasteintragung und der Beweglichkeit können dagegen zum Abbau der Sicherheit führen. Hierbei wirken sich vor allen Dingen Verformungen aus Temperatur, Schwinden und Kriechen negativ aus und bedingen Auflagerschäden /46/ oder Schäden an den angrenzenden Bauteilen.

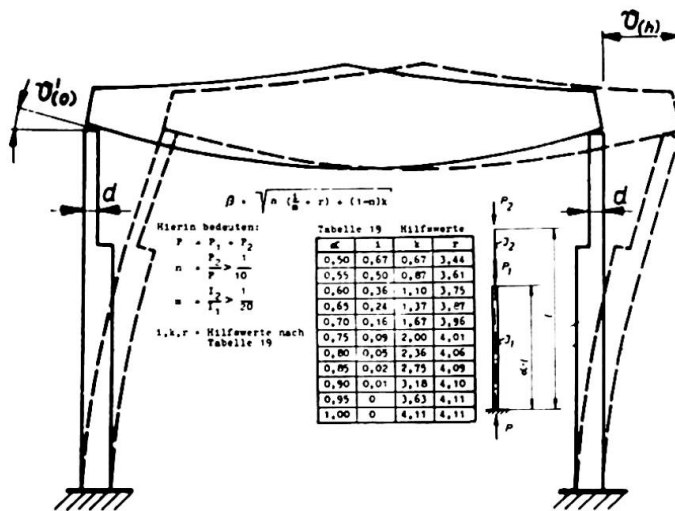


Bild 4

Ermittlung der
Knicklänge ein-
gespannter
Hallenstützen
/45/

Zur Diskussion ergeben sich folgende Probleme:

Welche Berechnungsmethoden werden zur Erfassung der Systemreserven von eingeschossigen Gebäuden benutzt?

Können Meßergebnisse über praktische Abweichungen von definierten Lagerbedingungen und daraus abgeleitete Verallgemeinerungen zur Qualifizierung der Berechnung mitgeteilt werden?

3.2. Mehrgeschossige Gebäude

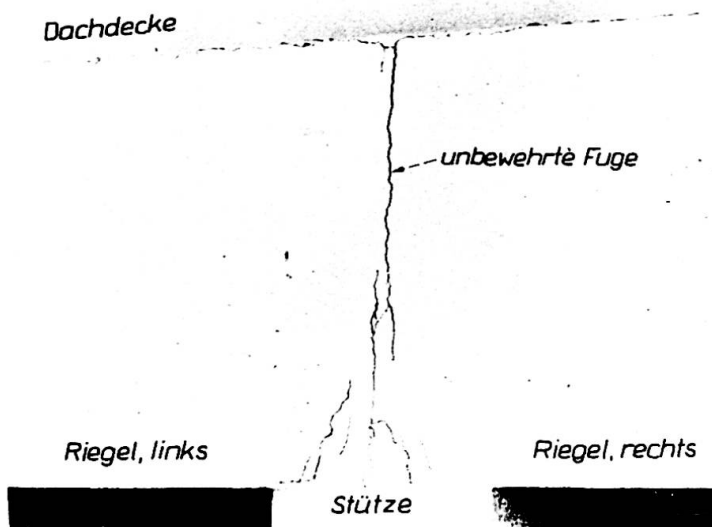
Für mehrgeschossige Gebäude aus Betonfertigteilen existieren eine große Anzahl von statisch-konstruktiven Systemen. Grundsätzlich kann in Skelett- und Wandbauweise getrennt werden je nachdem, ob die stabförmigen Stützen- und Riegelemente in Verbindung mit den Deckenplatten das tragende Gerüst bilden oder ob vertikale Wandscheiben zusammen mit horizontalen Deckenscheiben die Aussteifung und Lastabtragung übernehmen. Da sich diese beiden Bauweisen in ihrer Gestaltung der Fertigteile und ihrer Verbindungen wesentlich unterscheiden, werden sie getrennt behandelt, obwohl auch Beispiele bekannt sind, bei denen in einem Bauwerk beide Systeme gekoppelt wurden.

3.2.1. Skelettbauweise

Die Ausbildung biegesteifer Knoten, wie dies im Ortbeton der Regelfall ist, führt in der Fertigteilbauweise zu großen Aufwendungen. Es wurden deshalb Systeme für die statisch-konstruktive Durchbildung gewählt, die weitgehend gelenkige Verbindungen erlauben. Als Grundprinzip hat sich die Ausbildung von steifen Wandscheiben in Quer- und Längsrichtung oder zusammengefaßt zu einem biegesteifen Kern herausgeschält. An diese meist im Inneren des Gebäudes angeordneten Aussteifungen werden die ringsherum angeordneten Bauwerksteile in Form von steifen Deckenscheiben, gelenkigen Riegeln und Stützen angebunden. Vielfach werden auch Stützen als Fertigteile über mehrere Geschosse durchgehend angeordnet. Es ist einleuchtend, daß die Stabilität und Sicherheit eines solchen mehrgeschossigen Skelettbaues von der zweckmäßigen Wahl des statischen Systems und der konstruktiven Durchbildung der Verbindungen entscheidend abhängt.

Schäfer /47/ hat erste Ergebnisse einer allgemeinen Schadensanalyse im Stahlbeton dargelegt. Als häufigste primäre Schadensur-

sachen wurden u.a. zu hohe Beanspruchung durch Temperatur, Schwinden und Kriechen des Betons und Bewegungsbehinderung durch unwirksame Gleitschichten und Dehnungsfugen festgestellt. Dies trifft auch auf den Fertigteilbau zu, wie dies langjährige Anwendungserfahrungen in der DDR bestätigen (Bild 5). Grundsätzlich sind die



Auswirkungen einer nicht idealen Lagerung der Fertigteile (Auflagertiefe, Verdrehungsbehinderung, Eintragung von Zugkräften, Dehnungsbehinderung) und die Auswirkungen der Schnittkräfte aus der Systemwirkung des Gebäudes zu untersuchen. Dabei ist der Anordnung der Dehnfugen unter Berücksichtigung der Starrheit der Aussteifungskerne besondere Aufmerksamkeit zu schenken.

Bild 5 Rißbildung am Knoten Stütze-Riegel durch Temperatur und Dehnungsbehinderung

Für den Skelettbau wird infolge des plastischen Verformungsvermögens des Stahlbetons mit steigender statischer Unbestimmtheit die Versagenswahrscheinlichkeit reduziert /48/. Im Fertigteil skelettbau ist in der Regel die statische Unbestimmtheit durch die zweckmäßige Ausbildung von gelenkigen bzw. nur teileingespannten Verbindungen im Vergleich zum monolithischen Betonbau wesentlich geringer und damit diese Systemreserve nur bedingt nutzbar. Dem steht eine höhere Fertigungsqualität der Einzelelemente positiv entgegen. Für die Schnittkraftermittlung sind jedoch Abweichungen von den Idealisierungen des statischen Systems, z.B. außerplanmäßige Abweichungen der Stützen von der Systemlinie /49/, teilweise Einspannung der Stützen u.a. zu berücksichtigen.

Zur Diskussion werden damit folgende Fragen aufgeworfen:

Mit welchen sicherheitstheoretischen Methoden werden Abweichungen vom zugrunde gelegten idealisierten statischen System im Fertigteil skelettbau erfaßt ?

Welche Erfahrungen liegen hinsichtlich der Erfassung der Wirkungen von Schwinden und Kriechen sowie Temperaturänderungen im Fertigteil skelettbau vor ?

3.2.2. Wandbauweise

Die hauptsächlich für den industriellen Wohnungsbau entwickelte Wandbauweise, anfänglich für 5-geschossige Gebäude entwickelt, wird heute für Gebäude mit über 20 Geschossen angewendet (Bild 6).

Die aussteifenden Quer- und Längswände verleihen diesen Bauwerken in Verbindung mit den Deckenscheiben eine relativ große Stabilität, wenn die Verbindungen der einzelnen Wand- und Deckenelemente miteinander und untereinander ausreichend sicher konstruiert und ausgeführt werden.



Bild 6 Wohnhochhaus mit 24
Geschossen in Platten-
bauweise /50/

Ausbildung von profilierten Vertikalfugen gestattet die Betrachtung der Wandscheibe als monolithisches Bauteil, während bei glatter Fuge jeder Einzelstreifen nur für sich wirkt. Hierzu wurden zahlreiche Versuche und theoretische Untersuchungen durchgeführt, u.a. /54//55/ /56/ /35/ /36/. Für die Einbeziehung der nichtliniaren Einflüsse, die bei diesen räumlichen Faltwerken erst zur Beschreibung der wirklichen Verhältnisse führen, sind noch weitere Grundlagen über die plastischen Verformungen, das Rißverhalten und die Interaktionsbeziehungen zwischen Schub- und Normalkräften in den Fugen bereitzustellen.

Daraus folgt die Fragestellung:

Mit welchen Berechnungsmethoden auf der Basis welcher experimenteller Ergebnisse wird der wirkliche Spannungs-, Verformungs- und Bruchzustand des aus Decken- und Wandelementen mit Durchbrüchen zusammengefügt Bauwerkes als Grundlage der Sicherheitsbetrachtung mit der größten Annäherung ermittelt?

Wieweit stehen Grundlagen bereit, um eine umfassende Berechnung dieser Raumzellengebäude nach der nichtliniaren Theorie mit Erfolg durchführen zu können?

Für die praktische Berechnung der Plattenbausysteme wurden in den einzelnen Ländern spezielle Vorschriften aufgestellt u.a. /57/ /58/ /59/, die neben gesicherten Annahmen für die Berechnung auch konstruktive Maßnahmen zur Sicherung einer großen Dauerbeständigkeit umfassen. Das Europäische Betonkomitee -CEB- hat gleichfalls erste Empfehlungen für Großtafelbauten erarbeitet /60/ /61/. Das Einsturzungsglück am Ronan Point, einem 22-stöckigen Plattenbau, in London 1968 infolge einer Gasexplosion hat die Frage der Kettenreaktion bei Ausfall eines Baugliedes als spezielle Fragestellung der Sicherheit und Stabilität aufgeworfen und zu einer Reihe von Änderungen der nationalen Vorschriften geführt, die Manleitner /59/ andeutet. Grundsätzlich besteht Übereinstimmung, daß eine fort-

Der Massencharakter dieser Gebädekategorie hat dazu geführt, die Berechnung ständig zu verfeinern, um bei ausreichender Sicherheit geringsten Materialeinsatz für die tragenden und aussteifenden großflächigen Betonfertigteile zu ermöglichen. So ist deutlich der Trend erkennbar, die räumliche Tragwirkung dieser Zellengebilde der statischen Untersuchung zugrunde zu legen. Bekannt sind die Berechnungen mit dem Ersatzsystem nach Rosmann /51/, die Methode mit äquivalenten Rahmen oder Stabsystemen /52/ und neuerdings die Anwendung der finiten Elemente /53/. Die Genauigkeit dieser Berechnungsmethoden, d.h. wie wirklichkeitsnah die danach ermittelten Schnittkräfte und Verformungen sind, wird entscheidend von den getroffenen Annahmen über die Wirkungsweise der Fugen bestimmt. Ein besonderes Problem ist hierbei die Vertikalscheibenausbildung und -wirkungsweise. Die

schreitende Zerstörung durch Bruch eines Wandelementes möglichst auszuschließen bzw. ihre Auswirkung klein zu halten ist.

