Missouri University of Science and Technology
Scholars' Mine

Center for Cold-Formed Steel Structures Library | Wei-Wen Yu Center for Cold-Formed Steel Structures

01 Jan 1982

Structural Stability Research Council: Proceedings 1982

Structural stability research council

Follow this and additional works at: https://scholarsmine.mst.edu/ccfss-library

Part of the Structural Engineering Commons

Recommended Citation

Structural stability research council, "Structural Stability Research Council: Proceedings 1982" (1982). *Center for Cold-Formed Steel Structures Library*. 235.
https://scholarsmine.mst.edu/ccfss-library/235

This Technical Report is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in Center for Cold-Formed Steel Structures Library by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

About the Council

The Structural Stability Research Council (formerly Column
Research Co[illegible] 1944 to review and resolve
the confli[illegible] [illegible]ted at that
time with [illegible] [illegible]ems, and to
facilitate [illegible].

CCFSS LIBRARY Fritz Engineering Foundation
19 5 * 4917 STRUCTURAL RESEARCH COUNCIL
1982 PROCEEDINGS

The Counci[illegible] [illegible]ers and
practicing [illegible] [illegible]d and refined
calculatio[illegible] [illegible]ity problems,
and assess[illegible] [illegible].

CCFSS LIBRARY Fritz Engineering Foundation
19 5 * 4917 STRUCTURAL RESEARCH COUNCIL
1982 PROCEEDINGS

The membe[illegible] [illegible]esentatives
from orga[illegible] [illegible]) concerned
with spec[illegible] [illegible]tal structures,
represent[illegible] [illegible]ngineering
practice, [illegible] [illegible]ities and de-
sign offi[illegible] [illegible]ious countries
who are i[illegible] [illegible]r region.

The Counc[illegible] [illegible]el for stabi-
lity rese[illegible] [illegible]on research
activitie[illegible] [illegible]uide to Stabi-
lity Desi[illegible]

The Annu[illegible] [illegible]p-to-date in-
formatio[illegible] [illegible]tes where
deficien[illegible] [illegible] of structural
behavio[illegible] [illegible]cord of the
Council'[illegible] [illegible]rce of the
highligh[illegible] [illegible] problems
before [illegible] [illegible]al journals.

STRUCTURAL STABILITY RESEARCH COUNCIL

Established in 1944 by Engineering Foundation

Proceedings 1982

The Council has its Headquarters at:
Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania 18015

Conference supported by grants from
BROWN & ROOT, INC./CHEVRON U.S.A. INC./CHICAGO BRIDGE & IRON COMPANY/EARL AND WRIGHT/MC DERMOTT INCORPORATED/ MOBIL RESEARCH & DEVELOPMENT CORPORATION/ SHELL OIL COMPANY

Table of Contents

Foreword

By the time the 1982 Proceedings are published, the Fourth Edition of the "Guide" should be well underway with the first and second drafts of most chapters in the hands of Ted Galambos, who has taken over the difficult task of Editor for this volume. This is, of course, a major effort of the Council and each of you who have contributed to this new Edition, and who will continue to do so, deserve special thanks from the Executive Committee. As Chairman of the Council for the past four years, it is my privilege to thank each and every one of you for your dedication and hard work.

This coming year will also see SSRC participation in the 3rd International Colloquium on Stability. This Colloquium, being held in the four areas of the World that joined in the preparation of "Stability of Metal Structures: A World View", will follow this report with a report of new developments and a comparison of variations in the basis for design practices in different global areas. The 3rd International Colloquium will be held in Romania, France, Canada and Hong Kong with our Council planning and conducting the Session in North America. Preparation for this Session is being done by our Program Committee under the Chairmanship of Samuel Errera. This group also deserves our special thanks. With 11 Sessions and with invited theme speakers and reporters for each Session, the work effort for planning this Conference merits special commendation.

I also welcome representatives of our new participating firms, organizations and sponsors of our Council. Your participation as well as those who have stood by us in the past, provide the financial support that generates the volunteer involvement of our membership. The Financial Committee is now preparing a brochure that will document your contributions as well as the results of Council work which is SSRC's contribution to the field of Stability. Be sure to review this brochure when it becomes available. It is intended to substantiate your continued support of Council activities.

September 30, 1982 saw the conclusion of my four years as Chairman of SSRC. I can say sincerely that it was a lot of work but that I enjoyed it - especially because of the cooperation and assistance of the entire membership. I thank each of you for this help. Special thanks is due to Lynn Beedle, our Director, Lesleigh Federinic, our Administrative Secretary and Gulay Askar, past Technical Secretary, as well as all the other members of the Executive Committees during my time in office. I leave you now in the very able hands of your new Chairman, John Springfield and new Vice Chairman, Samuel Errera and I pledge to them my continued support and assistance as they initiate a new leadership with new ideas and plans for the Council.

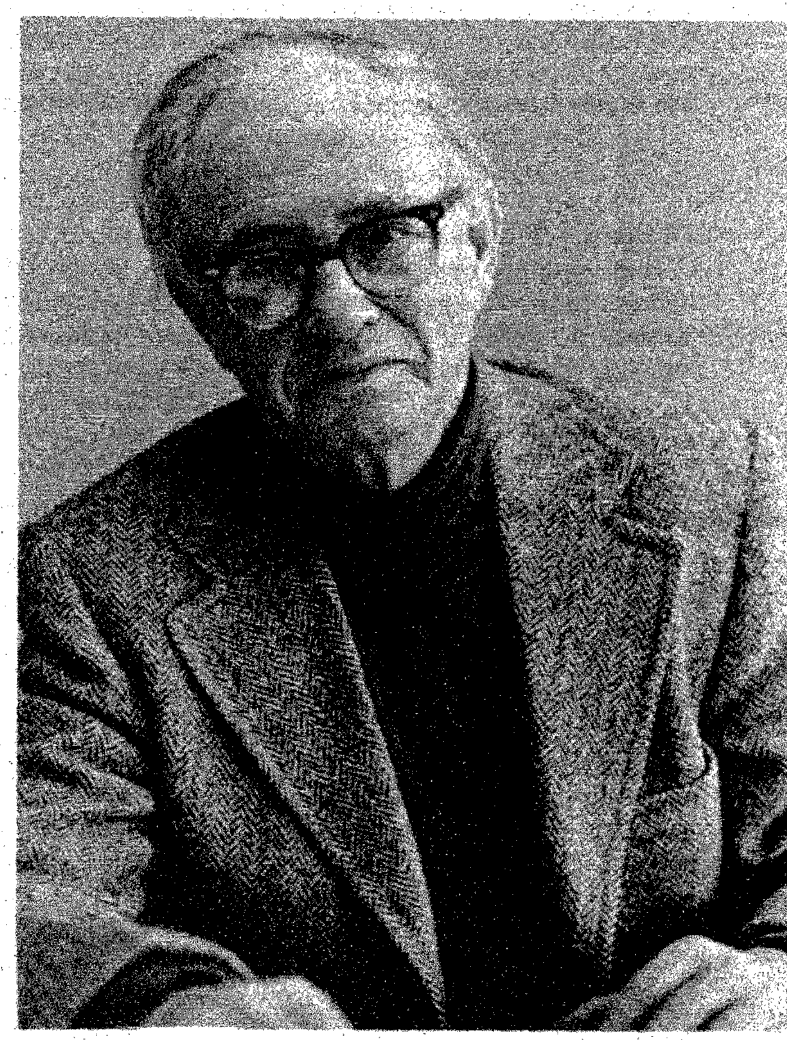
Jerome S.B. Iffland

Jerome S.B. Iffland
Structural Stability Research Council
Chairman, 1979-1982
New York, New York

In Memoriam

PROFESSOR GEORGE WINTER

1907-1982



Professor Winter died suddenly on 3 November 1982, in Ithaca, New York, following a coronary attack. He had enjoyed excellent health all his life, and was carrying forward his usual professional schedule at the time of his death. His numerous engineering friends and colleagues all over the world are left with a profound feeling of personal and professional loss. The news was particularly grievous to George's fellow members of the SSRC Executive Committee, who had enjoyed his company and benefited from his participation at their regular fall meeting only two weeks earlier.

George Winter was born and raised in Vienna. He came to the United States in 1938 to take up a graduate fellowship at Cornell University. He was awarded the Ph.D. degree in structural engineering in 1940 and served on the Cornell faculty over the next 35 years, holding the post of chairman of the department of structural engineering from 1948 to 1970. Cornell appointed him to an endowed chair as the Class of 1912 Professor of Engineering in 1963.

During his undergraduate years in Austria and Germany, George received a liberal education in the classical European tradition. "He spoke French before German, adding fluency in English and Russian before age 30. He learned to play the piano when quite young and remains a music lover. He and his friends went to concerts once or twice a week, having prepared themselves by studying the scores beforehand... It was customary, too, that students attend one of the many excellent theaters every week. They would choose a play to be performed, read it, and try to place it in historical perspective. And Vienna boasted great museums..." *

Following his graduation from the Technical University of Munich in 1930, George took employment in the construction industry. "His first job was working on the construction of Vienna's first skyscraper apartment house. The building still stands, but the company that built it collapsed in the depression. George became one of the large number of foreign specialists who found work in Russia during the first Five Year Plan... There were many professional challenges for him as well as a teaching assignment at the mining institute in Swerdlowsk. His wife, Anne, whom he married in Vienna in 1931, taught at the State Conservatory, and for both of them their Russian sojourn was professionally successful."

The Winter's move to the United States, and George's graduate fellowship and faculty opportunities at Cornell University, were implemented through the aid of several individuals including the late Dean S. C. Hollister; and by the end of 1940 George was launched into what soon became a distinguished academic career. One of the first things that Dr. Winter's students discovered about him was that here was a teacher and researcher who had practiced and understood the art of structural design. While George's lectures and research work were well fortified with theory, he invariably had his eye on an outcome that could be applied on a practical basis. What student could ever forget the typical Winter lecture? "The atmosphere which he consistently creates in the classroom is exhilarating; the clarity, the stimulation, the thought-provoking questions, the personal interaction, the sincerity, the dedication -- all of these things and more make his teaching both a challenge and an excitement."

---

* An excellent review of George's personal and professional life is given as the preface to "The Collected Papers of George Winter," published in 1975 by Cornell University upon his retirement from the faculty. This quotation and the others that follow are taken from that source.

While George's major professional contribution has undoubtedly been as a teacher, he wielded a vast influence on other areas of structural engineering. He became an international leader in structural research in three major materials: structural steel, light-gage steel, and reinforced concrete. He lectured widely on those subjects, and held visiting professorships at other leading institutions from time to time. Further, he was skilled in the art of translating research information into practical specification provisions for design use, and served with distinction on code-writing committees for each of those three materials.

George Winter shared his structural engineering insights and developments with the engineering profession and the public by means of a heavy schedule of writing. He was co-author of a widely used textbook on concrete, contributed to structural engineering handbooks and to the Encyclopedia Brittanica, and authored or co-authored some eighty technical papers -- selections of which are included in the volume of collected papers referred to heretofore. "As a researcher and thesis advisor, his standards are of the highest. His own ability to use language with an uncanny skill leads him to insist on clarity, organization and pertinence. When writing and correcting papers, theses and reports, he wields the pen like a rapier, with awesome effect." What student or researcher ever walked (or sometimes even staggered!) away from a Winter review of his draft material without the feeling that his work had been through the wringer -- and had turned out all the better for it?

But professionalism, for all its importance in Professor Winter's life, is by no means the whole story. "In the case of George Winter the dynamics of art, music, philosophy, politics, and nature pervading Vienna of the 1920s continue to be evidenced in a man who has been not only chairman of Cornell's department of structural engineering for twenty-one years, but also a member of the University Planning Committee, the Engineering College Policy Committee, and the Steering Committee of the Program on Science, Technology and Society; and chairman of the University Lecture Committee, the Faculty Committee on Music, and Friends of Music. Always George urged his students to make use of Cornell's offering of horizon-widening experiences. He led them beyond an appreciation of structural engineering, recommending a special lecture or a good concert to his classes."

During his long and distinguished career Professor Winter accumulated a most impressive list of honors and awards, including an honorary doctorate from his undergraduate university; the Moisseiff Award, the Croes Medal, and the Howard Award of the ASCE; the Wason Medal and the Turner Medal of the ACI; and the International Award of Merit in Structural Engineering of IABSE. He was elected a Fellow of ACI, a member of the National Academy of Engineering, and an Honorary Member of ASCE.

As a particularly fitting memorial, the George Winter Graduate Fellowship in Structural Engineering has been established at Cornell University in his memory.*

* Contributions may be sent to Department of Structural Engineering, Hollister Hall, Cornell University, Ithaca, N.Y. 14853

Finally, we take note of Dr. Winter's significant contributions to the Structural Stability Research Council and its predecessor organization the Column Research Council. He was active in CRC/SSRC affairs over a 34-year period commencing in 1948, serving as a member or as chairman of several committees and task groups. He was a member of the Executive Committee continuously since 1962, and served as SSRC Chairman for the four-year period 1975-78. It was under his instigation and chairmanship that the new, broader name Structural Stability Research Council was adopted in 1976.

As we look ahead to the future of SSRC, it is evident that George Winter's devoted and noteworthy service on the Council and its Executive Committee will be greatly missed by his fellow members. In recognition of his substantial contributions to the work of the Structural Stability Research Council, and of his notable achievements in structural stability theory, research and practice, over an extended period, we have designated the forthcoming North American session of the Third International Colloquium on Structural Stability, to be held in Toronto on 9-11 May 1983, as "The George Winter Memorial Session" of the Colloquium.

SSRC Executive Committee

J. S. B. Iffland, Chairman	- Iffland Kavanagh Waterbury, PC
J. Springfield, Vice Chairman	- Carruthers & Wallace Limited
L. S. Beedle, Director	- Lehigh University
J. L. Durkee	- Consulting Structural Engineer
S. J. Errera	- Bethlehem Steel Corporation
G. F. Fox	- Howard Needles Tammen & Bergendoff
T. V. Galambos	- University of Minnesota
R. R. Graham, Jr.	- Consultant
B. G. Johnston	- Consultant
R. M. Meith	- Standard Oil Company of California
W. A. Milek, Jr.	- American Institute of Steel Construction
C. D. Miller	- Chicago Bridge & Iron Company
D. R. Sherman	- University of Wisconsin-Milwaukee
G. Winter	- Cornell University

Back row: R. M. Meith, W. A. Milek, J. Springfield, J. S. B. Iffland, J. L. Durkee, L. S. Beedle

Front row: L. G. Federinic, D. R. Sherman, G. F. Fox, B. G. Johnston, G. Winter, S. J. Errera

Annual Technical Session

One of the purposes of the Council is to maintain a forum where the structural stability aspects of metal and composite metal and concrete structures and their components can be presented for evaluation, and pertinent structural research problems proposed for investigation. The Annual Technical Session provides an opportunity to carry out this function.

The 1982 Annual Technical Session was held on March 30 and 31 at the Rault Center Hotel, New Orleans. One hundred sixty-four persons attended the sessions and thirty-one papers were delivered.

A panel discussion on "Stability of Offshore Structures" was held in the evening of March 30, 1982. The panelists were P. W. Marshall, C. D. Miller, and J. R. Lloyd. The moderator was R. M. Meith.

This year's panel discussion was preceded by a special afternoon "Offshore Session". C. D. Miller presided over the session which featured papers on research in the United States, United Kingdom, Norway, Japan and Israel.

In conjunction with the Technical Session, an Annual Business Meeting was held for the purpose of electing new officers and members, approval of the budget, and discussion of other business matters.

Papers presented at the offshore session, panel discussion and summaries of the technical session papers are recorded herein. An attendance list and minutes of the Annual Business Meeting are also included.

J. [illegible]ra, Session Program Committee Chairman

P R O G R A M O F T E C H N I C A L S E S S I O N

Tuesday, March 30, 1982

8:00 a.m. - REGISTRATION

9:00 a.m. - MORNING SESSION
Presiding: S. J. Errera, Bethlehem Steel Corporation

INTRODUCTION
J. S. B. Iffland, SSRC Chairman
L. S. Beedle, SSRC Director

9:10 a.m. - TASK GROUP 1 - CENTRALLY LOADED COLUMNS
Chairman: R. Bjorhovde, University of Arizona

. Some Improvements to the Buckling Design of Centrally Loaded Columns
R. Maquoi, Universite de Liege

9:25 a.m. - TASK GROUP 3 - BEAM-COLUMNS
Chairman: J. Springfield, Carruthers and Wallace Limited

. Ultimate Strength of Concentrically and Bi-Eccentrically Loaded Single Angle Columns
Z. Y. Shen, X. R. Hu, L. W. Lu, Lehigh University

9:40 a.m. - TASK GROUP 4 - FRAME STABILITY AND COLUMNS AS FRAME MEMBERS
Chairman: J. S. B. Iffland, Iffland Kavanagh Waterbury, P.C.

. Bracing Stiffness Requirements for Braced Frames
C. Matsui, Lehigh University

. Stability Analysis of Simple Frames with Semi-Rigid Joint Connections
G. J. Simitses and A. S. Vlahinos, Georgia Institute of Technology

. The Effect of Various Directions of 3-D Seismic Inputs on Inelastic Buildings Based on INRESB-3D-82
F. Y. Cheng and A. J. Volker, University of Missouri-Rolla

10:25 a.m. - BREAK

10:45 a.m. - TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION
Chairman: W. W. Yu, University of Missouri-Rolla

. Interactive Buckling in Plate Structures
S. Sridharan, Washington University

. Ultimate Strength of Concentrically Loaded Cold-Formed Angles
M. K. S. Madugula, M. C. Temple and T. S. Prabhu, University of Windsor

P R O G R A M

11:15 a.m. - TASK GROUP 18 - UNSTIFFENED TUBULAR MEMBERS
Chairman: D. R. Sherman, University of Wisconsin-Milwaukee

. Local Buckling of Steel Tubular Columns
D. Grimm, McDermott Incorporated

11:30 a.m. - TASK REPORTER 16 - STIFFENED PLATE STRUCTURES
A. Mansour, University of California, Berkeley

. Structural Reliability and Strength of Stiffened Plates
A. Mansour, University of California, Berkeley

12:00 noon - LUNCH

1:15 p.m. AFTERNOON OFFSHORE SESSION
Presiding: C. D. Miller, Chicago Bridge and Iron Company
Recorders: D. A. Ross, University of Akron
S. X. Gunzelman, Brown & Root, Inc.

1:30 p.m. . Research in United Kingdom in the Stability of Circular Tubes
P. J. Dowling, Imperial College of Science and Technology

2:00 p.m. . Research in Norway in the Stability of Circular Tubes
G. Foss, Det Norske Veritas

2:30 p.m. . Research in U.S.A. in the Stability of Circular Tubes
D. R. Sherman, University of Wisconsin-Milwaukee

3:00 p.m. . Research in Japan in the Stability of Circular Tubes
Y. Kurobane, Kumamoto University; T. Atsuta, Kawasake Heavy Industries Ltd.; S. Toma, Hokkai-Gakuen University

3:30 p.m. . Shell Buckling Research in Israel
J. Singer, Israel Institute of Technology

4:00 p.m. . Stability Under Hydrostatic Pressure and Axial Tension
R. K. Kinra, Shell Oil Company

4:30 p.m. - CHEVRON U.S.A. INC. RECEPTION
R. Regl at piano

PROGRAM

6:00 p.m. -
8:15 p.m. PANEL DISCUSSION: "STABILITY OF OFFSHORE STRUCTURES"

Moderator: R. M. Meith, Chevron U.S.A. Inc.
Recorders: P. C. Birkemoe, University of Toronto
T. A. Bubenik, Exxon Production Research Co.

Panelists: P. W. Marshall, Shell Oil Company
. An Overview of Recent Work on Cyclic Inelastic Behavior and System Reliability

C. D. Miller, Chicago Bridge and Iron Company
. Stability Considerations in the Design of Circular Tubes as Members of Offshore Structures

J. R. Lloyd, Exxon Production Research Company
. Framing Patterns and Their Effect on Jacket Stability

Wednesday, March 31, 1982

8:30 a.m. - MORNING SESSION
Presiding: J. Springfield, Carruthers and Wallace Limited

- TASK REPORTER 13 - LOCAL INELASTIC BUCKLING
L. W. Lu, Lehigh University

. Local Overall Mode Interaction
G. Askar, Lehigh University

8:45 a.m. - TASK REPORTER 14 - FIRE EFFECTS ON STRUCTURAL STABILITY
K. H. Klippstein, U. S. Steel Corporation

. Stability of Fire Exposed Structural Steel Building Floors - A Computer Model and Full-Scale Test
D. C. Jeanes, American Iron and Steel Institute

9:00 a.m. - TASK GROUP 14 - HORIZONTALLY CURVED GIRDERS
Chairman: C. H. Yoo, Auburn University

. Out-of-Plane Buckling of Circular Rings
C. H. Yoo, Auburn University

9:15 a.m. - TASK GROUP 16 - PLATE GIRDERS
Chairman: M. Elgaaly, Bechtel Associates

. Stress Distribution of Buckled Shear Webs
C. Marsh, Concordia University

P R O G R A M

9:30 a.m. - TASK GROUP 21 - BOX GIRDERS
Chairman: R. C. Young, Iffland Kavanagh Waterbury, P.C.

- Steel Box Girders Subjected to Torsion, Bending and Shear
  A. Ostapenko, Lehigh University

9:45 a.m. - BREAK

10:05 a.m. - REPORT ON THE ECCS STABILITY PROGRAM
C. Massonnet and R. Maquoi, University of Liege

10:35 a.m. - TASK GROUP 23 - EFFECT OF END RESTRAINT ON INITIALLY CROOKED COLUMNS
Chairman: W. F. Chen, Purdue University

- Strength of Imperfect H-Columns with Simple End Restraints
  E. M. Lui and W. F. Chen, Purdue University

- Limit States of Flexibly-Connected Steel Building Frames
  M. H. Ackroyd, Rensselaer Polytechnic Institute

- Construction of Variable End Restraints for Small Scale Testing of Tubular Columns
  D. A. Ross, University of Akron

11:20 a.m. - TASK GROUP 7 - TAPERED MEMBERS
Chairman: G. C. Lee, State University of New York

- Full Scale Testing of Tapered Structural Members
  D. L. Johnson, Butler Manufacturing Company Research Center

- Web Crippling of Unstiffened Thin Webs Under Concentrated Load
  G. C. Lee and J. H. Chern, State University of New York

11:50 a.m. - ANNUAL BUSINESS MEETING

12:30 p.m. - ADJOURN

TASK GROUP REPORTS

TASK GROUP 1 - CENTRALLY LOADED COLUMNS

Chairman, R. Bjorhovde, University of Arizona

Some Improvements to the Buckling Design of Centrally Loaded Columns

R. Maquoi, University of Liege

Let me first tell you that this contribution reflects a work to which Assist. Prof. RONDAL has contributed with me, in the department of Professor MASSONNET.

In the foreword of the third edition of what is commonly designated by the "Guide" - I mean the "Guide to Stability Design Criteria for Metal Structures" -, I read that "the name of Column Research Council was changed to Structural Stability Research Council to reflect the broadened scope it has assumed during the more than 30 years of its existence". I observe that this changement of designation coincide nearly with the solution of the most simple stability problem: the buckling of the centrally loaded column. That means that a satisfactory solution of this problem took several centuries. Indeed, the first physical model is attributed to DA VINCI and the first empirical formula suggested by VAN MUSSCHENBROEK is 250 years old. EULER formulated his well-known formula for the critical load in 1759 and NAVIER showed in 1826 that the Eulerian load is an upper bound. Then, two approaches are developed: the first one is concerned with the bar with imperfections and the second one is dealing with the inelastic behaviour.

In 1955, ECCS decided to perform a large series of tests and to simulate numerically the behaviour of the centrally loaded columns; as a result, three curves were proposed in 1970 and lightly modified afterwards so that 5 curves exist now in non dimensional coordinates, with a yield plateau up to a reduced slenderness ratio of 0.2.

The selection chart for these curves takes account of the type of cross-section, of the buckling axis and eventually the depth to bredth ratio. The ECCS curves correspond to characteristic values of residual stress and of initial imperfection, this latter being 1/1000 of the buckling length.

The three SSRC buckling curves called 1, 2 and 3 are very close to European curves a_o, b and d respectively. European curves were only given by means of numerical tables and were thus not convenient for a use on a small desk computer. On the contrary, the SSRC buckling curves are given analytically but require 11 coefficients for curves 1 and 3 and 12 coefficients for curve 2.

A first attempt was to find simple analytical expressions for the ECCS buckling curves and, why not, much more simple equations for the SSRC ones. For this purpose, a similar approach was used; it is based on a physical model: an imperfect bar centrally loaded is subject to an axial force N and to an amplified bending moment $M^{II} = K\,N\,e_o$. It is well known that the magnification factor K is different in case of an initial deflection of the imperfect bar or to a load exxentricity in a perfect bar; however the numerical values are only slightly different and the magnification factor can be taken equal to $(1 - N/N_{cr})$. Using a collapse criterion according to which the ultimate strength is reached when the yield stress f_y occurs at the most compressed fiber, it is written:

$$\frac{N_K}{A} + \frac{M^{II}}{W} = f_y \tag{1}$$

or, in terms of stresses:

$$\sigma_K + \frac{\sigma_K}{1 - \frac{\sigma_K}{\sigma_{cr}}} \; \frac{e_o\,A}{W} = f_y \tag{2}$$

After some minor arrangements, one has the well-known AYRTON-PERRY like equation:

$$(\sigma_{cr} - \sigma_K)\;(f_y - \sigma_K) = \eta\;\sigma_{cr}\;\sigma_K \tag{3}$$

with the imperfection parameter:

$$\eta = \frac{e_o\,A}{W} = \frac{e_o\,A\,v}{I} \tag{4}$$

or by expressing e_o as a part $1/\gamma$ of the length L:

$$\eta = \frac{L}{\gamma i(i/v)} = \frac{\lambda}{\gamma\;(i/v)} \tag{5}$$

where λ is the conventional slenderness ratio L/i and (i/v) is the relative diameter of the ellipse of inertia for the buckling axis under consideration. That would show that for a specified slenderness and a given value of γ, the imperfection parameter η should be different for each type of section because of (i/v) is a characteristic of a class of sections but that it must be expected a more severe role of imperfection for buckling about the weak axis than about the strong axis. This conclusion is indeed reflected by the selection chart for the buckling curves.

With the non dimensional coordinates used by ECCS and SSRC, the AYRTON-PERRY equation is written:

$$(1 - \bar{N})\ (1 - \bar{N}\,\bar{\lambda}^{\,2}) = \eta\,\bar{N} \tag{6}$$

with:

$$\bar{N} = \frac{\sigma_K}{f_y} = \frac{N_K}{N_Y} \tag{7}$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_Y} = \frac{\lambda}{\pi}\sqrt{\frac{E}{f_y}} \tag{8}$$

It is obvious that such expression lies between the yield plateau $\bar{N} = 1$ and the Eulerian curve $\bar{N} = 1/\bar{\lambda}^{\,2}$. Such an equation will reflect an European curve if $\bar{N} = 1$ for $\bar{\lambda} = \bar{\lambda}_o = 0.2$ and if η takes account of both geometrical and structural imperfections.