Hieraus ergibt sich die Frage:

Ist im rechnerischen Standsicherheitsnachweis generell der Ausfall einer oder mehrerer Wandplatten zu berücksichtigen oder reichen gegebenenfalls experimentell erprobte konstruktive Maßnahmen (z.B. Anordnung von Verbindungsbewehrungen) zur Vermeidung von Kettenreaktionen aus ?

3.3. Brücken

Für Brückenbauwerke aus Stahl- und Spannbetonfertigteilen sind folgende zwei Ausführungen typisch

- Balken über die gesamte Stützweite, die nebeneinanderliegend die Brückenbreite bilden und
- einzelne Querschnittssektionen, die im freien Vorbau oder in selteneren Fällen auf Hilfsrüstungen aneinandergespannt die Brückenlänge ergeben.

Brücken, die mittels ein Feld überspannender Fertigteilträger ausgeführt werden, bedürfen eines sorgfältigen Nachweises hinsichtlich der Querverteilung der Lasten und ihrer Wirkungen /62/. Häufig wird deshalb eine Ort betonplatte, die gleichzeitig als Fahrbahnplatte dient, ausgeführt /63/ /64/ bzw. werden die einzelnen Balken durch Quervorspannung, insbesondere für Eisenbahnbrücken, zusammengefügt /65/ oder die Querverteilung erfolgt nur durch spezielle Längsfugenausbildungen /66/. Die dynamischen Verkehrslasten und die klimatischen Umwelteinflüsse, die bei einem Brückenbauwerk nahezu auf alle Tragglieder direkt einwirken, erfordern zur Aufrechterhaltung der Sicherheit und Stabilität eine dauerbeständige Verbindung der Fertigteile. Mit der Wirkungsweise verschiedener Fugenausbildungen bei Anwendung von Quervorspannung und ohne Quervorspannung für Balkenreihenbrücken bei statischer und dynamischer Belastung hat sich Spaethe /66/ experimentell auseinandergesetzt. Neben konstruktiven allgemeingültigen Schlußfolgerungen stellt Spaethe fest, daß ein zuverlässiger Nachweis der Tragfähigkeit infolge der komplizierten Spannungsverhältnisse nur experimentell möglich ist. Bei ausreichend erprobten Verbindungslösungen im Fertigteilbrückenbau gibt es, wenn man von den im Abschnitt 2 behandelten Problemen absieht, keine anderen Sicherheits- und Stabilitätsprobleme, als bei ganz aus Ort beton hergestellten Brücken.

Es wird deshalb folgende Frage gestellt:

Können positive und negative Erfahrungen über die Dauerbeständigkeit des die Lastverteilung in Querrichtung sichernden Verbundes der Fertigteilträger angegeben werden ?

Für Brücken, die mittels Querschnittssektionen im Freivorbau oder auf Hilfsrüstungen hergestellt werden, werfen die Fugen zwischen diesen Fertigteilsektionen eine zusätzliche Frage zur Sicherheit in Ergänzung der allgemeinen Sicherheits- und Stabilitätsbetrachtungen monolithisch ausgeführter Brücken auf. Über die Ausbildung dieser Querfugen sind u.a. in /67/ /68/ /69/ Ausführungen enthalten. Es stellt sich nach jahrelangem praktischen Einsatz die Frage, ob sich die Annahmen über die dauerbeständige Wirkung dieser Querfugen in Brücken voll bestätigt haben.

Es wird deshalb gefragt:

Können Untersuchungsergebnisse mitgeteilt werden über die Dauerbeständigkeit der Wirkung von Querfugen zwischen solche Brückenfertigteilen, die als Querschnittssektionen montiert wurden.

3.4. Sonstige Bauwerke

Die Fertigteilbauweise kommt weiter im Wasserbau /70/, im Tiefbau, bei Sportbauten /71/ /72/ und bei vielen anderen Bauaufgaben häufig zur Anwendung. Jede dieser Fertigteilkonstruktionen hat spezifische Sicherheits- und Stabilitätsbedingungen zu erfüllen. Es werden deshalb Beiträge erwartet, die verallgemeinerungswürdige Erfahrungen über die Sicherheits- und Stabilitätsprobleme solcher Fertigteilbauwerke mitteilen.

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ZUSAMMENFASSUNG

Die Betonfertigteil-Bauweise kommt im Zuge der fortschreitenden Industrialisierung des Bauwesens heute für nahezu alle Gebiete der Baukonstruktionen zum Einsatz. Ihre spezifischen Sicherheits- und Stabilitätsprobleme besitzen deshalb grosse Bedeutung. In den vorstehenden Ausführungen werden die für Elemente bei der Herstellung, während des Transportes und bei der Montage auftretenden besonderen Probleme der Sicherheit und Stabilität durch vorwiegend praktisch orientierte Fragestellungen herausgestellt. Gleiches erfolgt auch für ausgewählte Gruppen fertiggestellter Bauwerke aus Betonfertigteilen.

SUMMARY

The construction of prefabricated concrete elements is applied for nearly all spheres of civil engineering, following the industrialisation in construction. Therefore their specific problems of safety and stability are very important. In the present paper the problems of safety and stability arising on the fabrication of the elements, on the transport and assembly are put up mainly with practical considerations. The same problems are underlined for selected groups of structures completed with prefabricated elements.

RESUME

La construction par éléments préfabriqués en béton est employée dans presque tous les domaines du génie civil, par suite de l'industrialisation poussée de la construction; les problèmes de sécurité et de stabilité qui en découlent, prennent de plus en plus d'importance. Les problèmes particuliers de la sécurité et de la stabilité sont mis en évidence par des questions d'ordre pratique relatives à la fabrication des éléments, leur transport et leur montage. Ces mêmes problèmes sont posés pour des ensembles choisis de constructions en éléments préfabriqués.

Developments in Manufacture and Assembly

Développements dans la production et l'assemblage

Entwicklungen in Herstellung und Montage

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1. Introduction

In construction of concrete structures of recent, there have been strong demands expressed for shortening of construction periods, saving of labor, and economization. There have also begun to be heard loud clamors for alleviation of public nuisances accompanying construction work such as environmental pollution, noise and obstruction of traffic flow. Minimizing work requiring cast-in-place concrete and effectively utilizing precast concrete elements instead may be cited as an excellent method of complying with these demands. Furthermore, through effective application of precast elements, there will be many cases when other advantages may be gained such as not only construction in cold weather being made easier, but also safety and durability of structural members being improved since concrete of high reliability will come to be used.

Consequently, there has been much research done from the past in countries throughout the world in regard to manufacture of precast concrete elements used in construction of precast structures and methods of assembly using elements, and these have been discussed at a number of international conferences (1)(2)(3)(4)(5)(6)(7).

This paper summarizes the developments seen in manufacture of precast concrete elements and methods of assembly using elements, and discusses the problems involved as well.

2. Developments in Manufacture of Precast Concrete Elements

The conditions required of concrete elements to be used for precast concrete structures may be listed as being (1) accuracies within specified limits of shapes, dimensions, and arrangements of reinforcement and jointing steel, (2) concrete possessing the required quality, (3) weights, shapes and dimensions suitable for transport, and (4) economy. These conditions will of course be of varying degree depending on the object for which the concrete elements are to be used.

Of the conditions mentioned above, the condition of (1) is highly important, and since it governs the success of a precast concrete structure, strict limitations on permissible errors are provided in product standards and construction specifications in all countries.

2.1. Improvements in manufacturing facilities for concrete elements

All new manufacturing plants for mass-produced concrete elements indicate that efforts have been made for improvements in facilities for concrete elements, such as for handling and storing aggregates and other materials, for batching and mixing, for fabricating and placing steel, for placing and consolidating concrete, for curing, and for conveyance inside and outside the plant of semi-finished and finished products during and after manufacture. Of the various facilities, the major ones are discussed below.

Among materials for concrete, aggregates require the most careful consideration in handling, and needless to say, it is important for measures to be provided in order to maintain gradations and moisture contents uniform. There are cases of good results being obtained at large-scale precast concrete plants where aggregates are suitably sieved and classified into several fractions according to particle size, with each fraction stored in an individual silo. It is of advantage to do so since it will become unnecessary to add devices to batching equipment for correcting batched quantities of water and aggregates in accordance with variations in surface moisture, while adjustments in concrete mix proportions accompanying variations in aggregate gradation can be almost completely eliminated.

When selecting mix proportions for concrete, it is a basic principle to make the unit water content as low as possible within the limits of obtaining workability suitable for placement, and since it is normal for considerably high-grade placing and consolidating equipment to be available in case of a plant manufacturing precast elements, concretes of dry consistencies having especially low unit water contents are commonly used. For this reason, pan type mixers with mixing blades which revolve inside pans are employed in many cases as mixing efficiencies are improved and mixing times are shortened, while quick discharge of concrete is possible. In particular, mixing efficiencies are good for models with planetary movement of blades.

Equipment for fabrication and placing of steel as well as forms differ greatly according to the kind of concrete element and it can be seen that much attention has been paid to each. At plants manufacturing large quantities of prestressed concrete elements there are many cases where pre-tensioning systems using long-line prestressing beds are employed. Many of these plants have facilities capable of providing curvatures to prestressing tendons as required. Further, the long-line system is also suited to manufacture of long prestressed concrete elements. Prestressed concrete piles of lengths of 60 meters and prestressed concrete girders of lengths of 40 meters or more are being manufactured by this method. Sliding forms are sometimes used in such operations. In case of piles, the hollow parts are formed by using inflated rubber tubes or sliding mandrels and there are cases of these mandrels being equipped with heating units to accelerate hardening of concrete.

When manufacturing prestressed concrete piles and poles by the pre-tensioning system, there are cases when the reactions accompanying prestressing are made to be carried by the forms.

With columns, girders, slabs, railway sleepers, various types of block, piles, etc., there are cases when all or part of a form is immediately removed after thorough consolidation of concrete of dry consistency. This is a rational system for surface finishing and curing also, and is adopted when conveying to the curing apparatus will not be a problem.

A considerable amount of manual work is generally required when assembly and arrangement of steel used for a concrete element are complex, and many kinds of automatic apparatus have been devised at various large plants to save on the labor cost involved. For example, treatment and fabrication of steel, assembly of spiral reinforcement to be used for piles, pipes and poles, and assembly of reinforcing bars for slabs and wall panels have been automated.

It goes without saying that it is extremely important to have powerful consolidation apparatus capable of thorough compaction in a short period of time when concrete of dry consistency is used. Consequently, precast concrete element manufacturing plants utilize various kinds of consolidation equipment each suited to a certain type of product. The various consolidation apparatus, when classified by principle, would be internal vibration type, external vibration type, vibration table type, tamping type, compression type, roller type and centrifugal type. The internal vibration type is used most widely with frequencies of vibration generally being in the range of 80 to 130 cycles per second. However, since consolidation capability is increased the higher the number of cycles, there are cases when vibrators with frequencies of around 250 cycles per second are used. Tamping types are often used for products like concrete block, roller types for large-diameter pipe, and very widely, centrifugal types for piles, poles and pipes.

When using external vibration types, vibration table types, centrifugal types, etc., there are cases in which good results are not obtained unless consolidation is carried out after first performing preliminary consolidation at low frequency or low speed.

When concrete of dry consistency is to be placed in elements of comparatively narrow cross section or in elements with arrangements of reinforcement which are not simple, there are cases when good results are obtained if the abovementioned methods are used in suitable combinations. For example, in case of placing concrete for pipes and large-diameter poles using vertical-type forms, it would be suitable for application of pressure and external vibration to be used in combination, while the combined use of external vibration would be effective in compression consolidation of sheet piles and slabs. This is because there is a tendency for the transmission of pressure to be hindered by interlocking of aggregate particles when only compression is used. Sometimes, extremely dry concrete of slump of zero is placed in slabs and girders and consolidation is performed using vibro-stampers which combine tamping and vibrating actions. And, to cause concrete of dry consistency to completely fill complex cross sections, there are cases when a special type of concrete placement apparatus equipped with external vibrators is used. This apparatus first applies vibrations to the concrete to make it fluid and then extrudes it into the form by pressure.

The system of transferring prestress in case of manufacturing pre-tensioned prestressed concrete beams, hollow floor panels and other elements by passing electric current through high-tensile steel to heat and expand the steel, fixing the steel to the two ends of a form or to jaws installed in a prestressing bed, and causing the form ends or jaws to bear the reaction accompanying cooling of the steel is used in the Soviet Union, Czechoslovakia and elsewhere⁽⁷⁾.

Other than the facilities discussed above, those for curing are also of importance, but since they have a particularly close relationship with concrete quality, they will be discussed in 2.2., "Improvement of concrete quality."

Uniformities of elements are improved and reliabilities increased through advances in manufacturing processes of precast elements at plants, especially automation, but that is not all — labor is saved to bring about economy and the merit of improving plant facilities lies in this effect. For example, at a certain prestressed concrete pile plant in Japan whose production capacity is as much as 480,000 metric tons per year, aggregates were recently separated into fractions while various facilities were automated as much as possible. As a consequence, the results of concrete control tests on specimens which were centrifugally consolidated and high-pressure steam-cured in the same manner as elements were as are shown in Fig. 1. The coefficient of variation of concrete strength during a period of a single month was only 3 percent indicating that the concrete was being controlled in an excellent manner. As for the operating cost after the facilities had been renovated, it is given in Table 1 and is lower than the cost in the past which had included a considerable amount for manual work. Naturally, the investment on the beforementioned apparatus was enormous — as much as 11,900,000 dollars — but the difference with the investment for the old plant can be recovered in about two years, and the benefit of automation can be clearly recognized. This may be only a single example, but it does serve to suggest that new capital investment in accordance with the scale of each plant will lead to economization.

Table 1. Comparison of Prestressed Concrete Pile Manufacturing Costs of New and Old Plants (Manufacturing Costs per Ton of Product)

Manufacturing Cost	New Plant	Old Plant
Principal materials	\$24.22	\$24.22
Auxiliary materials (fuel, supplies, other)	3.32	3.22
Labor	3.17	6.83
Electric power, repairs, other expenses	1.92	1.74
Depreciation	2.97	2.55
Total	35.60	38.56

Fig. 1 — Example of quality control test of concrete (for month of September, number of days operated, 21).

Elements used in various kinds of precast structures are not necessarily mass-produced articles which are sold in the market. In such case it would be difficult to install superior machinery and equipment as previously described, but it is thought necessary at least for concrete manufacturing facilities to be fully equipped for securing the required uniformity. Since it will be possible to assume smaller influence of strength variation and lower reduction coefficients when using precast elements made with such concrete, design strength will be improved to offer an advantage to the designer.

2.2. Improvement of concrete quality

2.2.1. Improvement of concrete strength

The design strength of a concrete element will differ depending on the purpose for which the element is to be used, and although cases of about 200 to 350 kilograms per square centimeter are greatest in number, there are also cases of about 400 kilograms per square centimeter, and recently, there have even been elements with high strengths of 800 kilograms per square centimeter or more which have come into practical use.

Increasing concrete strength markedly above present levels has been called for to a maximum degree in concrete engineering circles and this has been studied from the past by an extremely large number of researchers. According to the reports of these researchers, compressive strengths of 1,000 to 1,500 kilograms per square centimeter have been obtained, but most of these strengths were gained with small specimens of cement pastes and there have been very few studies which have progressed to the extent of practical use in concrete.

Yoshida reported in 1940 that compressive strength of 700 kilograms per square centimeter was obtained at the age of 6 hours and 1,040 kilograms per square centimeter at 28 days⁽⁸⁾. This was achieved using strong, hard aggregates and ordinary portland cement to make concrete of dry consistency and filling it in cylinder molds of 15-centimeter diameter. Pressure of 100 kilograms per square centimeter was applied to squeeze out excess water and air, and the concrete which had then become of water-cement ratio of about 0.23 was cured for three hours immersed in boiling water while still under pressure. This technology was first put into practical use in part of the shield segments for the Kanmon Undersea Railway Tunnel (completed in 1942) of Japan, while recently, it is being widely utilized in manufacture of reinforced concrete segments for subway projects in urban areas of the country and of concrete sheet piles. In case of products being sold on the market, compressive strengths of around 400 kilograms per square centimeter at the age of 5 hours and 750 kilograms per square centimeter at the age of 28 days are being obtained by steam-curing under pressure after pressurizing for 6 to 8 minutes at 8 to 10 kilograms per square centimeter while employing simultaneous external vibration.

Roy and Gouda have reported that compressive strength of 6,700 kilograms per square centimeter and tensile strength of 650 kilograms per square centimeter were obtained with cement paste at the age of 28 days on application of pressure of 3,500 kilograms per square centimeter while maintaining the paste at a temperature of 250°C⁽⁹⁾. The water-cement ratio was 0.09 with the minimum value of paste porosity a mere 1.8 percent. The paste consisted of the outer sides of extremely compacted unhydrated cement particle groups surrounded by dense cement gel. In any case, such high strengths are very much worthy of attention.

Short-term strengths also comprise an important factor of concrete elements for shortening periods of time required for stripping forms, for transferring prestress, and for making possible handling, conveying and assembling at an early time. With elements which are mass-produced, there is a strong tendency for rise in strength over the long term to be sacrificed and increase in short-term strength to be aimed for.

As means of increasing short-term strengths of concrete elements, use of high early-strength cement, use of admixtures, reduction in water-cement ratio, utilization of high-performance consolidation apparatus, atmospheric-pressure steam curing, high-pressure steam curing, etc. are conceivable. However, since most manufacturing plants already have accelerated curing facilities, ultra-rapid-hardening cements and set-regulated cements are not used very much from the standpoint of running costs. A method of heating concrete during mixing to 65 to 75°C using a special type of mixer which is heated with steam has been adopted in some cases⁽¹⁰⁾. Such concrete is said to attain approximately 60 percent of 28-day compressive strength at the age of three hours. In attempting to gain strengths as intended by this method, there are such drawbacks as the difficulty of controlling temperature and the necessity of placing concrete within 10 minutes after finishing mixing. However, in case of continuing with steam curing, there is no trouble even if the temperature were to fall to about 50 to 40°C, and for the reasons that the time for surface finishing is shortened and labor costs are saved, this method is being used to a considerable extent.

In regard to utilization of various admixtures, the development of the technique of economically producing extremely high-strength concrete through large-quantity use of high-performance water-reducing agents may be cited as being especially noteworthy. Such water-reducing agents are mainly constituted of polyaromatic sulfonates and all of them cause hardly any air to be entrained, while retarding effects are comparatively minor. When these water-reducing admixtures are added at rates of about double the standard quantities and concrete is mixed, prominent water-reducing effects are demonstrated and unit water content of concrete can be reduced by approximately 30 percent.

This type of technology was developed in manufacture of prestressed concrete piles with the purposes of increasing resistance through high strength and eliminating damage accompanying driving of piles. Compressive strengths higher than 900 kilograms per square centimeter have been attained using combined atmospheric-pressure and high-pressure steam curing on hollow piles which had been made with concrete using large quantities of water-reducing agent thoroughly compacted by applying high-speed revolutions at centrifugal acceleration of approximately 30g⁽¹¹⁾. In case of using good-quality aggregates, it is easy for strength at one day to be raised to around 1,300 kilograms per square centimeter. Approximately 2,200,000 tons of this type of concrete pile were manufactured at numerous plants in Japan in 1973 and were used in various construction projects. This technology has begun to be utilized not only for prestressed concrete piles, but also in manufacture of other types of precast elements such as members of trusses, and a concrete truss railway bridge with a span of 45 meters has recently been erected by assembling such concrete members.