This second degree equation has following solution:

$$\bar{N} = \frac{1 + \eta + \bar{\lambda}^2}{2\,\bar{\lambda}^2} - \frac{1}{2\,\bar{\lambda}^2}\sqrt{(1 + \eta + \bar{\lambda}^2)^2 - 4\,\bar{\lambda}^2} \tag{9}$$

which is nothing else than the analytical expression of a buckling curve. RONDAL and myself adopted the most simple expression for η, which gives a sufficient accuracy.

Last, for steel, following value was chosen for ECCS curves:

$$\eta = \alpha\,(\bar{\lambda} - 0.2) \tag{10}$$

and the values of α was calibrated in order to adjust as well as possible on the European curves:

curve a_o : $\alpha = 0.125$
" a : $\alpha = 0.206$
" b : $\alpha = 0.339$
" c : $\alpha = 0.489$
" d : $\alpha = 0.756$

For the SSRC curves, the solution is quite similar:

$$\bar{N} = \frac{1 + \eta + \bar{\lambda}^2}{2\,\bar{\lambda}^2} - \frac{1}{2\,\bar{\lambda}^2}\sqrt{(1 + \eta + \bar{\lambda}^2)^2 - 4\,\bar{\lambda}^2} \qquad (11)$$

but with an imperfection parameter η given by:

$$\eta = \alpha\,(\bar{\lambda} - 0.15) \qquad (12)$$

and α given in accordance with the buckling curve:

curve 1 : 0.103
" 2 : 0.293
" 3 : 0.622

Thus, a first improvement was an analytical formulation of the ECCS buckling curves and, in my opinion, a much more simple expression for the SSRC ones. For these latter, a curve is characterized by a simple parameter instead of 11 or 12 and the formulation is continuous in the whole range of slenderness ratio $\lambda > 0.15$.

In order to take account that the probability is small to reach simultaneously in a specified section, the characteristic values of all the parameters, including dimensions and thicknesses, ECCS Recommendations allow for fictitious values of the yield stresses, generally higher than the actual value of the material yield stress. This trick is not at all satisfactory for because of consistency, a single value of the yield stress, that is the actual one, ought to be used, and many codes have amended the ECCS Recommendations accordingly. Doing so, some loss of ultimate strength results unfortunately; recently we suggested to modify slightly the widespread reproduced ECCS buckling curves by correcting the coefficients as follows:

$a_o = 0.093$
$a = 0.17$
$b = 0.29$
$c = 0.43$
$d = 0.68$

TG-1

The ECCS Recommendations recognize that the detrimental effect of the residual stresses in rolled profiles decreases when the material yield stress increases and therefore, a jump to the immediately upper buckling curve is allowed when f_y reaches 430 MPa. This procedure is highly criticable because of its discontinuity and there results no benefit for producing materials the yield stress of which lies in the range 235-430 MPa. To compensate this penalty, that is unjustified as demonstrated by tests, we suggested that it will refer in future to the set of 4 buckling curves a, b, c, d with an imperfection parameter η, taking account of the steel grade as follows:

$$\eta = \alpha\ (\bar{\lambda} - 0,2)\ \left(\frac{235}{f_y}\right)^{0.8} \tag{13}$$

The exponent 0.8 of the ratio $(235/F_y)$ was calculated in such a way that the curves adjust themselves on a_o, a, b, c when f_y = 430 MPa.

In addition to these improvements concerning the formulation of the buckling curves properly, the physical model used hereabove allows to define the equivalent imperfection by which a column of a frame must be affected in order to analyse its stability.

For this purpose, the expression of η:

$$\eta = \frac{e_o\ A}{W} = \frac{\lambda}{\gamma(i/v)} \tag{14}$$

is written with non-dimensional values:

$$\eta = \frac{(\bar{\lambda} - 0,2)\ \pi\ \sqrt{E/f_y}}{\gamma(i/v)} \tag{15}$$

and as η has also been written in the form:

$$\eta = \alpha(\bar{\lambda} - 0,2)\ \left(\frac{235}{f_y}\right)^{0.8} \tag{16}$$

it results from equating (15) and (16), that:

$$\gamma = \frac{L}{e_o} = \frac{\pi\sqrt{\frac{E}{f_y}}\ \left(\frac{f_y}{235}\right)^{0.8}}{\alpha(i/v)} \tag{17}$$

is the measure of the relative imperfection e_o/L expressed versus the yield stress, the type of cross-section and the buckling curve which is to be applied. The use of this formula leads to numerical results which are close to values obtained and suggested by more sophisticated procedures.

Another question is: what should be the detrimental effect of a geometrical imperfection which could accidentally be higher than 1/1000 of the buckling length L whilst the residual stresses are standard like. Therefore let us assume that in factor η, the coefficient α, which characterizes every buckling curve, can be divided into a sum of two parts, which represent respectively the effects of structural imperfections and of geometrical ones, thus:

$$\alpha = \alpha_1 + \alpha_2 \tag{18}$$

and α_2 can be drawn from (17), so that for the bar with a larger imperfection measured by γ^*, the buckling parameter α^* is:

$$\alpha^* = \alpha + \frac{\pi\sqrt{\frac{E}{f_y}}\left(\frac{f_y}{235}\right)^{0.8}}{(i/v)}\left(\frac{1}{\gamma^*} - 10^{-3}\right) \tag{19}$$

The validity of this approach is demonstrated by the agreement with the results of a numerical simulation performed by DJALALY and SCHULZ.

The reverse question is: what is the detrimental effect of residual stresses higher than standard values used for establishing the buckling curves, in case of a geometrical imperfection equal to 1/1000 of the buckling length? Such a question is namely of interest for cold formed tubes for which the manufacturing procedure has a governing influence on the distribution and the amount of residual stresses. I had hoped to benefit from this session to present here the first results of our analysis; unfortunately, though having all the numerical results, investigations required some more time than expected and I am not yet able to give you some definitive conclusions.

Last, and though what will follow regards rather another task group, let me tell you that the physical model discussed above can be used for the analysis of a beam-column. Then, for uniaxial bending, it can be written:

$$\frac{N}{A} + \frac{N\, e_o \frac{1}{W}}{1 - \frac{\sigma_K}{\sigma_{cr}}} + \frac{N\, e \frac{1}{W}}{\cos \frac{\pi}{2}\sqrt{\frac{\sigma_K}{\sigma_{cr}}}} \leqslant f_y \tag{20}$$

where the denominators represent respectively the amplification factor for the bending moment in case of an initial imperfection e_o of the bar and of an eccentricity of the loading respectively. Both values are about the same, as already said, and thus:

$$\sigma_K + \frac{\sigma_K}{1 - \frac{\sigma_K}{cr}} \left(\eta + e \frac{A}{W}\right) = f_y \qquad (21)$$

or with non-dimensional coordinates and after some minor arrangements:

$$(1 - \overline{N})\ (1 - \overline{N}\ \overline{\lambda}^2) = \eta^* \overline{N} \qquad (22)$$

with:

$$\eta^* = \eta + e \frac{A}{W} \qquad (23)$$

This equation is quite similar to that obtained for the buckling of centrally loaded columns; the difference lies in the fact that η^* is substituted to η. The presentation of (21) is in accordance with the ECCS Recommendations; it differs of the U. S. check of a beam-column, to which the Belgian code is identical. To my opinion, the ECCS formulation shows clearly in that way that a centrally loaded bar is a peculiar case of a beam-column.

REFERENCES

1. E.C.C.S. Recommendations for Steel Construction. E.C.C.S. - EG 77-1E.

2. Manual on the Stability of Steel Structures. Second International Colloquium on Stability of Steel Structures. Introductory Report, Liege, 1977.

3. R. MAQUOI and J. RONDAL: Formulations d'AYRTON-PERRY pour le flambement des barres metalliques. Construction Metallique, N° 4, 1979.

4. J. RONDAL and R. MAQUOI: Le flambement des colonnes en acier. Chambre Syndicale des Fabricants de Tubes d'Acier, Paris, Notice 1091, Nov. 1980.

5. J. RONDAL and R. MAQUOI: Single Equations for SSRC Column-Strength Curves. Technical Note. Proc. A.S.C.E., Jl. Struct. Div., ST1, Jan. 1979, pp. 247 - 250.

TASK GROUP 3 - BEAM-COLUMNS

Chairman, J. Springfield, Carruthers and Wallace Limited

Ultimate Strength of Concentrically and Bi-Eccentrically Loaded Single Angle Columns

Zu-Yan Shen, Xue-Ren Hu, and Le-Wu Lu, Lehigh University

The use of single angle members in trusses, joists, and other types of structures is quite common. Depending on loading condition, single angle members in compression can be divided into two categories. One is concentrically loaded angles and the other is eccentrically loaded angles with connection in one leg only. In this paper, a finite element method, which takes into account the effects of residual stresses, initial crookedness and end restraints provided by gusset plates is developed to obtain ultimate strength solutions for compression members of both categories.

The strength of simply supported and warping restrained angles with equal and unequal legs and subjected to concentrical load is first presented in the form of column curves. The results show that the strength of unequal leg angles (on a non-dimensional basis) is lower than that of equal leg angles.

For the case of eccentrical loading, ultimate strength curves are obtained for the 2" x 2" x 1/4" angle with three types of end condition, as illustrated in Figure 1. Curve 1 is for the case where the gusset plate is fixed against rotation about both the x and y axes. Curve 2 assumes that the gusset plate is hinged about the x axis and fixed about the y axis. These conditions are interchanges for Curve 3. Figure 1 also shows comparisons of the theoretical curves with the results of tests conducted at Washington University.* In these tests, the angles were welded to the stem of a T stub. The flange of the T stub was perpendicular to the longitudinal axis of the angle. Three types of end conditions were adopted. In the first type the T stub bore directly against the platform (or the movable head) of the testing machine. A knife edge was provided in the x direction in the second type and in the y direction in the third type. The theoretical solution of Curve 2 shows very good agreement with the test results obtained with the second type of support agreement. In this case the flange of the T stub provided a substantial restraining effect against rotation about the y axis. For the other two cases, because full fixity of the ends could not be achieved in the support arrangements, the test results fall considerably below the theoretical curves. The actual restraints provided by the test setups are not known and can not be rationally estimated.

* "Eccentrically Loaded Single Angle Columns" by N. S. Trahair, T. Usami, and T. V. Galambos, Washington University Research Report NO. 11, August, 1969.

Numerical calculations have also been carried out for the 3" x 2" x 1/4" angle and the results are compared with the available Washington University test results in Table 1. Again, the theoretical solutions for end condition 2 agree very well with the experimental results.

Finally, using the concept of effective length as adopted in some national codes, a family of column curves for eccentrically loaded equal and unequal leg angles with connection in one leg only are given. For convenience of design use, the effective length factor is taken as the parameter for these curves.

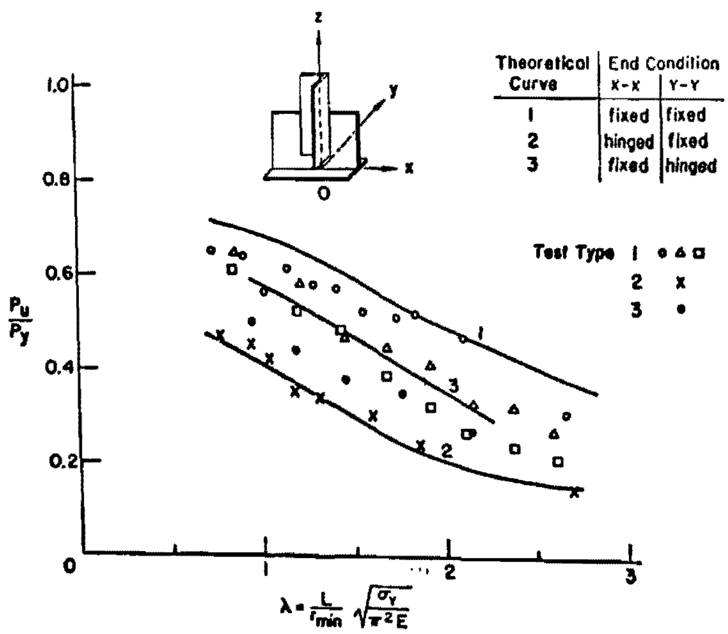


FIG. 1 - THEORETICAL AND EXPERIMENTAL RESULTS OF L3" x 2" x 1/4" OF MODIFIED A242 STEEL

Table 1. Comparison of Theoretical and Experimental Results of L3" x 2" x 1/4" of Modified A441 Steel

Specimen	L/r_{min}	End Condition and Attitude		P_u/P_y Test	P_u/P_y Theoretical
1	130.0	1*	a**	0.46	0.50
2	130.0	1	b	0.36	0.43
3	87.8	1	a	0.53	0.59
4	87.5	1	b	0.51	0.56
5	133.0	2	a	0.36	0.35
6	133.0	2	b	0.18	0.16
7	89.5	2	a	0.45	0.44
8	90.2	2	b	0.27	0.27
9	133.0	3	a	0.21	0.23
10	133.0	3	b	0.34	0.36
11	83.5	3	a	0.34	0.34
12	83.5	3	b	0.43	0.45

* end condition as specified in Fig. 1.

**end attitude "a" long leg outstanding
"b" short leg outstanding

TASK GROUP 4 - FRAME STABILITY AND COLUMNS AS FRAME MEMBERS

Chairman, J. S. B. Iffland, Iffland Kavanagh Waterbury, P.C.