In basic research carried out by Yamamoto at the University of Tokyo, it was shown that in case of concrete using ordinary portland cement at unit cement content of 500 kilograms per cubic meter, admixture of water-reducing agent at a rate of one percent by weight of cement, and curing at 21°C, high strengths of 430 kilograms per square centimeter at the age of one day and 950 kilograms per square centimeter at 28 days could be obtained. The water-cement ratio of the concrete was only 0.27, but the slump was approximately 12 centimeters.

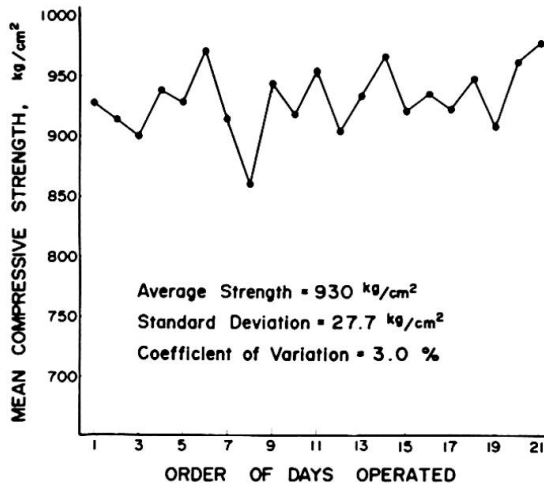


Fig. 2 — Truss railway bridge of Sanyo Super-express Line.

Furthermore, it was found that there were practical limits to unit cement content and dosage of water-reducing admixture; in this case, when making comparisons on the basis of concretes with equal workabilities, there was little gain in strength even if unit cement content were raised above 500 kilograms per cubic meter, unit water content was also a minimum when unit cement content was roughly around 500 kilograms per cubic meter, there was not very much effect of increasing water-reducing admixture beyond a certain extent (1 percent in this case), etc., and it is suggested that there would necessarily be an appropriate mix design in using such an admixture⁽¹²⁾.

Concrete with unit cement content above a certain degree and with addition of a large quantity of a water-reducing admixture which is thoroughly mixed, besides indicating high strength of about the level mentioned above without any special curing, will demonstrate prominent mobility under the action of a vibrator even though the outward appearance of the concrete may be that of dry consistency, and therefore, will be easy to consolidate with very little segregation of materials. Consequently, this type of high-strength concrete can be used widely for various kinds of elements and it is expected to become widely utilized. Fig. 2 shows a double-track railway bridge truss for a new super-express line using reinforced concrete members cast with this type of high-strength concrete, cured at outdoor temperature, and assembled by prestressing. The two examples of railway bridge trusses, including the one of 45-meter span previously cited, may be noted as cases of lightening dead weight through the use of slender members of high-strength concrete in structures subjected to extremely frequent repetitions of live loading. The decisions to use slender members were made after careful studies of fatigue resistance properties.

2.2.2. Accelerated hardening of concrete

There are various drawbacks to steam curing which are conceivable such as that hardly any strength gain of concrete at long-term age can be expected, that there is risk of microcracks being formed due to internal strains produced by the differences in thermal coefficients of expansion of the various materials used for elements such as cement, aggregates, water, air bubbles and steel, and that there is danger of cracks being produced during the process of temperature change, especially from temperature and moisture differences between interiors and surfaces of elements occurring with falling temperature, but it is being very widely used in manufacture of various kinds of concrete elements since the economic effects are prominent. Both drying shrinkage and creep are smaller for steam-cured concrete elements compared with cast-in-place concrete. For example,

even in case of atmospheric-pressure steam curing, drying shrinkage is about 30 percent and creep about 50 percent smaller. This would be of advantage for a structural member.

Venuat showed that the cement hydration products of cement steam-cured at atmospheric pressure are of practically the same kinds as the hydration products in case of curing at 20°C, but that differences can be recognized in the morphologies of the hydrates, sizes of crystals and porosities, and that the features of the various physical properties of steam-cured concrete can be explained from these differences⁽¹³⁾.

The important conditions in case of performing atmospheric-pressure steam curing are such as the presteaming period, rate of temperature rise, maximum temperature and duration of maximum temperature period, and rate of temperature fall, with the quality of a concrete element varying greatly according to these conditions, and it is necessary for these four conditions to be selected as suited in order that the purpose of curing can be achieved economically in accordance with the purposes of use, shapes and dimensions, materials used, and mix proportions. If maximum temperature were to be raised and duration of maximum temperature period prolonged within suitable limits, strengths would naturally be increased. Consequently, the curing cycle for achieving the desired purpose will differ according to the case. However, there are many countries where standards for these conditions have been established considering concrete elements marketed in general, among which there are cases requiring moist curing for several days after having performed steam curing. The recommendations of the American Concrete Institute on steam curing⁽¹⁴⁾ are very good and there are many other countries which are utilizing them. These recommendations provide that maximum temperature in general should be between 66 and 82°C with rise in temperature not more than 22°C per hour in case of presteaming period of 3 hours and not more than 33°C per hour in case of 5 hours.

In case of concrete made by hot mixing, steam curing may be performed raising the temperature immediately to 80 to 90°C since the temperature of the concrete will already be up to a level of 40 to 50°C. The curing period will therefore be extremely short and in most cases this will be economical.

With high-pressure steam curing, compared with atmospheric-pressure steam curing, high strength of concrete is obtained at an earlier age, drying shrinkage is reduced, chemical resistance is increased, and efflorescence is reduced. This is said to be because the tobermorite formed during curing by this method is more dense than the tobermorite gels produced in the cases of atmospheric-pressure steam curing and normal moist curing. In case of performing moist-air curing or atmospheric-pressure steam curing, the aggregates in concrete are inert except for special kinds, while in case of curing in high-pressure steam of pressure of approximately 7 kilograms per square centimeter or higher, tobermorite of high silica content is produced in which lime and silica in the cement, water, and even silica in silicate admixtures and aggregates are combined. The previously mentioned superior properties are imparted because this tobermorite is hydrous calcium silicate which is strong and dense. In the event that high-pressure steam curing is to be performed on cement paste, it is necessary for a silicate admixture to be included. However, there are also cases which differ and the use of silicate admixtures will not be necessary when performing high-pressure steam-curing concrete. In essence, there are cases when the silicate in aggregates alone will be adequate⁽¹¹⁾.

The cycles of high-pressure steam curing must be determined suitably in accordance with the specified strengths, dimensions, materials used, concrete

mix proportions of elements, etc., while steam curing is divided into two stages in most cases. In effect, presteaming is omitted with atmospheric-pressure steam curing performed as first-stage curing and high-pressure steam curing as the second stage. The various properties of concrete subjected to high-pressure steam curing are governed according to the second-stage curing while it appears the influences of the various conditions at the first-stage curing are small⁽¹⁵⁾.

In case of prestressed concrete piles (made by a pre-tensioning system with diameters not more than 0.6 meter and lengths not more than 15 meters) in Japan using the concrete described previously, compressive strength of 900 kilograms per square centimeter are being secured on completion of curing. Atmospheric-pressure steam curing is performed for approximately 10 hours at temperature between 60 and 70°C. Prestress is transferred, forms stripped, the piles are placed in an autoclave and the temperature is raised to 180°C in 3 to 5 hours. High-pressure steam curing is then carried out again for 3 to 5 hours at a temperature of 180°C and pressure of 10 kilograms per square centimeter. The rate of temperature drop is made to be as small as possible in accordance with the cross-sectional area of the pile. The report⁽¹⁵⁾ that when high-pressure steam curing is carried out on concrete using a large dosage of high-performance water-reducing admixture, the tobermorite produced is particularly dense due to the dispersion effect on cement particles is of special interest.

2.2.3. Weight-reduction of concrete

When high-strength concrete is appropriately utilized, the cross sections of structural concrete members can be made smaller and the dead weight of a concrete structure can thus be lightened. However, a more effective means of reducing dead weight is the utilization of lightweight concrete. An advantage brought about by reduction in the weight of a structure other than lightening of the superstructure is simplification of the foundation, and this advantage becomes greater especially when bearing capacity of the ground is small. Furthermore, with a long-spanned concrete bridge, there will additionally be simplification of piers, abutments and shoes. In case of precast concrete elements, lightweight concrete will be of still more advantage as the costs of transporting and assembling will be reduced. Since the cost of artificial lightweight aggregate is much higher than that of normal-weight aggregate, the economy of the case of using lightweight concrete elements should be judged upon careful comparison of the abovementioned advantages and materials costs, but it is felt there will be many cases when the use of lightweight elements will be economical⁽¹⁶⁾. Lightweight concrete elements are excellent in the properties of heat insulation and sound absorption, and are being used to a considerable extent as wall, floor and roof materials of various types of buildings.

The variety of lightweight aggregates used for lightweight concrete is very great. Not only do the qualities vary, but also the properties differ with each country and each region even for aggregates belonging to the same classification. Therefore, to attempt to clearly outline what will be the unit weights, strengths, durabilities and other physical characters of concretes using these lightweight aggregates is a difficult matter. However, when lightweight aggregates of comparatively good qualities are used, concretes of unit weights between 1.6 to 2.0 tons per cubic meter and compressive strengths between 350 to 500 kilograms per square centimeter can readily be obtained so that they are even used in prestressed concrete bridges. The modulus of elasticity of a lightweight concrete will naturally be low, but with regard to the amounts of shrinkage and creep, whereas there are reports that these were as much as 1.5 to 2.0 times those of normal-weight concrete, there are reports on the other hand stating that for practical purposes there were no significant differences. This

discrepancy is probably due chiefly to differences in the qualities of the aggregates. Since it is inconceivable for drying of civil structures standing outdoors to progress in the same manner as drying in the laboratory, there will be cases when it will be unrealistic to consider laboratory values of shrinkage and creep for design. However, as cracking due to difference in moisture content between surface and interior portions of concrete is liable to occur, it would be proper to provide additional reinforcement or combined use of natural aggregates to a suitable degree in case of precast elements to be employed in an important structure. Also, it is suitable for more thorough reinforcement to be provided than for normal concrete at places where stress concentration will be produced.

2.2.4. Self-stressing concrete

The method of transferring prestress to concrete through restraint of expansion of concrete containing expansive cement by forms or steel is beginning to come into practical use for various structures such as floor and roof slabs of buildings, water tanks, pressure pipes and pavement slabs. This method is being actively used especially in the Soviet Union, but in most cases, the prestress transferred is not more than 40 kilograms per square centimeter.

Prestressed pressure pipes are representative of precast concrete elements utilizing self-stressing concrete and have found widespread practical use. In order for self-stressing concrete to be extensively used in other types of precast elements, there will be accurate control of prestress needed, while problems requiring clarification still remain such as loss of prestress accompanying shrinkage and creep of concrete, but this is a material which is important for saving labor and economizing in construction of precast structures. Moreover, as will be described later, this is without any doubt an extremely useful material for joining precast elements.

2.3. Standardization of concrete elements

All industrial products will show remarkable development as a result of standardization, and precast concrete elements are not exceptions. As they have become standardized in various countries, mass production has progressed to make possible lowering of product costs, and general use and development have been realized. Although the details of standardization differ with each country, materials, manufacturing methods, testing methods, etc., are specified for many kinds of concrete elements.

One key to making design of a precast concrete structure economical may be said to be in suitably joining together standard concrete elements which are widely being marketed or making composite members out of elements and cast-in-place concrete. For the designer, in such case, it will be desirable for an element to be usable for several other purposes besides its main purpose. For example, a prestressed concrete foundation pile can be used as all or part of a bridge pier, or as a structural column of a building. Consequently, there will be cases when it would be advantageous to slightly modify designs of structures in accordance with the shapes and dimensions of elements available on the market.

It is desirable for standardization of precast elements to be done not on a mere national basis but on an international scale. This will be extremely difficult to accomplish, but at the least, it is wished to internationally unify standards for materials used in construction of structures and testing methods of elements. However, if standardization of elements were to go ahead, a trend

to hinder revolutionary progress may be produced because the natures of manufacturing facilities, materials and practices will tend to become fixed, and it will be absolutely necessary to take precautions that such a bad effect will not result.

3. Developments in Assembly of Precast Concrete Elements

3.1. Joining of precast elements

It is not an overstatement to say that design and construction of joints between precast concrete elements, connections between precast concrete members, and connections between precast concrete elements and cast-in-place concrete members comprise the most important problem in construction of precast structures. All of these joints and connections must be such that the required precisions will be possessed, that there will be no overstressing under design load, and that the required safety factors against failure will be secured. Also, it is necessary that construction of joints and connections can be done quickly and economically.

Materials used for joints in making prestressed concrete members by connecting precast elements may be broadly divided into the three varieties of concrete, mortar and epoxy compound, the last being in essence a mixture of epoxy resin and hardening agent. Joint construction using concrete or mortar is a method which has been widely employed from the past. The work is easy, and generally, it is the most economical method. However, since prestress cannot be transferred until the concrete or mortar has hardened to a certain extent, there is the great shortcoming that the construction period will be prolonged. Nevertheless, it is possible to transfer prestress immediately even when using mortar in case elements are to be joined one on top of another, and there have been cases of good results obtained by setting the upper element on stiff mortar thinly spread across the top of the lower element, vibrating the elements to transmit vibrations to the mortar to increase its mobility and make it spread uniformly throughout the joint surface, stopping the vibration when the thickness of the mortar had become reduced to a specified amount, and immediately transferring prestress. The method of using an epoxy compound has been devised to shorten construction periods, but since epoxy is an expensive material, in addition to which its deformation is large, it is desirable for joint thickness to be small. However, if the joint thickness is to be made very small, it will be necessary to finish end surfaces of elements to be joined so that they will fit together perfectly, but this would be almost impossible to accomplish. Therefore, a method using epoxy compound mixed with filler of fine sand or cement has come into use with joint thickness being one to two centimeters, but in case of this method also, transfer of prestress must wait until the epoxy has hardened.

The coming into practical use of a jointing method using only epoxy compound and no filler may be cited as one of the remarkable advances made in recent prestressed concrete bridge construction. In this case, in order to cause the ends of two elements to be joined to fit perfectly, when manufacturing the elements, separating material was coated on the end of one element to be joined and concrete was cast up against this to use it as a form. Consequently, elements were required to be manufactured on a precasting bed matching the intrados line of a bridge, and the operations involved were troublesome in such a case as when the intrados profile of a long bridge happened to vary. However, since there were perfect fits between elements at the joint portions, the thickness of epoxy

could be made extremely thin, and upon fitting, it was possible to transfer prestress immediately. Accordingly, the construction period required for jointing was shortened to about one fifth of that for the ordinary case of using mortar joints. Subsequently, it has been devised so that adjustments could be made even if the intrados or some other feature of a bridge should vary, or in case an error should be produced in erection, by referring to the results of computer calculations and using one fixed mold and two movable molds. Thus, success has been achieved in simplifying the abovementioned troublesome work. Cantilever erection without using shoring is facilitated going by the method of using epoxy only.

There is an extremely great variety of joints and connections being used which do not rely on mechanical prestressing, and broadly divided, they are those utilizing welding, bolting or other means of connection between steel provided at ends of elements or members, those depending on bond or anchorage in concrete of steel protruding from ends of elements or members, those employing sleeves, those using the likes of epoxy compound, those embedding ends of members in cast-in-place concrete, and others, with various innovations provided depending on the respective purposes. There are also cases of good results having been obtained by filling self-stressing concrete or mortar between members to be joined.

3.2. Assembly

With the recent advances made in concrete technology and the rapid progress evidenced in cranes, trucks and other machinery, precast concrete elements and their assembly have become of extremely large scale. This is in response to the demands of society for rapid construction and saving of labor. To take a bridge using precast concrete elements as an example, Tiel Bridge in the Netherlands is a cable-stayed concrete bridge of a total length of 612 meters having a center span of 267 meters. The approach viaduct of this bridge is a continuous structure consisting of ten spans each of 78.5 meters and a cantilever of 22 meters. The viaduct was constructed by cantilever erection of precast elements using epoxy compound, and each element had a length one fifteenth of span length and a weight of 120 tons. It is said the stays of the main bridge were made of prestressed concrete assembled by connecting precast concrete elements 5.15-meter long with joints of cast-in-place concrete. This was for protection of prestressing cables and for increasing rigidities of stays.

Precast concrete bridges built by cantilever erection of precast elements are being constructed at many places throughout the world and it is not unusual for center spans to exceed 100 meters in length. When rivers or sea routes can be utilized for transportation, the use of much larger elements is permissible, and for the center of Tiel Bridge, four suspended lightweight concrete girders each of length of 65 meters and weight of 425 tons were used, and these were erected on hauling by barge.

The reason cantilever erection is recommended is because there are such advantages as suitability for construction across deep canyons and over seawaters, no obstruction of river water flows or highway and railway traffic flows since shoring can be omitted, and because rapid erection is possible. However, when only direct construction costs are compared, cantilever erection is generally more expensive than cases of providing shores and making connections on top of falsework on the shores with joints of concrete or mortar, or cases of elements connected at some other site with joints of concrete or mortar and installed using erection trusses or other equipment, and it is particularly more expensive in case of a short bridge. On the other hand, the shortness of the construction period results in the benefits of earlier opening for use and

reduction of interest on investment, while the benefit accompanying reduction in obstruction of traffic flow is also substantial. Therefore, in selecting the method of construction, the true economics cannot be judged unless careful study is made of the net result of these benefits and the direct construction costs.

The discussion above is on only one example — there is much research activity going on in many fields on design and execution of assembly according to the type of precast structure, and these studies are unquestionably making steady advances. Namely, research is being conducted on methods of rapid yet safe assembly according to type of project, conditions of work site, period of construction, and construction cost and construction equipment as well. If circumstances should permit, it will naturally be advantageous to reduce the number of joints by using large-sized elements. Accordingly, elements have increased in size as advances have been made in equipment for their erection, particularly cranes, and there has even been a case of a precast reinforced concrete well weighing approximately 1,800 tons used for the pier of a marine bridge.

It is thought that offshore structures will be important precast concrete structures to be developed from here on, and besides marine bridges already mentioned, marine airports, marine power stations and marine petrochemical complexes can be cited as not-too-distant examples for development which come to mind. It may be said that construction of these types of structures will be impossible unless precast concrete is utilized since work on these structures must proceed while protecting the ocean environment. It is thus thought that even more development of precast structures will be seen in the future.

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SUMMARY

Recent facilities and practices for manufacturing precast concrete elements and new developments in assembly of elements are summarized and the problems involved are discussed. It is pointed out that extra-high-strength concrete can be readily obtained through use of a high-quality water-reducing admixture and the performance in actual use is described.

RESUME

La contribution présente les développements récents dans la production d'éléments préfabriqués en béton ainsi que dans l'assemblage de ces éléments. Les problèmes soulevés par cette évolution sont discutés. On souligne qu'une résistance exceptionnellement élevée du béton peut être facilement réalisée par l'utilisation d'un produit de haute qualité diminuant la quantité d'eau nécessaire. Les résultats obtenus sont décrits.

ZUSAMMENFASSUNG

Im vorliegenden Beitrag werden die neuesten Möglichkeiten und Verfahren zur Herstellung vorfabrizierter Betonelemente und neue Entwicklungen in Bezug auf die Montage zusammengefasst und die damit verbundenen Probleme diskutiert. Es wird hervorgehoben, dass extrem hohe Betonfestigkeiten leicht durch den Wasseranspruch reduzierende Rezepte erzielbar sind. Entsprechende Ergebnisse bei der praktischen Anwendung werden beschrieben.

New Applications including Submerged and Floating Structures

Utilisations nouvelles, comprenant les constructions sous-marines
et flottantes

Neue Anwendungen einschliesslich Unterwasserbauten und schwimmende
Konstruktionen

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1. INTRODUCTION

Precast concrete, as applied to structures, may be reinforced or it may be prestressed. The precast concrete elements may be produced on the construction site or they may be plant-manufactured and transported to the construction site.

The technique of precasting concrete structures has been known for more than 100 years. In fact, the first recorded example of reinforced concrete is a small boat built in France by Joseph Lambot in 1848. One may wonder why precast concrete has not been more extensively used since Lambot's time. Perhaps the comparative ease of building wooden forms on the site and the placement of wet concrete by relatively unskilled labor with simple tools resulted in the lowest construction cost. Many entrepreneurs with limited capital have competed successfully employing transient labor utilizing simple tools.

In contrast to dry-land construction, submerged and floating structures have not been adaptable to in-situ concrete methods. For technical as well as economic reasons, structures located under the sea as well as floating structures have been precast in special facilities equipped with graving docks or launching ways. Production equipment such as batching and mixing plants, large capacity cranes, stressing beds and permanent steel forms are used.