Bracing Stiffness Requirements for Braced Frames

C. Matsui, Lehigh University

In frames where lateral stability is provided by diagonal bracing, the effective length factor K for columns is usually taken as unity. The design of frames based on this assumption seems to be very simple; however, a question remains concerning the magnitude of the required stiffness or cross sectional area of the bracing in order to satisfy the assumption K = 1.

In this study, in order to calculate the stiffness and cross sectional area, design formulas have been obtained by performing stability analyses for uniform pinned-base rigid frames.

The relationship between the horizonal axial stiffness of the bracing and the effective length factor for columns in a frame which has no initial imperfection shown in Fig. 1 can be obtained using the stability condition for the whole frame. More complex frames are also solved by the same method. In this analysis, general stability functions are used and only the tension braces are assumed to be effective for lateral stability.

Figure 2 shows the results of the analysis of the relationships between the required horizontal bracing stiffness for one column C', expressed in dimensionless form, and the beam-to-column stiffness ratio $\frac{E_g I_g}{E_c I_c} \cdot \frac{L_c}{L_g}$ in several frames. E_c, I_c and L_c represent elastic modulus, moment of inertia and length of the column, respectively. Similarly, E_g, I_g and L_g are for the girders. In the ordinate, $\tau E_c I_c$ represents the flexural stiffness of a column based on the tangent modulus theory. The solid lines in Fig. 2 are for the case of K = 1. The dotted lines are for the case in which the buckling load of a braced frame is equal to the buckling load of an unbraced frame. The buckling load of an unbraced frame corresponds to the non-sway mode.

The total horizontal bracing stiffness C is expressed in the following simple form for uniform multi-story, multi-bay, pinned-base perfect rigid frames shown in Fig. 3. C should be placed in each story in order to satisfy the condition K = 1.

$$C = (n + 1)\pi^2 \tau E_c I_c / L_c^3 \qquad (1)$$

where n is the number of bays. Equation (1) shows that C is not affected by the number of stories m and flexural stiffness of a girder $E_g I_g$.

The effect of the initial sway displacement in a frame on the required bracing stiffness has been investigated using a model shown in Fig. 4. The relationship between the required bracing stiffness C and the initial sway displacement δ_o has been obtained using stability functions. C is calculated for the condition at which the brace starts to yield while the frame is subjected to the buckling loads which are calculated for the perfect frame under the condition K = 1. According to calculated results, the maximum required bracing stiffness in an initially imperfect frame is approximately twice that of a perfect frame when the following values are assumed: the yield stress of bracing σ_{Yb} is from 2.4 t/cm^2 to 4.1 t/cm^2, δ_o/L_c is 1/500, and slope of a brace is from 30^o to 60^o.

Considering the effect of an initial imperfection in a frame, the design formulas for the pinned-base rigid frames may be finally written in the following forms:

$\lambda \leq \pi\sqrt{E_c/\sigma_{Yc}}$

$$C = 2(n+1)P_Y/L_c \tag{2.a}$$

$$A = 2(n+1)\left(\frac{\sigma_{Yc}}{E_b}\right)\frac{A_c}{\cos^2\theta\cdot\sin\theta} \tag{2.b}$$

$\lambda > \pi\sqrt{E_c/\sigma_{Yc}}$

$$C = 2(n+1)P_{cr}/L_c \tag{2.c}$$

$$A = 2(n+1)\left(\frac{\pi}{\lambda}\right)^2\left(\frac{E_c}{E_b}\right)\frac{A_c}{\cos^2\theta\cdot\sin\theta} \tag{2.d}$$

where $P_Y = A_c\sigma_{Yc}$, $P_{cr} = \pi^2E_cI_c/L_c^2$, $\lambda = L_c/r$

A_c, I_c, σ_{Yc}, r = cross sectional area, moment of inertia, yield stress and radius gyration of a column.

E_c, E_b = modulus of elasticity of columns and braces

The formulas (2.a) (2.b) are based on the assumption that when the slenderness ratio of a column, λ, is equal to or less than $\pi\sqrt{E_c/\sigma_{Yc}}$, the column can carry the axial yield load P_Y. The total required cross sectional area of bracing A may be calculated as shown. A was obtained from the condition that the required bracing stiffness for lateral stability is equal to the bracing stiffness obtained under the compatibility condition for a frame with sway displacement. These formulas may also be used conservatively in the design of fixed-base rigid frames.

Some design recommendations are available for bracing stiffness of a rigid frame in order to judge the resistance of the frame to side sway instability. Figure 5 shows the comparison of the design recommendations by ECCS [1] and ACI [2] with Eq. (2.c). My calculations show that ECCS and ACI recommendations underestimate the requirement of the stiffness of braces for the case of pinned-base rigid frames.

REFERENCES

1. European Convention for Constructional Steelwork, Introductory Report of Second International Colloquium on Stability, 1976, p. 236.

2. American Concrete Institute, Building Code Commentary, 1977, p. 48.

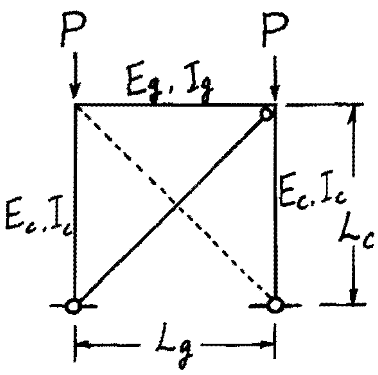


FIG. 1

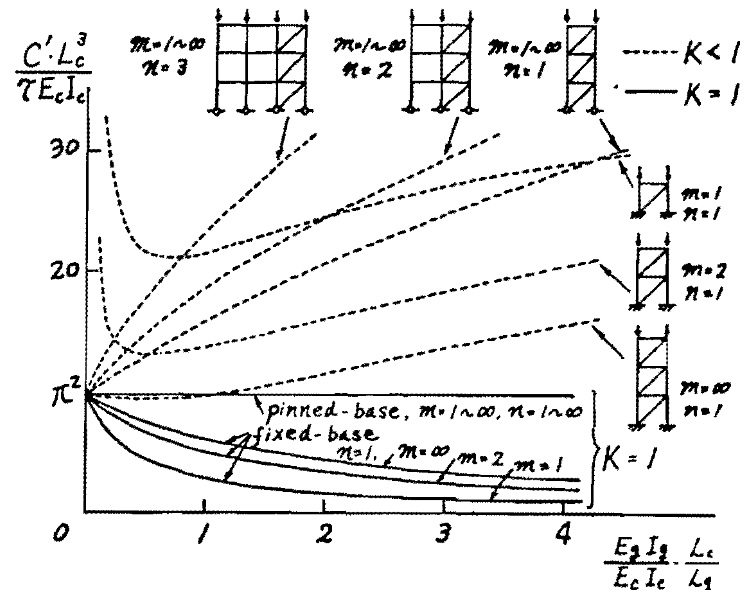


FIG. 2

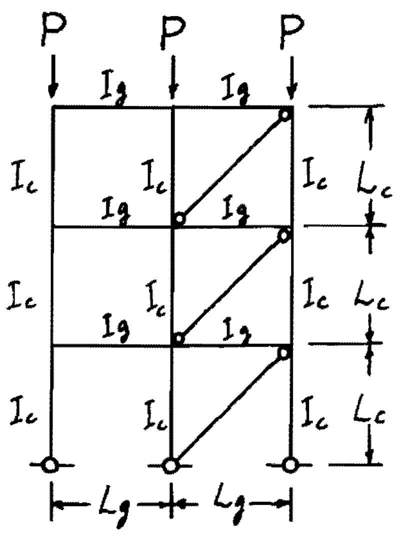


FIG. 3

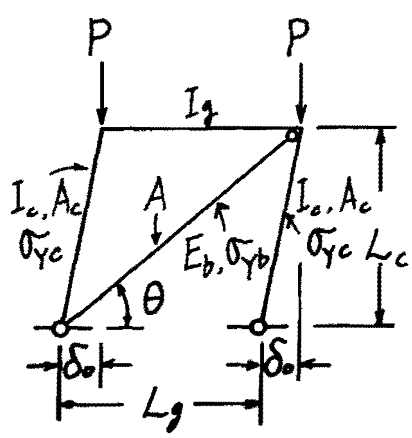


FIG. 4

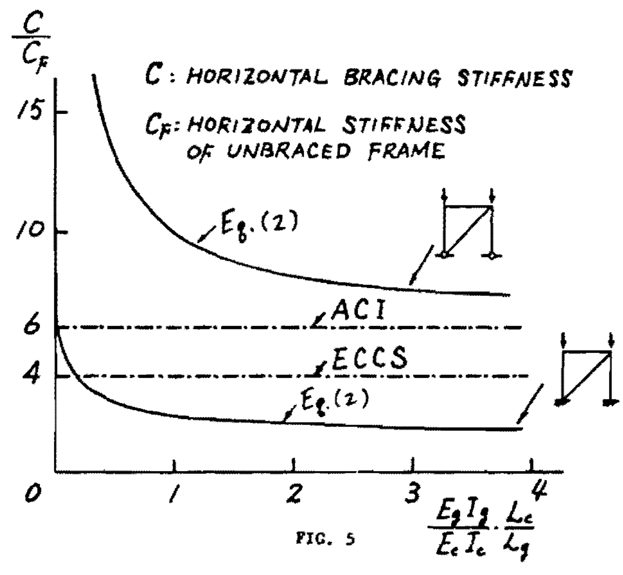


FIG. 5

TG-4

Stability Analysis of Simple Frames with Semi-Rigid Joint Connections

G. J. Simitses and A. S. Vlahinos, Georgia Institute of Technology

Introduction

Frames are widely used in buildings and other structural configurations. Frame analysis has been simplified, by most investigators, by assuming either a pin-jointed corner connection or a rigid one. Some efforts have been made recently to account for the effect of flexible connections in frame design. Defalco and Marino[1] propose the use of an effective column length (for frame design purposes) based on a modified beam stiffness, which is a function of the semi-rigid connection factor, Z, proposed by Lothers[2]. This factor is a measure of the initial slope, 1/Z (and therefore a constant) of the joint bending moment versus the relative rotation curve for the flexible connection. This curve is in general nonlinear. Moreover, the factor Z has the units of radian per unit of bending moment. Note also that, the slope (1/Z) decreases as the bending moment increases. The decreases in slope should not be very large, if the joint is to perform well under loading. Finally, note that the units of the slope are those of the stiffness of a rotational spring.

In the present investigation, the response characteristics (including buckling loads) of simple frames with flexible connections are analyzed. The studies include: (a) the analysis of frames with linear rotational springs at the joints, and spring stiffnesses varying from zero (pin joint) to infinity (rigid joint); and (b) the analysis of frames with realistic flexible joints. Such joints (see Ref. 1) are modeled with nonlinear rotational springs at the connections. Results are presented for simple two-bar frames and portal frames. For all geometries considered, the bars are of equal length and stiffness.

Discussion of Results

The solution procedure is a modification of the one appearing in Refs. 3-6. The modification accounts for the flexible joints.

Results are obtained in the form of complete response for each load level, and they include establishment of critical conditions. In addition, several parametric studies are performed in order to establish the effects of such parameters as bar slenderness ratio, load eccentricities and others.

For the sake of brevity, no detailed discussion of the results is presented. Instead, the most important conclusions of the investigation are presented below:

(1) For both cases of flexible connections and all geometries considered, the effect of slenderness ratio on the nondimensionalized response characteristics is negligibly small.

(2) The effect of load eccentricity on the nondimensionalized critical loads (limit point loads for the two-bar frames, bifurcation loads [sway buckling] for the portal frames) is the same as in the case of rigid-jointed frames. For instance, the sway-buckling load decreases (slightly) with increasing load eccentricity. The limit point load also decreases with increasing eccentricity (in a given direction; for large eccentricities inside the frame - load to the right of joint - there is no buckling).

(3) The effect of varying the constant rotational spring stiffness is as expected. The larger the stiffness the stronger the configuration for both geometries. Note that as the stiffness changes from zero to infinity (a large number) the connection condition changes from a pin joint to a rigid joint.

(4) For the case of realistic flexible connections (type 2 of Ref. 1), the effect of variable spring stiffness on the nondimensionalized response characteristics is negligibly small, provided that the moment-relative rotation curve possesses a positive slope at the instance of buckling.

References

1. DeFalco, F., and Marino, F. J., "Column Stability in Type 2, Construction", AISC Engineering Journal, Vol. 3, No. 2, April 1966.

2. Lothers, J. E., Advanced Design in Structural Steel, Prentice-Hall, Inc. Englewood Cliffs, N. J., 1960 (Ch. 8).

3. Kounadis, A. N., Giri, J., Simitses, G. J., "Nonlinear Stability Analysis of an Eccentrically Loaded Two-Bar Frame", J. Appl. Mech., Vol. 44, Series E, No. 4, 1977, pp. 701-706.

4. Simitses, G. J., and Kounadis, A. N., "Buckling of Imperfect Rigid-Jointed Frames", J. Eng. Mech. Division, ASCE, EM3, Vol. 104, 1978, pp. 569-586.

5. Simitses, G. J., Kounadis, A. N., and Giri, J., "Nonlinear Buckling Analysis of Imperfection Sensitive Simple Frames" in Stability of Structures under Static and Dynamic Loads, Proceedings of Int'l Colloquium, Washington, D. C., May 1977, pp. 158-179 (published by ASCE).

6. Simitses, G. J., Giri, J., and Kounadis, A. N., "Nonlinear Analysis of Portal Frames", Int'l J. for Numerical Methods in Engineering, Vol. 17 1981, pp. 123-132.