The permanent nature of a precast/prestressed concrete enterprise depends on management by technical specialists and steadily employed skilled production workers.

Factory-produced precast concrete can reach very high strengths (500 to 800 kg/cm²) and, when prestressed with high tensile strength steel, impressive gains in economy and structural performance are attainable.

2. SUBMERGED STRUCTURES

Precast/prestressed concrete piles for bridges and harbor structures in sea water have demonstrated performance with minimum maintenance problems. To avoid cracking, concrete piles are pre-

stressed in the range of 60 to 90 kg/cm². When exposed to sea water, dense, well-compacted concrete with a water-cement ratio of 0.38 to 0.42 is recommended for durability and corrosion protection for the reinforcing steel.

Submerged sewers constructed from precast/prestressed concrete, such as the Hyperion Outfall in Los Angeles and the Lake Washington Interceptor Sewer in Seattle¹ (Fig. 1) are outstanding examples.

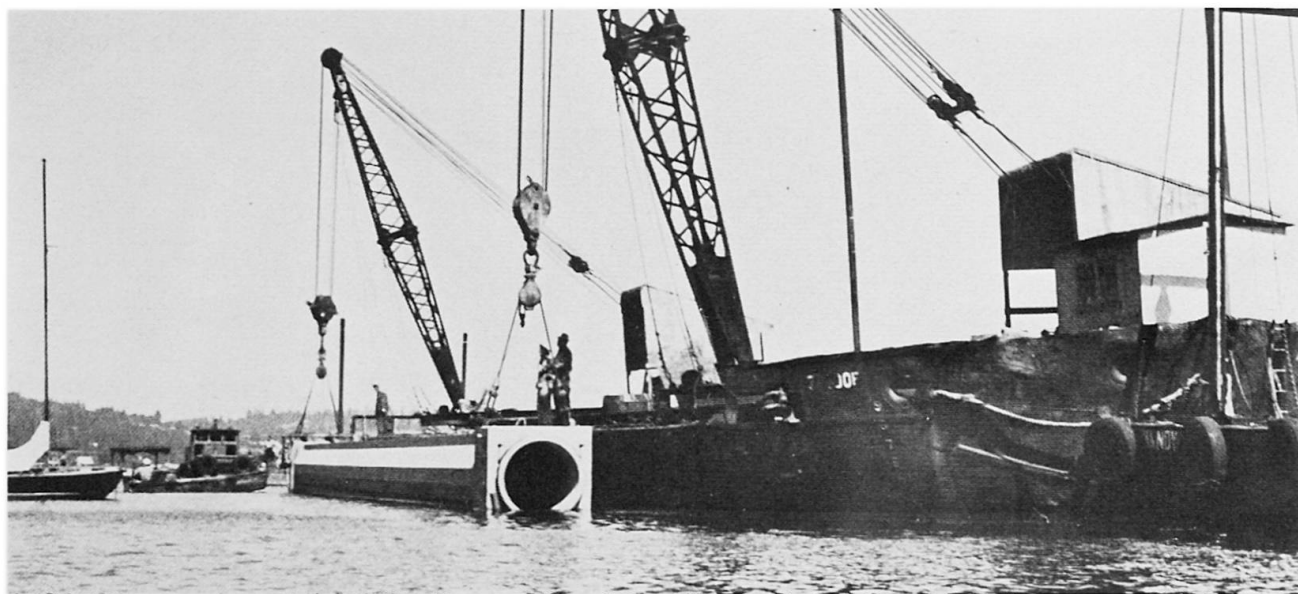


Fig. 1: Metro sewer - Lake Washington Interceptor Sewer in Seattle, Washington.

Large caissons for subaqueous tunnels have been precast in dry basins. The caissons are launched by flooding the basin. When afloat, they are towed to the site where they are submerged to their final location (Fig. 2). Good examples of highway and rail-

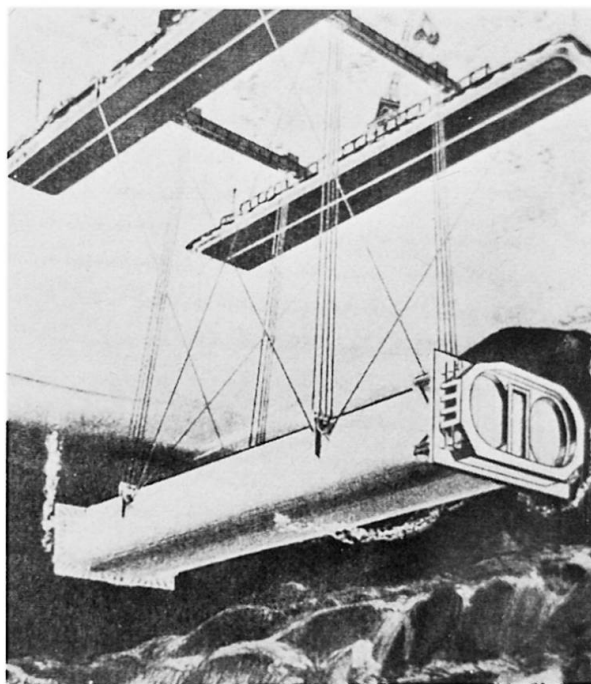


Fig. 2: San Francisco Rapid Transit Tube being submerged to its final location.

way tunnels include the La Fontaine Tunnel at Montreal, the San Francisco Rapid Transit Tube² (Fig. 2) and the new Tokyo Bay Loop Expressway Tunnel.

The latter is a submerged tunnel 1 km long, containing 9 precast caissons. Each caisson measures 115 x 37 x 9 m, and weighs 38,000 tons. The elements were precast in a dewatered basin 645 m long and 126 m wide. When all caissons were completed, the basin was flooded and the precast concrete elements were floated and towed to their final location and then sunk to the bottom.

Some recent and dramatic examples of submerged precast structures are to be found in the North Sea. The first of the giant oil drilling and production facilities is Norway's Ekofisk I,³ which was completed in June 1973. Shown in Fig. 3, this structure contains 83,000 m³ of concrete, 8,600 metric tons of reinforcing bars and 3,300 metric tons of half-inch diameter prestressing strand. Ekofisk I was constructed in Stavanger and towed 480 km to its designated location in the North Sea and then lowered to the bottom in 70 m of water.

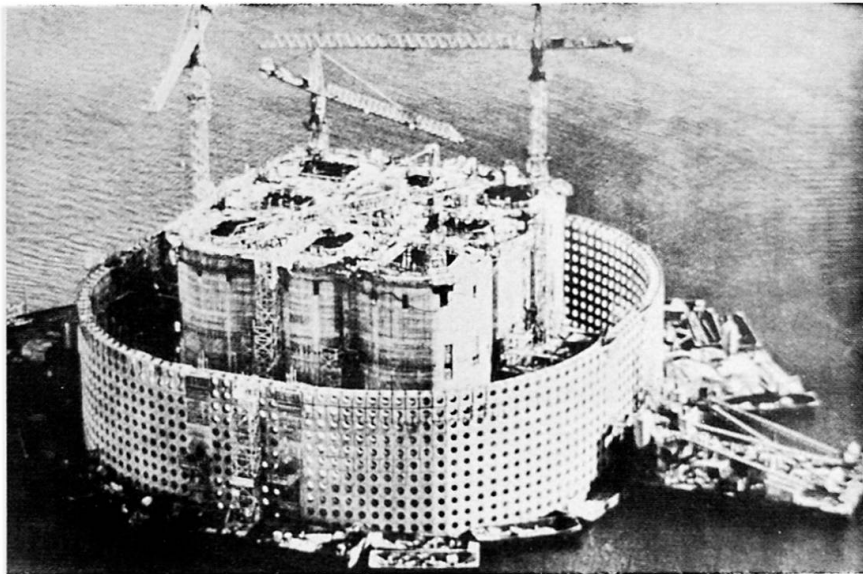


Fig. 3: Ekofisk I, oil drilling and production facility for use in the North Sea.

Immediately following Ekofisk I, the Stavanger facilities have started construction of a series of multicell precast concrete oil production facilities called Condeeps (Fig. 4). Like Ekofisk, the Condeeps (four under construction in 1974) are started in a dewatered basin 10 m below sea level. When constructed to floatable depth, the basin is flooded and the partly-complete elements are moved into the Stavangerfjord where construction continues while they remain at anchor in deep water. (Fig. 5) The nineteen cylindrical cells are concreted in slip forms to a height of 80 m. Each cylinder is closed with a dome roof. Rising above the 19 cells are three slip-formed concrete legs which support the operating platform.

Another precast concrete submerged oil facility, the Selmer Tripod (Fig. 6), is being developed in Norway. Designed for a water depth of 130 m, it has an oil storage capacity of 1 million barrels.

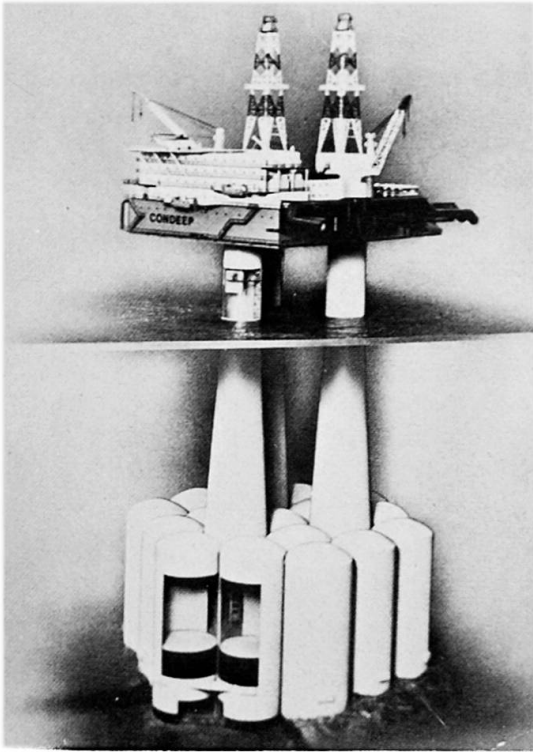


Fig.4: Condeep, Norway's multi-cell precast concrete oil production facility.