TG-4

The Effect of Varying Directions of Seismic Inputs on Inelastic Steel Buildings Based on INRESB-3D-82

F. Y. Cheng and J. A. Volker, University of Missouri-Rolla

An analytical study is presented for the effect of the seismic input directions on the elastic and inelastic structural response. The response behavior was observed for the steel building shown in Fig. 1 subjected to three-dimensional seismic excitations. Four groups of input directions of El Centro, 1940: -18.02°, 0°, 71.98°, and 90° are used which are measured (positive clockwise) from the reference x-axis to the N-S component of the seismic data. The structure was analyzed for three loading cases: a) N-S component only (for elastic analysis), b) N-S, E-W, and vertical components (for elastic analysis), and c) N-S, E-W, and vertical (for inelastic analysis). These three cases were all studied for the four groups of input directions.

Results for each case include a) lateral and rotational displacements at each floor, b) axial displacements of columns, c) input, dissipated, and kinetic energies of the entire structure, and d) ductility factors and excursion ratios of beams and columns. Only the displacements are given here. The absolute maximum values of the displacements along the principal axes at levels 2, 4, and 6 are summarized in Table 1. It can be seen that when N-S component is applied (case 1) along the minor axis, I_{min} (-18.02°), the displacement along I_{min} is maximum and the displacement along I_{max} is minimum in comparison with the responses associated with the other three groups of seismic inputs. However, when three dimensional ground motions are applied, the displacements along I_{min} direction are not consistently to be maximum for the seismic input of -18.02°. This is due to interaction of the ground motion on the response. The inelastic displacements (loading case 3) along I_{min} and the vertical displacements of columns are shown in Figs. 2 and 3 from which one may observe that the seismic input direction can significantly influence the response behavior.

TABLE 1. MAX. DISPLACEMENTS ALONG THE PRINCIPAL AXES

Level	Angle	Max. Displacement in I_{min} Direction (in.)			Max. Displacement in I_{max} Direction (in.)		
		Cases			Cases		
		1	2	3	1	2	3
2	-18.02	3.1	3.2	4.1	1.7	4.1	4.6
	0	2.7	3.4	4.1	2.7	4.6	5.6
	71.98	1.7	3.2	2.7	4.6	3.6	4.3
	90	2.2	3.8	3.3	4.0	3.4	4.5
4	-18.02	7.5	6.9	7.6	2.9	7.7	7.9
	0	6.9	7.2	6.7	4.3	9.2	9.5
	71.98	2.9	6.1	5.0	7.4	8.1	10.0
	90	3.7	7.7	6.2	7.0	7.7	10.0
6	-18.02	11.8	12.1	11.0	3.5	9.5	9.4
	0	11.1	10.9	9.6	4.7	11.6	11.5
	71.98	3.5	7.9	9.7	12.3	11.7	15.1
	90	5.1	10.3	7.5	11.6	12.0	14.1

TG-4

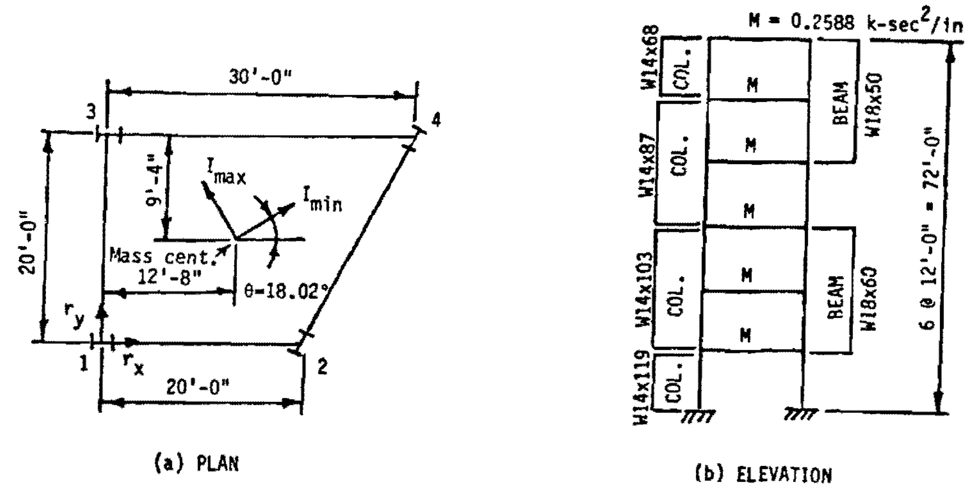


Fig. 1. Six-Story Test Structure

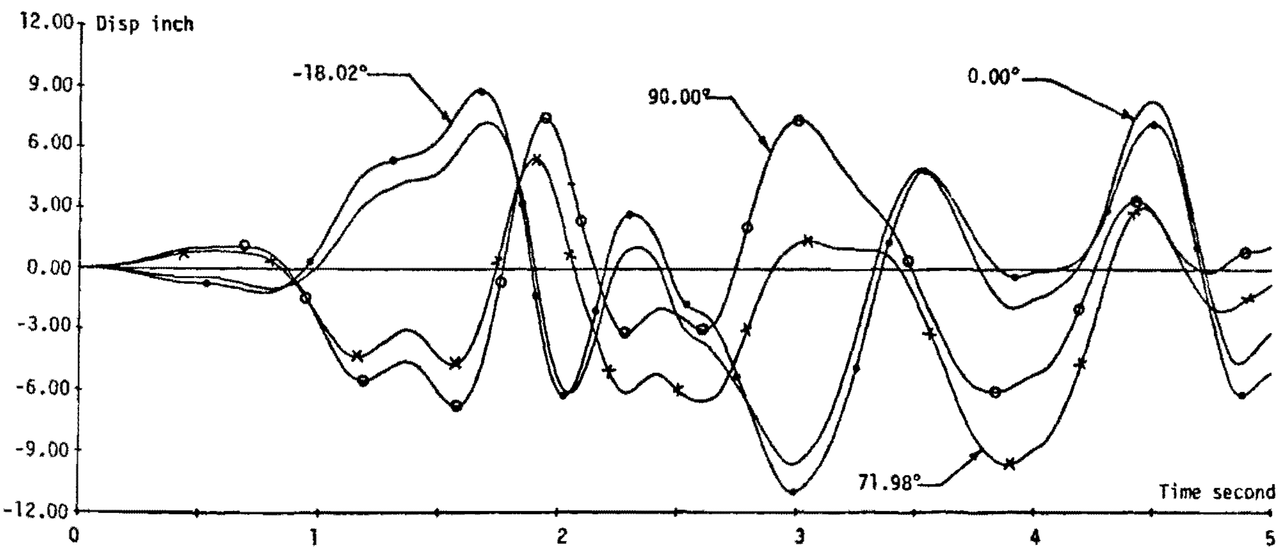


Fig. 2. Displacement in I_{min}-Direction at 6th Floor of Fig. 1

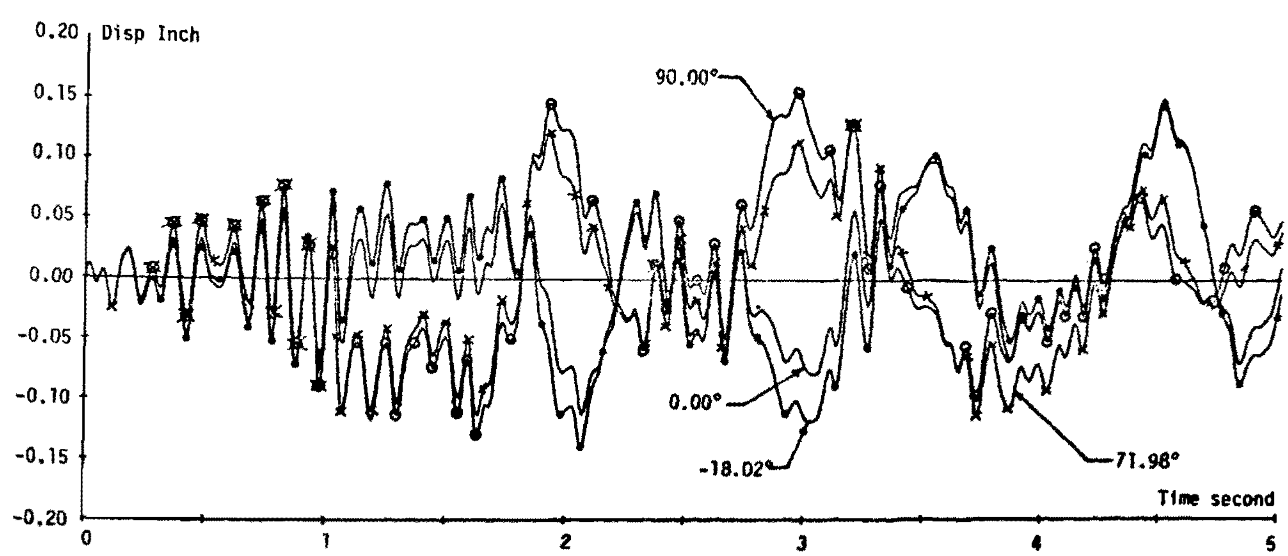


Fig. 3. Vertical Displacement of Column 1 at 6th Floor of Fig. 1

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

Chairman, W. W. Yu, University of Missouri-Rolla

Interactive Buckling in Plate Structures

S. Sridharan, Washington University

Interaction of local and overall modes of buckling under axial compression, in prismatic plate systems is a subject of current interest in design.

The paper presents a new approach to the interactive buckling in plate structures based on a combination of finite strip analysis and the theory of mode interaction. In particular the paper deals with the 'doubly symmetric' interaction, i.e. both the participating modes are assumed to have an even number of half waves along the length of the structure. The approach developed is capable of dealing with a variety of interactive buckling phenomena, viz. local and local, local and local-torsional, local and overall. Effects of initial imperfections are considered. Specific examples of plate structures are discussed in detail. Comparison with the results obtained using the AISI effective width approach is made where appropriate. A critical review of the present methods of design is presented.

Ultimate Strength of Concentrically Loaded Cold-Formed Angles

M. K. S. Madugula, T. S. Prabhu and M. C. Temple, University of Windsor

Structural steel angle sections are used extensively as leg and diagonal members of latticed electrical transmission line towers. Because it is difficult to obtain hot-rolled angle sections in bar sizes (angles with a maximum cross-sectional dimension of 75 mm or less), steel fabricators would like to substitute cold-formed angles for hot-rolled sections. Extensive test data on the behaviour of cold-formed angles is lacking. Thus this series of tests on cold-formed single angles subjected to a concentric axial load was undertaken.

The failure loads were calculated using the following codes of practice:

1. ASCE Manual 52, a "Guide For the Design of Steel Transmission Towers".

2. AISI's "Specification For the Design of Cold-Formed Steel Structural Members".

3. CSA's S136-1974 Standard for "Cold Formed Steel Structural Members".

4. ECCS's Introductory Report (1976).

In all cases ultimate loads were calculated. All factors of safety or performance factors were removed from the code equations.

A finite element program, which calculates a load-deflection curve using the initial out-of-straightness, was written. The upper limit of the curve was taken as the failure load.

Eight 45 x 45 x 3 mm and eight 65 x 65 x 4 mm specimens were tested as pin-ended columns. The initial out-of-straightness was measured before each test. The load was applied in increments to failure. The deflection was recorded at each load increment. The failure in all cases was a flexural failure about the minor principal axis. No significant rotation or bending about the major principal axis occurred.

Residual stresses were determined by the method of sectioning.

The results may be summarized as follows:

1. The four codes of practice predicted essentially the same failure loads. The ECCS predicts loads which are, in most cases, conservative. The other codes of practice predicted loads which, in general, are slightly unconservative.

2. The experimental failure loads are very close to those predicted except for two specimens whose slenderness ratio may be classed as being in the intermediate range. In this case the ECCS predicted a load close to the failure load while the other codes predicted loads which are as much as 12% too high.

3. The distribution of residual stresses in these cold-formed angles is similar to those obtained for hot-rolled angles. the maximum residual stresses are about 30% of the yield stress.

TASK GROUP 18 - UNSTIFFENED TUBULAR MEMBERS

Chairman, D. R. Sherman, University of Wisconsin-Milwaukee

Local Buckling of Steel Tubular Columns

D. F. Grimm, McDermott, Inc. and A. Ostapenko, Lehigh University

There is considerable disagreement among existing local buckling theories for large-diameter tubular columns, and relatively few test results are available. This project was undertaken to provide additional test information for the development of a more consistent design approach. The scope of this investigation included the testing of seven large-diameter, fabricated tubular columns under axial load only. The diameter-to-thickness (D/t) ratios varied from 59 to 294, and the nominal yield stresses ranged from 36 to 100 ksi. The static yield stresses, which were determined from tensile coupon tests, ranged from 30 to 90 ksi. The slenderness (L/r) ratios were less than 9.0 to minimize the effect of overall column buckling. The effect of welding residual stresses and initial imperfections on the buckling pattern was also observed. The properties of the test specimens are shown in Table 1.

Figure 1, which is indicative of typical pre-buckling behavior, shows the stress-strain relationship for Specimens T1 and T2. After some initial adjustments, the stress-strain behavior was linear up to a proportional limit of about 0.8 F/Fys. Specimens T1 and T5, with D/t ratios less than 80, strained considerably more than the other specimens before buckling.

Local buckling occurred in all of the specimens. The ultimate stress reached was limited by the formation of buckles in all of the specimens except T1 and T5. These two specimens continued to carry additional stress after the buckles became visible. Specimens T1 to T5 buckled at one end through the gradual formation of a uniform, circumferential ring bulge, as illustrated in Fig. 2. Specimens P10 and P11 buckled suddenly with an explosive sound into a diamond-shaped pattern. Surface yielding was quite extensive over the length of T1 and T5, but yielding was localized in the buckled regions of the other specimens.