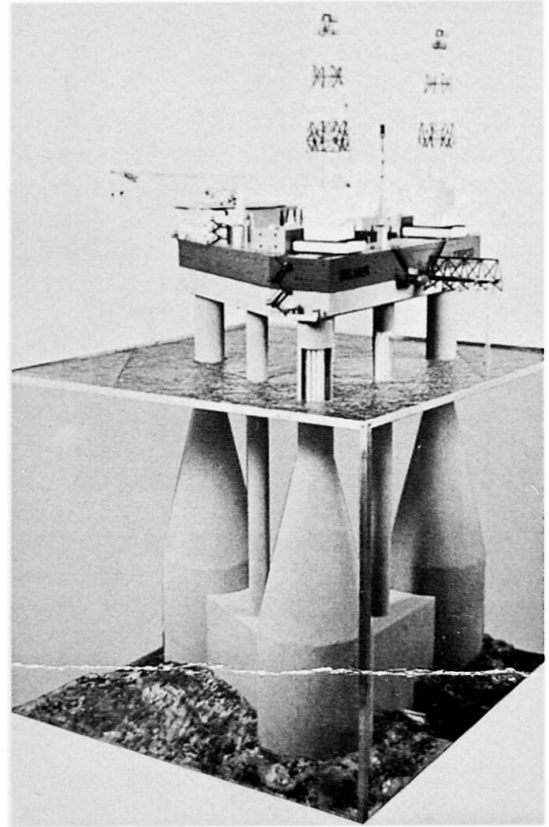


Fig.6: Selmer Tripod, million-barrel oil storage facility being developed in Norway.

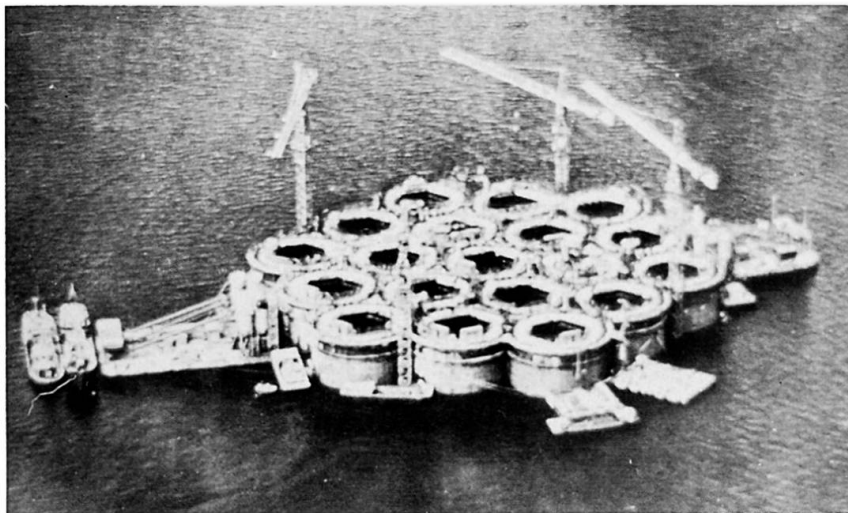


Fig.5: After flotation, construction on the Condeep continues in the Stavangerfjord.

Concrete platforms for North Sea oil production are under construction by McAlpine-Sea Tank in Scotland. Shown in Fig. 7, the structure is intended for installation in water depths ranging from 137 to 163 m. Each structure requires 257,000 tons of concrete and 13,000 tons of reinforcing steel.

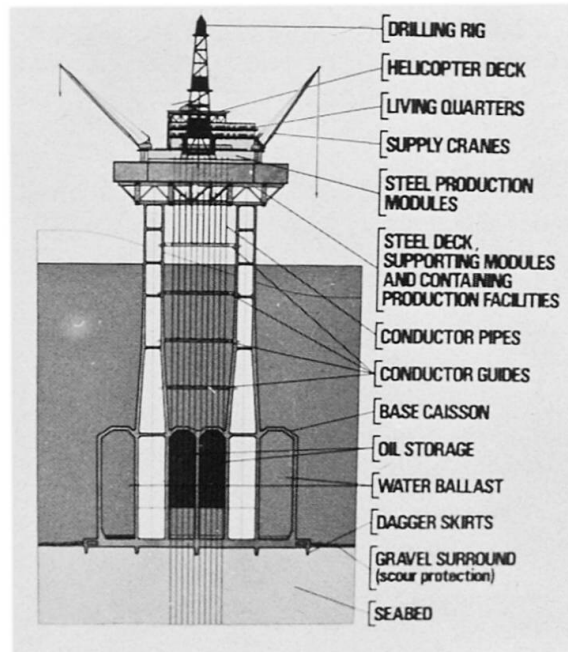


Fig. 7: McAlpine-Sea Tank gravity platform being constructed in Scotland.

Very deep submerged precast concrete structures have been tested by the U.S. Navy.⁴ Figure 8 shows a cylindrical concrete shell structure that was tested at a depth of 180 m below sea level. After eleven months' submergence, the vessel was raised. The interior walls were dry, but approximately 3 liters of water had leaked around a hull fitting.



Fig. 8: SEACON, tested at 180 m below sea level by the U.S. Navy.

3. FLOATING STRUCTURES

The total number of concrete floating structures, including concrete ships that have been launched, probably exceeds one thousand.

In service as floating highways are three precast concrete multi-pontoon structures, two crossing Lake Washington (fresh water), at Seattle, and one crossing Hood Canal (salt water), in Washington State.

All three floating bridges feature precast concrete pontoons, whose dimensions are: length, 110 m; width, 20 m; and depth, 4-1/2m. The shell thickness is 23 cm (Fig. 9).

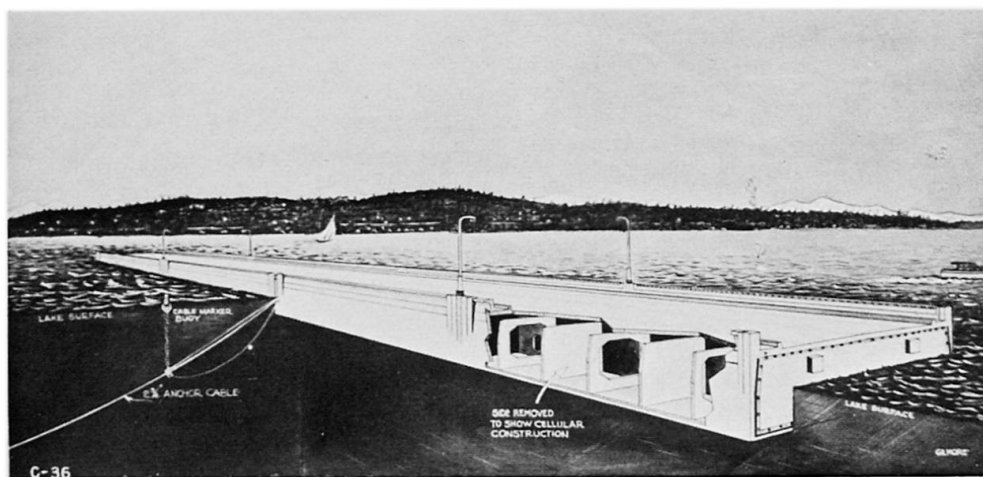


Fig. 9: Cutaway view of floating bridge pontoon.

The first of the floating bridges, built in 1939, was reinforced with standard deformed bars. The second bridge, built in 1955, and the third bridge, built in 1962, were post-tensioned with seven-wire strand tendons.

All precast pontoons were built in a graving dock and launched by flooding the dock (Fig. 10).

After the pontoons were afloat, the elevated superstructure was built and, when completed, the pontoons were towed to the construction site (Fig. 11).

A very large concrete floating harbor for super tanker cargo transfer on the high seas has been proposed by Ulrich Finsterwalder. Shown in Fig. 12, this huge concrete structure would accommodate the ultra large crude carrier having a cargo capacity of 1,000,000 tons.

A. J. Harris has proposed a floating airport, with two parallel runways 4,270 m long, and 4,000,000 square meters of taxiways and parking aprons⁵.

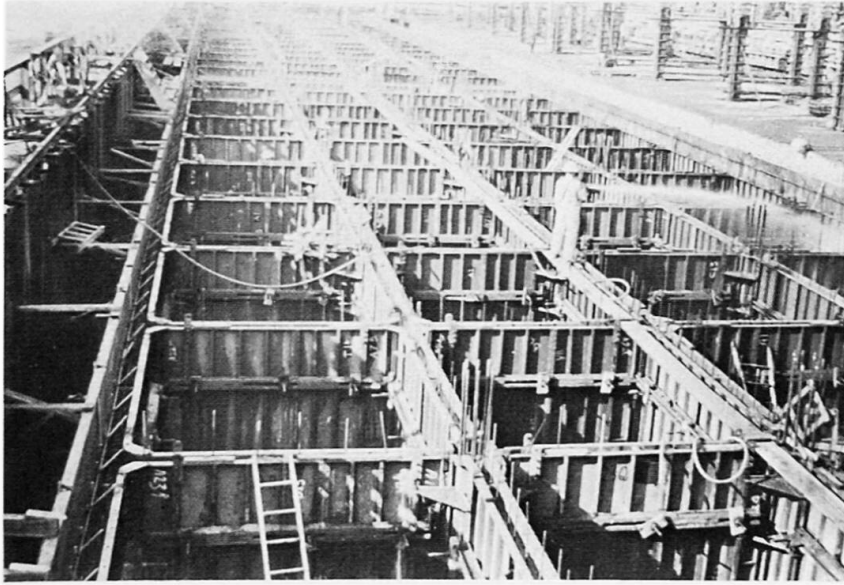


Fig. 10: Precast pontoon construction in graving dock.



Fig. 11: Precast pontoon under tow to elevated super-structure construction site.

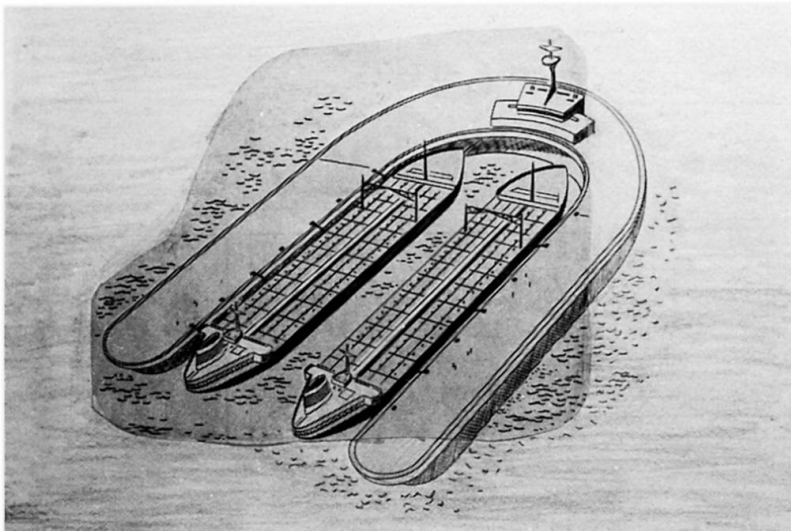


Fig. 12: Concrete floating harbor proposed by Ulrich Finsterwalder.