Fig. 3, which is indicative of typical post-buckling behavior, shows stress-deformation curves for Specimens T1 and T2. The ultimate capacity was followed by a sudden reduction in the measured stress. The post-buckling capacities of the specimens stabilized at about 20% of the buckling stress with an overall longitudinal shortening of 3 to 6 inches.

The American Petroleum Institute used these test results along with some others to develop a new equation to predict the local buckling strength of unstiffened tubular members (1).

The design curve is defined by:

$$F_c/F_{yd} = 1.0 \qquad D/t \leq 60$$

$$F_c/F_{yd} = 1.64 - 0.23\sqrt[4]{D/t} \qquad 60 < D/t < 300$$

Figure 4 compares the design curve with the test results obtained in this investigation. The buckling stresses have been non-dimensionalized with respect to the dynamic yield stress determined by the ASTM allowable rate of straining for tensile tests.

Another empirical equation for the local buckling strength was also developed in Ref. 3. A non-dimensional parameter which reduces the scatter in the test data was used in the equation, and the buckling strengths are non-dimensionalized with respect to the static yield stress. In order to apply this equation to design, the statistical difference between the static and dynamic yield stresses must be determined.

Longitudinal residual stresses caused by welding were measured in Specimens T2, T4, and P10. The distribution and magnitude of the stresses are shown in Fig. 5 for Specimen T2. The width of the compressive zone was dependent on the diameter, and the magnitude was dependent on the yield stress level. There was no correlation between the residual stresses and either the pattern or location of the local buckles.

Initial geometric imperfections were determined from out-of-roundness and out-of-straightness measurements. There was no correlation between the magnitude of the imperfections and the dimensions of the specimens; nor was there any correlation between the measured imperfections and either the buckling pattern or load in Specimens T1 through T5 and P10. In Specimen P11, a dent remained from a previous test that may have influenced the buckling pattern and load.

References

1. American Petroleum Institute, Div. of Production, API RP 2A, RECOMMENDED PRACTICE FOR PLANNING, DESIGNING, AND CONSTRUCTION FIXED OFFSHORE PLATFORMS, 12th ed., Dallas, Texas, pp. 24, 25, 75, 76, (January 1981).

2. Johnston, B. G., Editor, GUIDE TO STABILITY DESIGN CRITERIA FOR METAL STRUCTURES, Chapter 10, Structural Stability Research Council, 3rd ed., John Wiley and Sons, Inc., New York, N. Y., 1976.

3. Ostapenko, A., LOCAL BUCKLING OF WELDED TUBULAR COLUMNS, Proceedings of the International Colloquium on Stability of Structures under Static and Dynamic Loads (Washington, D. C., May 1977), ASCE, New York, p. 367, 1977.

No.	Steel	Coupon F_y		Measured			$\frac{D}{t}$	$\frac{L}{r}$	c^{**}	Test	
		F_{ys} (ksi)	F_{yd} (ksi)	OD (in)	t (in)	L (in)				$^{*}\frac{F_c}{F_{ys}}$	$\frac{F_c}{F_{yd}}$
1	2	3	4	5	6	7	8	9	10	11	12
T1	A36	34.67	38.5	30.40	0.391	80	76.7	7.54	0.124	1.056	0.903
T2	A36	29.56	34.67	30.30	0.265	80	113.3	7.53	0.088	1.004	0.881
T3	A36	29.56	34.67	40.21	0.265	120	150.7	8.50	0.066	0.999	0.876
T4	A36	29.56	34.67	60.30	0.265	120	226.5	5.65	0.044	0.880	0.772
T5	A36	48.95	52.5	23.02	0.383	48	59.1	5.99	0.143	1.069	0.981
P10	A572 Gr50	53.13	*55.31	23.64	0.080	48	294.5	5.76	0.028	0.844	0.812
P11	A514 Tp B	90.32	*93.60	60.29	0.258	77	232.9	3.63	0.030	0.829	0.797

*Dynamic yield stress determined at 52 μ in/in/sec

** $c = \sqrt[3]{\frac{E}{F_{ys}}} \cdot \frac{1}{D/t}$

TABLE 1. SPECIMEN DATA

TG-18

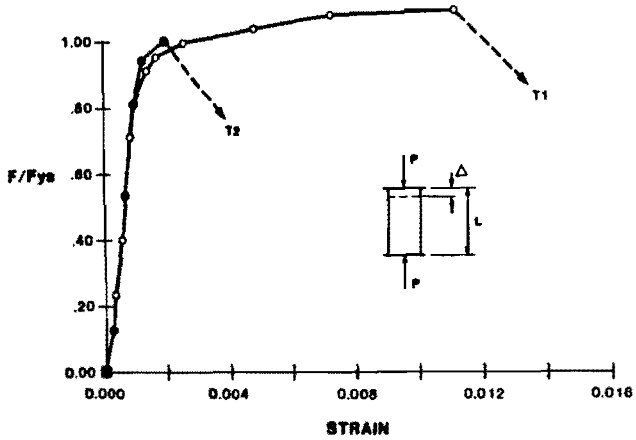


Fig. 1 Stress-Strain Curves for T1 and T2

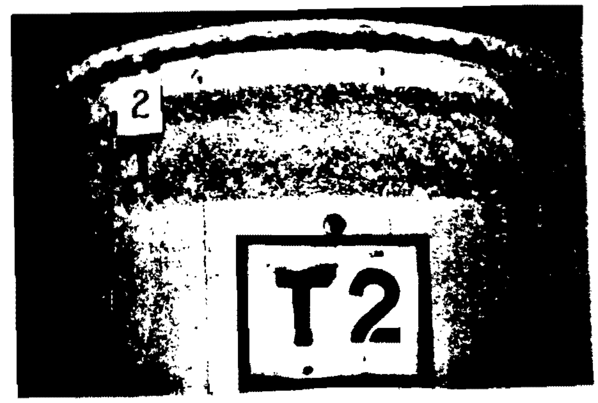


Fig. 2 Ring Bulge in T2

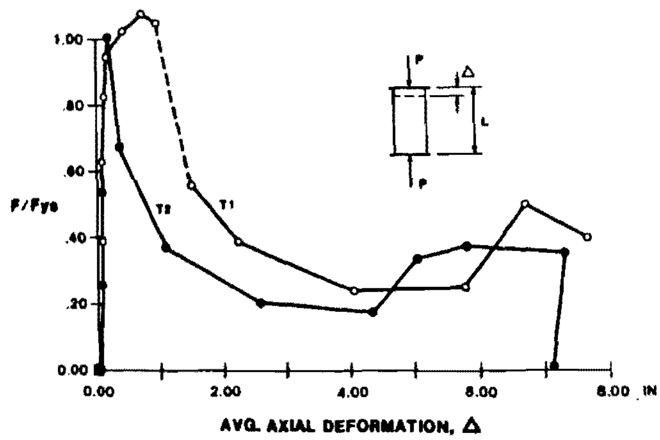


Fig. 3 Stress-Deformation Curves

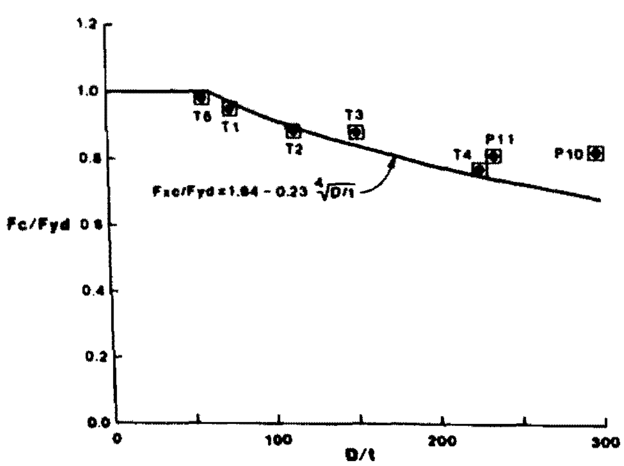


Fig. 4 Test Results and API Local Buckling Design Curve

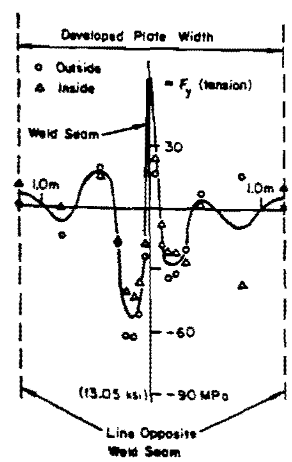


Fig. 5 Residual Stress Distribution in T2

OFFSHORE SESSION

Presiding: C. D. Miller, Chicago Bridge and Iron Company

Recorders: D. A. Ross, University of Akron
S. X. Gunzelman, Brown & Root, Inc.

RESEARCH IN GREAT BRITAIN ON THE STABILITY OF CIRCULAR TUBES

P. J. Dowling, Imperial College, London

SUMMARY

This paper reviews recent research in the UK on the buckling performance of cylindrical members as used in offshore installations. Unstiffened tubulars as well as ring stiffened, longitudinally stiffened and orthogonally stiffened cylinders are covered within the scope of the research. A nationally coordinated program of laboratory tests sponsored by the Department of Energy together with theoretical studies sponsored mainly by the Science and Engineering Research Council are discussed. The loss of strength in damaged unstiffened tubes has been studied along with the effect of initial imperfections caused by manufacturing processes on load carrying capacity of stiffened and unstiffened cylinders. The research has made significant progress in establishing techniques for the manufacture of small scale models which give results representative of large scale ones, has gone a long way towards establishing the validity of various analytical techniques ranging from simple methods to comprehensive computer based numerical methods incorporating material and geometric nonlinearity, and has provided reliable and well-documented data against which existing and new design rules can be calibrated. Finally, several areas needing further research have been identified.

1. INTRODUCTION

The topic of fatigue is perhaps the one which has received most attention from researchers in the context of the behaviour of offshore steel structures. Indeed it has been (and still is to a large extent) extremely difficult to persuade rig owners, operators and designers that there are requirements for buckling research on offshore rigs. A careful study of design guidance on buckling that existed when the first rigs were being installed in the North Sea would reveal for those who were sufficiently interested that there were deficiencies, inconsistencies and large gaps in the available information.

One such study[1] showed quite clearly that there was a need for a better understanding of buckling behaviour of cylindrical members of the types commonly used in offshore jacket construction. Since then the need has become even more acute as there is a marked tendency towards the use of more slender structures to minimize construction costs and more particularly to reduce dead weight for floating platforms and the new generation of Tension Leg Platforms (TLPs). At the time of writing the first of the new TLPs is under construction for the North Sea Hutton Field.

Although it has been customary in the United States to design rigs on an allowable stress basis, the custom in Europe for fixed rigs has been to use limit state design philosophy. Such an approach is increasingly being adopted by regulatory authorities worldwide and requires designers to consider explicity the ultimate limit state. An understanding of buckling failure and its sensitivity to imperfections is central to a rational application of limit state design as on it depends the choice of strength formulae and appropriate partial safety factors.

This situation is recognised within the UK by government departments such as the Department of Energy and the main engineering research sponsoring body, the Science and Engineering Research Council, and the majority of the research outlined below as has been sponsored by these bodies. Many of the results obtained so far have been reported in Ref(2) which contains the papers presented at an international conference held at Imperial College in 1981. The research is discussed in a sequence which reflects the increasing complexity of stiffening to cylindrical components as used offshore.

2. UNSTIFFENED TUBULARS

Unstiffened tubulars are used not only in pipelines but are extensively used as primary and secondary framing members within steel platforms. They are generally of fairly robust construction ($D/t < 60$) and the behaviour up to collapse of such tubes has been fairly well researched, particularly under the action of compressive axial and eccentric loading. More complex loading conditions such as combined axial and lateral pressurization have received less attention until relatively recently. However, the main problem receiving attention in the UK relates to the residual strength of damaged tubes. Damage is likely to be caused during launching or installation, by collisions involving supply boats or by dropping of heavy objects from deck level. the need to assess the implications on strength of damage incurred is clear.

Smith et al (Admiralty Marine Technology Establishment) have examined this problem and reported their findings in several papers [3,4,5,6].

Both full-scale and small scale tests were carried out. The full-scale tests were carried out on four bracing members removed from the BP West Sole platform. These tubes were some 8m long and had D/t's ranging from 29 to 40. Sixteen smaller scale ($\frac{1}{4}$-scale) tests were carried out on tubes with a D/ts ranging from 29 to 88. In addition four small scale tests roughly matching those of the full-scale ones were preformed to assess the scaling effect on results.

Two types of damage were simulated. One consisted of a dent, typically induced in the experimental work by applying a load through a knife edge. The second consisted of a combined dent and overall bend. The dents ranged up to four times the wall thickness, whilest the bends typically were a bow with a maximum amplitude of 0.005 times the length.

The sixteen small scale tests were on a series of cold-drawn steel tubes which were divided into four sets of nominally identical geometries (although a complete survey of actual geometries was made before testing). In each group of nominally identical tubulars one with no intentional damage was tested under axial compressive loading and another under eccentric compressive loading. The remaining two were tested under axial compression with various types and magnitudes of damage.

The results of these tests showed that whilest slight bending should cause significant strength loss (15% or more), severe denting (4 times the thickness) can cause large strength losses of the order of 50% in the medium range of D/t's (D/t = 45) used in practice. It was also concluded that by providing small scale models with representative geometries and material properties (obtained by heat treating the cold drawn steel) small scale testing provided a satisfactory way of assessing the effects of damage on tubular bracing members or other unstiffened cylinders with similar ranges of slenderness. (See Tables 1 and 2).