A very recent application of precast/prestressed concrete is a floating platform for processing and storage of liquid petroleum gas. The vessel, 140 x 41 x 17.4 m, contains 9,000 cubic meters of concrete. The full-load displacement of the vessel is 65,000 tons. (Fig. 13)

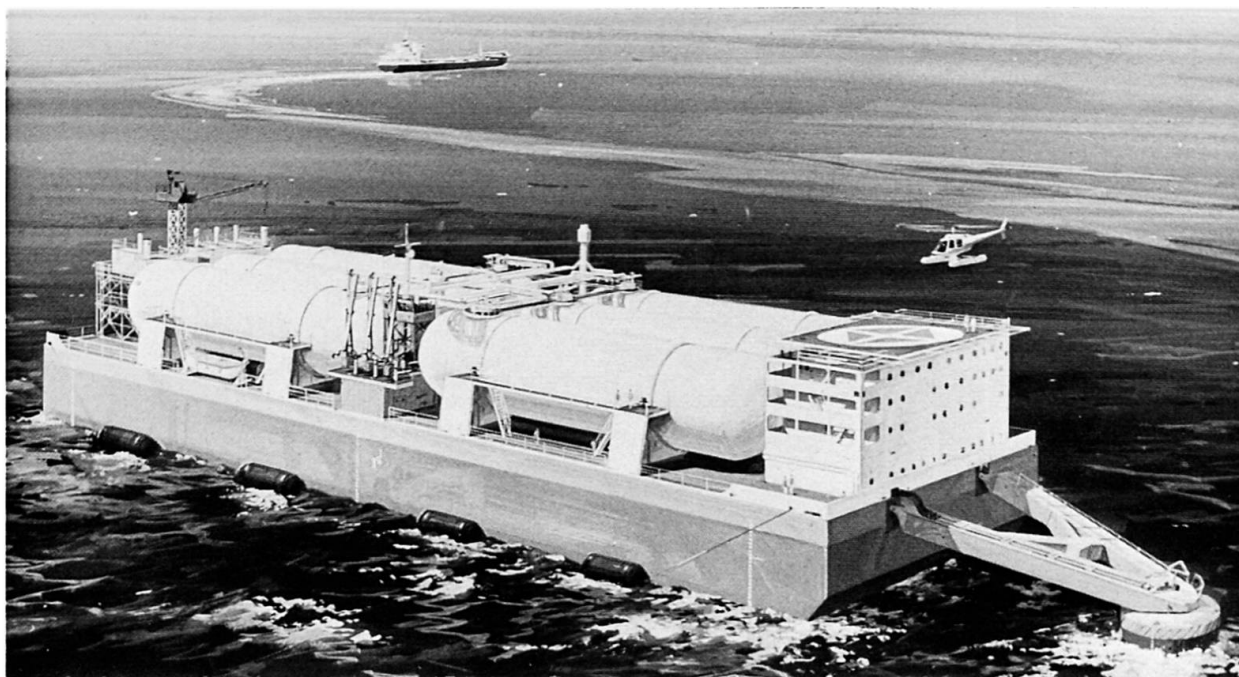


Fig. 13: Liquid petroleum gas production facility, pre-stressed concrete hull. Total displacement 65,000 tons. To be located in the Java Sea.

4. RULES AND RECOMMENDATIONS FROM OTHER GROUPS

Much valuable information on design, construction and inspection of fixed offshore structures has recently been published by Det Norske Veritas*, the Norwegian ship classification Society. Their rules for design and construction have, in the past, primarily related to steel ships and offshore structures.

The 1974 rules include considerable material applicable to reinforced and prestressed concrete offshore structures.

The rules require analyses for two loading conditions:

- a. Functional loads.
- b. Environmental loads and associated functional loads.

Functional loads are those loads incidental to the structures' existence, use and treatment under ideal conditions for each design condition (ideal conditions means no wind, waves, etc.), i.e., no "environmental" loads.

Environmental loads are all directly or indirectly due to environmental actions, such as wind, waves, currents and ice.

For design purposes, the maximum environmental load conditions are based on the most probable severity over a 100-year period.

*Det Norske Veritas, Grenseveien 92, Oslo, Norway.

The required material properties and principles for design and analysis are thoroughly explained. The rules are based on the limit state method.

In addition to the functional and environmental loads given above for all offshore structures, effects unique to prestressed and precast concrete structures must be considered. These include:

- Prestressing
- Creep
- Shrinkage and absorption
- Heat of hydration
- Temperature of stored fluids
- Load concentrations due to uneven sea floor
- Differential settlement

The limit states for design of concrete offshore structures are placed in two categories:

- a. The ultimate limit states (ULS) which are those corresponding to maximum load carrying capacity:
 - Loss of overall equilibrium
 - Rupture of critical sections
 - Instability by deformation
 - Plastic or creep deformation necessitating replacement of the structure, etc.
- b. The serviceability limit states (SLS) which are those related to the criteria governing normal use and durability of the structure:
 - Premature or excessive cracking
 - Unacceptable deformations
 - Corrosion of reinforcement or deterioration of concrete
 - Undesirable vibrations, etc.

Valuable recommendations for constructional arrangements and practices are given, as are provisions for quality assurance.

Recommendations for the design of concrete sea structures published in 1973 by the Fédération Internationale de la Précontrainte contains much valuable basic information on prestressed concrete sea structures, which parallels and augments information from other sources. Included in the FIP Recommendations is a comprehensive bibliography of reference material.

5. LOADS AND FORCES FROM THE SEA

Floating and submerged structures are exposed to systems, loads and forces not fully understood nor quantifiable. More experience and research is needed in order to construct safe, yet economical concrete offshore structures.

Recent work in this field has been presented at the Sixth Off-shore Technology Conference, May 6-8, 1974, Houston, Texas.

Tørum, Larsen and Hafskjold, in Paper No. OTC 1947⁶, discussed the safety of a concrete gravity structure resting on the bottom of the sea. Research on wave forces, hydraulic aspects related to bottom-mounted concrete structures, shock waves, wave damping devices and scour are discussed.

Investigations have been carried out on the effect of wave loads and wave damping under the sponsorship of Ingeniør F. Selmer A/S, Norway, and with financial support by the Royal Norwegian Council for Scientific and Industrial Research.

Tests were conducted at the River and Harbor Laboratory at the Norwegian Institute of Technology, where research work on shock waves and scour was also performed.

Conclusions reached from this work include:

- Sea bottoms on which gravity structures rest are often vulnerable to scour, small structures more so than large ones.
- Designs based on forces from regular waves may give conservative results when compared to irregular waves.
- For evaluation of stresses in the sea bottom soil, one should consider possible dynamic amplification. There is no doubt that, for certain types of soil, such amplification may occur for large structures in deep water.
- When large shafts protrude upward above the waterline, they become susceptible to shock pressures. Model studies indicated the presence of shock pressures during combined wind and irregular wave loading. For a curved surface, the irregular contact face between water, air and structure is difficult to model mathematically. Neither theoretical nor empirical relationships between shock intensities, duration or distribution and wave height, wave steepness or shaft diameters are available.
- The structural response to shock pressures may be separated into global and local effects. The global response to the shock pressure is in most cases negligible, as the resulting load is small compared with the loading from non-breaking waves. For example, a 12 m diameter shaft subjected to 20 t/m² shock pressure over a 10 x 10 m lateral area is approximately 100 tons. For comparison, the lateral load from a non-breaking wave with a height of 25 m and period of 15 sec at 80 m depth is 2,800 tons.

However, the repeated action of the wave shocks sets up small amplitude shock waves propagated down the shaft, and this effect must be included when evaluating the risk of failure at the base of the shafts.

- Wave damping devices have been studied to find methods to damp waves where vessels are moored next to gravity platforms. Much remains to be learned about scour at the base of large concrete gravity structures resting on the ocean floor.
- Scour protection may be provided by a stone blanket which is placed on a suitable filter.

6. CONCLUSIONS

The potential for precast concrete floating and submerged structures almost defies imagination, especially offshore construction for petroleum production facilities. Considerable work has already been accomplished or is in advanced stage of engineering and design.

The advantages for precast and prestressed concrete construction are manifold, and include the following:

- a. Can be produced under factory conditions with labor-saving mechanization.
- b. Precast concrete made with low water-cement ratio and vigorous mechanical compaction can reach very high strength and resistance to sea water attack.
- c. Production proceeds independently of weather conditions at the site, assuring high quality and time saving on construction schedule.
- d. Economy in total capital cost and maintenance cost favors precast construction.

To advance the state of the art, research and development is needed. More understanding of hydrodynamic loading and stability of foundations on the ocean floor is very necessary.

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⁵Seadrome -- a floating airport. A. J. Harris, Proceedings of the FIP Symposium, Concrete Sea Structures, Tbilisi, 1972.

⁶Offshore Concrete Structures - Hydraulic Aspects. Alf Tørum, Per K. Larsen and P. S. Hafskjold, Paper No. OTC 1947, Offshore Technology Conference, 6200 North Central Expressway, Dallas, Texas 75206, May 1974.

SUMMARY

New applications for precast structures are becoming increasingly interesting for offshore construction, both floating and submerged. High quality prestressed concrete is especially attractive for environments where resistance to fatigue as well as corrosion are a major concern.

RESUME

L'emploi de structures préfabriquées devient toujours plus intéressant pour les constructions en mer, qu'elles soient flottantes ou submergées. Le béton précontraint de haute qualité est particulièrement favorable à l'environnement, partout où la résistance à la fatigue et à la corrosion est essentielle.

ZUSAMMENFASSUNG

Die Verwendung vorfabrizierter Tragwerke wird für Bauwerke im Meer immer interessanter, sei es für Unterwasserbauten oder schwimmende Konstruktionen. Der Spannbeton von hoher Qualität ist besonders umweltfreundlich was bezüglich Widerstand bei Ermüdung und Korrosion von grösster Bedeutung ist.