Analytical methods have been developed to allow the rapid evaluation of damage effects. Elasto-plastic beam-column analysis provides an accurate estimate of residual strength and stiffness for members which have been bent [6]. It has been shown that the loss of strength depends primarily on the maximum amplitude of the bow but is relatively insensitive to its location along the length of the tubular. An approximate representation of dent effects using a similar analysis has also shown satisfactory correlation with test data, although further work is needed on dents of different configuration to establish confidence in the technique.

More extensive analytical work on unstiffened cylinder stability has been done recently by Harding[7] and Batista and Croll[8]. In Ref.(7) the finite-difference form of the large deflection equations for an initially distorted cylinder are solved by dynamic relaxation and account for initial residual stressing and material non-linearity.

Although the results in the paper related to cylinders with idealised end boundary conditions appropriate to rigid ring stiffeners, the results are also of direct interest in relation to unstiffened thin walled tubulars where the buckling mode is a multi-lobe one rather than an axi-symmetric bulge type failure. It was concluded that the mode of initial imperfection particularly in the axial rather than circumferential direction, played a very important role in determining the collapse strength and certainly more than has been generally recognised to date. Such slender unstiffened tubulars are uncommon in offshore construction at the moment and indeed designers should be wary of using such tubes to minimise weight as they are highly imperfection sensitive and show catastrophic post-collapse behaviour.

The authors of Ref.(8) have developed a reduced stiffness analysis for axially loaded tubulars which provides lower bounds to this imperfection sensitive problem. Extensive comparisons made between theory and test have shown that these are valid lower bounds.

Although much work has been done in connection with pipeline buckling and will not be covered here it is worth noting the simple solutions provided by Hobbs[9] for submarine pipelines in which axial compressive loads are induced by frictional restraint of axial extensions due to temperature changes or internal pressure. Both the upward (from sea-bed) mode and the lateral snaking mode are considered within the analysis.

3. RING STIFFENED CYLINDERS

A substantial amount of research has been concentrated on the behaviour of ring stiffened cylinders. This might seem surprising in view of the extensive research relating to submarines which has been going on for many decades. Indeed it is true to say that the case of externally pressurised capped ring stiffened cylinders is well understood, and covered by existing codes.

However, the behaviour of ring stiffened cylinders under axial compression and bending has received relatively little attention until recently. It is only because such potentially offer constructional advantages (even for predominantly axial loading) that they are now the topic of intense investigation. Clearly the possibility of reducing costs and time by more extensive use of automated welding procedures because of the regular stiffening arrangement and the absences of costly welded intersections encountered with orthogonal stiffening is attractive to clients and fabricators alike. This form of construction is used for the columns of floating structures such as semi-submersibles and has been used in the new Hutton Field TLP. It also has potential advantages for the more conventional type of jacket structures. There is considerable uncertainty about the behaviour of such cylinders under the combined loading for which they must be designed. Load combinations include axial compression, bending, shear, torsion and external pressure.

Dowling and Harding[10,11,12,13,14] have investigated their behaviour both experimentally and theoretically. Their studies reported to date have concentrated on shell collapse between frames.

Three large scale cylinders with shell plating approximately 3mm thick have been tested at Imperial College[12]. Two cylinders were nominally identical and had D/t ratios of 426. They were three bay cylinders in which the end bays were made of high yield steel in order to induce collapse to occur in the mid-bay and thus away from the end effects. (In the first test the end bay collapsed as the steel supplied was thinner than specified - but it produced an extra experimental point!). The third cylinder had a D/t of 256. All cylinders were extensively surveyed for imperfections prior to testing and strains and deflexions were measured throughout the test. Imperfection coefficients were produced by Fourier analysis for use in correlating test and analytical results. The collapse loads measured varied between 0.54 and 0.86 of the cylinder squash load. The slender cylinders failed near the ring stiffeners very suddenly. In each case pronounced outward bulging of the cylinder shell was occurring adjacent to the rings prior to collapse and finally the shell snapped through to assume a lower energy multiple inward lobe buckling configuration. The residual strength of each of the two cylinders was only about forty percent of the peak loads - which were surprisingly close for the centre bay failures of these nominally identical geometries. The failure mode of the stocky cylinder was an outward plastic bulging between rings which were at relatively much closer centres than those in the other two cylinders.

Walker et al[15] tested ten small scale thin steel stiffened cylinders. Some of the geometries were duplicated and the number of bays in each model varied between 1 and 5. The thickness of the plate used was only 0.81mm and the D/t's were either 300 or 500. These smaller scale models were not as extensively monitored for out-of-plane deflexions or strains as were the larger scale ones. As in the case of the large scale models only concentric axial loading was used. The two collapse modes observed in the large scale specimens also were picked up in this test series. In the majority of shells with D/t = 300 an axisymmetric localised bulge mechanism formed at or shortly before collapse while with the two of the three shells with D/t = 500 the final buckled mode was a cyclic symmetric or multilobed form. In the third the rings were so closely spaced as to break up the mechanism associated with unstiffened cylinder failure and forced it to fail by the formation of a short wave length outward bulge around the cylinder and adjacent to a ring.

When the results of the large and small scale tests were taken together they showed a high degree of imperfection sensitivity for axially loaded ring stiffened cylinders for the range of slendernesses studied. This is illustrated in Table 3.

Analyses using geometrical and material non-linear finite difference programs have been used to study the response of ring stiffened shells for combined as well as axial loading[13]. Discrete ring stiffeners have not been modeled exactly in these studies, but boundary conditions appropriate to their presence by assuming inter frame buckling to be the critical failure mode have been used. Some information on the effect of ring spacing on loading has been produced by these studies although the analytical results for cylinders under combined compression and external pressure, and eccentric axial loading are probably the most useful data produced apart from the data on axially loaded cylinders. Sridharan et al [16] have produced a mechanism approach and an elasto-plastic finite-element axi-symmetric program[17] to study the collapse of ring stiffened cylinders. The latter theory when used with DnV tolerances (rather than measured ones) gives reasonable correlation with the small scale tests. However, as both the analysis of Ref. (13) and (17) show the problem to be very imperfection sensitive, further validation using actual measured imperfections would seem to be highly desirable. Attempts to do this have produced encouraging but limited results in Ref. (14).

Correlation of test data with the results of two established programs, STAGS-C1 and BOSOR 5 has been undertaken on behalf of Lloyd's Register of Shipping by Richards[18]. Severe restrictions were imposed on the correlation exercise by the available budget and only some general conclusions can be drawn with confidence. In principle, the sophisticated STAGS-C1 program which is a finite element program allowing for shell structures of general shape and for geometrical and material non-linearities should be capable of predicting accurately the strengths of stiffened cylinders. The main limitation is an economic one. Ring stiffened shells were modelled with 90-120° segments and imperfections were included using 10 biaxial Fourier terms. No attempt was made to model residual stresses. Good agreement was obtained nonetheless for the three large scale cylinders (12) with the theoretical and actual ultimate loads differing by less than ten percent. BOSOR 5, which accepts only axi-symmetric imperfections, was reported to give poor agreement with the same three tests. For the reader to draw his own conclusions he should, of course, study the way in which both of these programs were used to model the tests.

A problem which has been studied theoretically in a systematic fashion is that of a ring-stiffened shell under combined axial compression and external pressure. Harding[19] has reported a parametric study in which allowance was made for initial imperfections, plasticity and res residual stresses. The results have been presented in the form of interaction curves. Currently a test program is underway at Imperial College and Surrey University to provide some experimental data against which the validity of the theoretically generated curves can be judged. Galletley[20] has just reviewed the problem with particular attention being paid to the way it is treated within existing or proposed design rules.

Meanwhile Ellinas and Croll have extended the reduced stiffness analysis developed earlier for isotropic shells to stiffened shells including ring stiffened shells[21]. They have attempted to obtain a more fundamental understanding of overall shell behaviour by studying the role the membrane and flexural stiffnesses in both the axial and circumferential directions play in influencing the stability of cylindrical shells.

4. LONGITUDINALLY STIFFENED CYLINDERS

Walker et al[22,23,24] have studied the problem of the buckling of longitudinally stiffened cylinders both experimentally and theoretically. Because of the high costs involved in fabricating large specimens using normal materials and processes they used very thin steel sheeting (less than 1mm) which was specially rolled for small scale ultimate load testing to steel bridges at Imperial College(25). They developed special welding techniques which have been described in detail in Ref. (22). Using the same sheeting Dowling et al[12,26] developed alternative fabrication techniques to check the usefulness of such small models as a source of reliable data for the elasto-plastic buckling of stiffened cylinders of the type used in offshore rigs. All of the small scale models tested were one bay long and represented the longitudinally stiffened portion between ring stiffeners in an orthogonally stiffened cylinder. The models tested by the two teams attempted to reproduce boundary conditions at each end of the model which were fixed against rotation.

Some of the models tested at Imperial College duplicated those tested by Walker. Those tested by the latter, however, usually contained residual welding stresses, which were measured in some cases, and the plating was not heat treated either before or after fabrication with the result that the stress-strain curve of the material exhibited a reduced but appreciable stiffness after yield had occurred. In the Imperial College tests the models were stress relieved after fabrication and the material had a stress-strain curve representative of normal structural steels with a distinct yield plateau. Both methods of manufacture produced models which had tolerances of the order of ones normally specified within rules such as the DnV Rules.

In general models had either twenty or forty longitudinal stiffeners. The narrow-panelled cylinders generally failed in overall buckling while the broad-panelled cylinders failed by sudden local panel buckling. Three of the tests in Ref. (12) were loaded eccentrically and exhibited similar behaviour to their concentrically loaded counterparts on the side where maximum combined bending and axial stressing occurred. There was a little reserve of strength above that was obtained by using the maximum stressing from simple elastic bending calculations and applying it as concentric loading, but because the failure was of such a localised nature the redistribution capacity was less then fifteen percent.

Extensive data on initial imperfections for many of these small scale tests are contained in Refs.(26) and (27) as are the complete details of the small scale test procedures, results and conclusion. The differences between the results obtained from nominally identical models manufactured in two quite different ways were not very great in terms of collapse load achieved but the collapse modes of comparable models were different. In the models of Ref.(27) local inter-stringer panel buckling occurred at low loads, a phenomenon not observed in the tests of Ref.(26). This is thought to be due to the large residual stresses and relatively higher local imperfections in the models described in the former reference. The end support detail is also believed to have affected the experimental results, particularly those of the longer models.

Two large scale models of single bay longitudinally stiffened cylinders were tested by Green and Nelson[28,29] at the University of Glasgow. These were larger scale replicas of two small scale models tested at University College, London. One model had eight stringers while the other had twenty. The latter failed by local panel buckling but the former failed by a plastic mechanism involving both panels and stiffeners. It should be noted however that the eight stiffener model had relatively thicker walls than the twenty stiffener one. Although there was a difference in response during loading between the large scale and equivalent small scale models, the ultimate loads expressed as fractions of the squash loads were in reasonable agreement. (See Table 4).

Recently an experimental program on the problem of combined axial and external pressure loading of stringer stiffened shells has been completed in a joint program carried out by Imperial College and the University of Surrey for DnV. [30,31]

Analytical work relating to stringer stiffened models has been done by both the University College and Imperial College teams. The work at University College has been concerned with the development of simplified analytical procedures and is described in Refs. (32) and (33). On the other hand, the work at Imperial College has concentrated on the development of more detailed numerical procedures, which can be used to validate the tests and as a basis for computer studies which can supply the data needed for the formulation of simple design rules. Dowling and Ho[34] examined the effect of local curved panel buckling representative of that which occurs between stringers and later extended it to include stiffeners, Refs. (13) and (34). Currently work is nearing completion on a specially designed finite-element inelastic buckling program, NL-ASAS, developed jointly by Imperial College, W.S. Atkins and AMTE. As well as generally curved shell elements a powerful new stiffener element which can account for stiffener tripping type failures of open stiffeners of general form has been developed. This program will be used to study theoretically some of the problems of stiffened shells still needing attention and which are listed later.

5. ORTHOGONALLY STIFFENED CYLINDERS

In the recent UK research on shells only two tests, both at large scale, have been done on orthogonally stiffened cylinders. One model was tested at Galsgow(29) and the other at Imperial College(26). They were both three bay models and were designed to fail in the centre bay in a mode not involving the ring stiffeners. The information available from the series is, therefore, very limited.

The cylinder of Ref. (26) had twenty longitudinal stiffeners and failed at eighty seven percent of its squash load. Initial panel buckles appeared shortly before collapse so that only a small load margin existed between the first buckle and final collapse. This behaviour is in sharp contrast to the slender large scale ring stiffened cylinders which were tested where no such distinction could be made. In addition, the model had a considerable load capacity (eighty percent of peak load) after collapse. (See Table 4).

The Glasgow cylinder failed suddenly at sixty nine percent of the squash load due, it is thought, to a premature welding failure. (See Table 4).

Measurements of geometrical imperfections and residual stresses in actual orthogonally stiffened cylinders used in offshore construction have been reported by Dwight(36). His team at Cambridge have also developed methods for predicting the level of residual stresses in stiffened cylinders. This work is essential input to any research program on the stability of offshore tubulars.

Little analytical work involving overall buckling of orthogonally stiffened shells has been carried out recently in the UK.

6. NEED FOR FURTHER RESEARCH

Some items needing further research are listed below in no particular order of priority

(i) Data on imperfections existing in actual cylindrical components of offshore rigs are urgently needed both to aid the specification of rational tolerances and the generation of useful design curves and formulae.

(ii) More research is needed on the collapse of orthogonally stiffened shells, particularly in relation to buckling involving the orthogonal stiffening system.

(iii) Local buckling of stiffeners, both ring and longitudinal, in the tripping mode needs attention.

(iv) The influence of ring frame spacing and sizing on imperfection sensitivity under axial and bending loads needs attention.

(v) The strength of ring stiffened and orthogonally stiffened cylinders to combined axial loading and external pressure needs to be assessed experimentally as well as theoretically.

(vi) The residual strength of damaged unstiffened cylinders needs to be researched further while little information exists in relation to damaged stiffened cylinders.

(vii) Some true optimisation studies involving cost as well as structural considerations need to be carried out on the form of stiffening which is most appropriate for new and future generation rigs. For example, orthogonal stiffening of the legs of TLPs may be preferable to ring stiffening in view of the load combinations to which they are subjected.

(viii) Design rules which take advantage of the recent research work in the UK and elsewhere are needed for stringer-stiffened and orthogonally stiffened cylinders under various loadings and combinations of loading. A thorough review of existing new rules would be valuable and would reveal several inconsistencies and gaps which still exist.

(ix) Interaction between local and overall buckling in thin unstiffened tubulars has not received adequate attention.

(x) A need exists to continuously update and expand a comprehensive data bank of the type started by the SSRC task group so that maximum benefit can be obtained from the research being conducted internationally in the most efficient way possible. Many existing tests are reported in insufficient detail with respect to imperfections, residual stress levels, boundary conditions and loading conditions as to make them of little use to researchers, code drafters and designers alike.

7. CONCLUSIONS

1. Experimental and theoretical methods which quantify the effect of damage on unstiffened tubulars for certain types of damage have been developed successfully.

2. New small modelling techniques have been developed for unstiffened and stiffened steel cylinders of the type used in offshore construction which are relatively cheap to make and can be used to provide experimental data to verify design rules. Comparisons between these small scale tests and more realistic larger scale ones suggest that the latter are more suitable for verifying the sophisticated numerically based computer programs which have been produced to predict inelastic stiffened shell buckling.

3. The data which have been provided by the experimental and theoretical work carried out in the UK should help calibrate existing design rules on stringer stiffened and axially stiffened shells. They can also be used to produce a more rational set of rules when taken together with recent research carried out in other countries.

ACKNOWLEDGEMENTS

The author wishes to thank the UK Dept. of Energy and the Science and Engineering Research Council for the funding provided for most of the work described. He particularly wishes to acknowledge the cooperation of his friends and colleagues Dr. J. E. Harding, Prof. A. C. Walker, Mr. H. M. Nelson, Dr. D. M. Richards, Dr. C. S. Smith, Dr. P. A. Frieze, Mr. J. B. Dwight and Mr. W. W. Snedden.

REFERENCES

1. Ho, T. K. "Collapse of singly-curved shell structures". MSc Thesis, Imperial College, University of London, 1976.

2. "Buckling of shells in offshore structures" (Editors, Harding, J. E., Dowling, P. J. and Agelidis, N.), Granada, London 1982 also published as "Offshore Structures Engineering 111: Buckling of shells in offshore structures," Gulf Publishing, Houston, Texas, 1982.

3. Smith, C. S., Kirkwood, W. and Swan, J. W. "Buckling strength and post-collapse behaviours of tubular members including damage effects". Proc. of 2nd Int. Conf. on Behaviour of Offhore Structures (BOSS '79), Imperial College, London, August, 1979.

4. Smith, C. S., Somerville, W. L. and Swan, J. W. "Residual strength and stiffness of damaged steel bracing members". OTC Proceedings, Houston, May, 1981.

5. Smith, C. S. and Dow, R. S. "Residual strength of damaged steel ships and offshore structures". Journal of Constructional Steel Research, Vol. 1, No. 4, Granada, London Sept. 1981.

6. Smith C. S. "Strength and stiffness of damaged tubular beam columns", pp. 1-23, Ref.(2) above.

7. Harding, J. E. "The elastic-plastic analysis of imperfect cylinders." Proc. Instn. Civ. Engrs., Part II, Vol. 65, Dec. 1978.

8. Batista, R. C. and Croll, J. G. "A design approach for axially compressed unstiffened cylinders". Institute of Physics Conference, Cardiff, 1978 (Editors, Richards, T. H. and Stanley, P.) Applied Science, London, 1979.

9. Hobbs, R. E. "Pipeline buckling caused by axial loads". Journal of Constructional Steel Research, Vol. 1, No. 2, Granada, London, 1981.

10. Dowling, P. J. and Harding, J. E. "Strength of steel jacket leg components". Proc. of 11th OTC, Houston, Texas, May 1979.

11. Dowling, P. J. and Harding, J. E. "Current research into the strength of cylindrical shells used in steel jacket construction". Proc. of 2nd Int. Conf. on Behaviour of Offshore Structures (BOSS '79), Imperial College, London, August 1979.

12. Dowling, P. J. and Harding, J. E. "Experimental behaviour of ring and stringer stiffened shells". pp. 73-107 of Ref. (2) above.

13. Harding, J. E. and Dowling, P. J. "Analytical results for the behaviour of ring and stringer stiffened shells". pp. 231-256 of Ref. (2) above.

14. Harding, J. E. and Dowling, P. J. "Correlation between experimental and analytical results for the buckling of ring stiffened shells". Proc. of 2nd Conference on Integrity of Offshore Structures. Applied Science, London and New Jersey, 1981.

15. Walker, A. C., Andronicou, A. and Sridharan, S. "Experimental investigation of the buckling of stiffened shells using small scale models". pp. 45-71 of Ref. (2) above.

16. Sridharan, S., Walker, A. C. and Andronicou, A. "Local filastic collapse of ring-stiffened cylinders". Proc. Instn. Civ. Engrs. Part II, Vol. 70, 1981.

17. Walker, A. C., Andronicou, A. and Sridharan, S. "Theoretical analysis of stringer and ring stiffened shells". pp. 183-208 of Ref. (2) above.

18. Richards, D. M. "Shell buckling research and design appraisal procedures" pp. 109-136 of Ref. (2) above.

19. Harding, J. E. "Ring-stiffened cylinders under axial and external pressure loading". Proc. Inst. Civ. Engrs., (London), Part 2, Vol. 71, pp. 863-878, Sept. 1981.

20. Galletley, G. D. "Buckling of fabricated cylinders subjected to compressive axial loads and/or external pressure a comparison of several codes", to be presented at ASME Pressure Vessel Design Symposium, PVP Conference, Orlando, Florida, June 1982.

21 Ellinas, C. P. and Croll, J. G. A., "Overall buckling of ring stiffened cylinders". Proc. Inst. Civ. Engrs., Part 2, Vol. 71, pp. 637-661, Sept. 1981.

22. Walker, A. C. and Davies, P. "The collapse of stiffened cylinders". Steel Plated Structures (Edit. P. J. Dowling et. al.), Crosby Lockwood, pp. 791-901, 1977.

23. Walker, A. C. and Elsharkowi, K. "Small scale model studies of the buckling of welded thin plate structures". Proc. of Int. Conf. and Behaviour of Slender Structures, The City University, London, 1977.

24. Walker, A. C., Andronicou, A. and Sridharan, S. "Experimental investigation of the buckling of stiffened shells using small scale models". pp. 45-72 of Ref. (2) above.

25. Owens, G. W., Dowling, P. J. and Hargreaves, A. C. "Experimental behaviour of a composite bifurcated box girder bridge". to be presented at Int. Conf. on Short and Medium Span Bridges. Toronto, Canada, August, 1982.

26. Dowling, P. J. et. al. "Report on the testing of large and small scale stiffened shells under axial compression", CESLIC Report SS1, Imperial College, London, August 1981.

27. Sridharan, S. and Walker, A. C. "Experimental investigation of the buckling behaviour of stiffened cylindrical shells". Report, University College, London, February 1980.

28. Green, D. R. and Nelson, H. M. "Compression tests on large-scale, stringer-stiffened tubes". pp. 25-43 of Ref. (2) above.

29. Nelson, H. M., Green, D. R. and Phillips, D. V. "Buckling tests of offshore structures: preliminary tests of stringer-stiffened cylinders". Report, University of Glasgow, May 1980.

30. McCall, S. and Walker, A. C. "Buckling tests on stringer stiffened cylinder models subject to load combinations Vol. 1" Report to DnV, University of Surrey, Dec. 1981.

31. Agelidis, N., Harding, J. E. and Dowling, P. J."Buckling tests on stringer stiffened cylinder models subject to load combinations, Vol. 2" Report to DnV, Imperial College, London, Dec. 1981.

32. Walker, A. C. and Sridharan, S. "Analysis of the behaviour of axially compressed stringer-stiffened cylindrical shells". Proc. Instn. Civ. Engrs., Part 2, Vol. 69, 1980.

33. Walker, A. C., Andronicou, A. and Sridharan, S. "Theoretical analysis of stringer and ring stiffened shells". pp. 183-208 of Ref. (2) above.

34. Dowling, P. J. and Ho, T. K. "Effect of initial deformations on the strength of axially compressed cylindrical panels". Proc. Conf. on Significance of Deviations from Design Shapes, Applied Mechanics Group, Instn. Mech. Engrs, London, March 1979.

35. Ho, T. K. "Collapse of stiffened cylindrical shells". PhD thesis, submitted to University of London, 1982.

36. Dwight, J. B. "Imperfection levels in large stiffened tubulars". pp. 393-412 of Ref. (2) above.

Test Reference	t (mm)	R/t	L/R	Eccentricity of axial loading	Intentional damage before testing	$\frac{\text{Collapse load}}{\text{Squash load}}$
AMTE (¼-scale)						
(1) 1	2.11	14.6	69.9	0	None	0.84
2	2.12	14.5	69.9	0.32R	None	0.49
3	2.11	14.6	69.9	0	Bend and dent	0.48
4	2.12	14.5	69.9	0	Bend only	0.50
(2) 5	1.74	22.3	55.3	0	None	1.00
6	1.71	22.7	55.3	0.26R	None	0.60
7	1.72	22.6	55.3	0	Bend and severe dent	0.52
8	1.70	22.9	55.3	0	Bend and slight dent	0.61
(3) 9	1.66	30.1	43.0	0	None	1.10
10	1.73	28.9	43.0	0.20R	None	0.58
11	1.72	29.0	43.0	0	Dent	0.76
12	1.73	28.9	43.0	0	Dent	0.84
(4) 13	1.02	43.6	48.3	0	None	0.75
14	1.01	44.0	48.3	0.34R	None	0.50
15	1.03	43.1	48.3	0	Severe dent	0.53
16	1.05	42.4	48.3	0	Dent	0.64

TABLE 1 - Geometry and Results of Unstiffened Tubulars

Notes:

(1) R/t at the lower end of the range found offshore. Local damage would not normally be expected to influence performance significantly.

(2) Medium R/t in practice.

(3) R/t at the top of the practical range. Local buckling would be expected to influence post-collapse behaviour.

(4) High strength, thin-walled tubes - of interest where weight-saving is important in structural design.

Test reference	t (mm)	R/t	L/R	Intentional damage before testing	Collapse load / Squash load
AMTE (full-size)					
17	10.6	14.7	49.8	None	0.85
18	10.0	15.7	49.3	Bend	0.73
19	9.9	20.0	39.1	None	0.86
20	9.9	20.0	39.2	Slight bend & severe dent	0.57
AMTE (¼-scale and 1/6-scale)					
21	2.63	15.2	49.2	None	0.82
22	2.66	15.0	49.2	Bend	0.63
23	1.60	20.3	41.3	None	0.89
24	1.59	20.4	40.7	Bend & severe dent	0.47

TABLE 2 - Geometry and Results of Full Scale Tubulars and Corresponding Models

Test reference	t(mm) (1)	R/t (1)	L/R (1) (2)	No. of bays	Steel used (1)	Collapse load / Squash load
Imperial College (¼-scale)(3)						
25A(4)	2.87	261	1.0	3	Grade 50	0.54(4)
25B	3.52	213	1.0	3	Mild (Grade 43)	0.61
26	3.52	213	1.0	3	Mild (Grade 43)	0.62
27	3.52	128	0.2	3	Mild (Grade 43)	0.86
University College (1/20-scale)(3)						
28	0.81	150	1.0	3	Mild	0.68
29	0.81	150	1.0	3	Mild	0.69
30	0.81	150	1.0	3	Mild	0.82
31	0.81	150	1.0	5	High tensile	0.65
32	0.81	150	1.0	5	High tensile	0.69
33	0.81	150	0.58	1	Mild	0.81
34	0.81	150	0.58	1	Mild	0.74
35	0.81	250	0.33	3	High tensile	0.57
36	0.81	250	1.0	3	High tensile	0.43
37	0.81	250	1.0	3	High tensile	0.56

TABLE 3 - Geometry and Results of Ring Stiffened Cylinders

Notes:
(1) Dimensions, ratios and specifications refer to the bay where failure took place.
(2) L is the length between ring stiffeners.
(3) Some cylinder geometries are duplicated.
(4) The extra test result came from an initial, unplanned, failure in one of the end bays of cylinder 25. It was retested after the end bays had been stiffened up.

Test reference	t (mm)	R/t	L/R	No. of stringers	No. of bays	Eccentricity of axial loading	Collapse load / Squash load
University College (1/20-scale)							
38	0.81(1)	94	1.33	8	1	0	0.93
39	0.81	200	0.4	20	1	0	0.82
40	0.81	200	1.11	20	1	0	0.76
41	0.81(1)	280	0.78	20	1	0	0.61
42	0.81(1)	280	1.11	20	1	0	0.60
43	0.81(1)	280	1.56	20	1	0	0.54
44	0.81	360	1.11	20	1	0	0.51
45	0.81	200	1.11	30	1	0	0.96
46	0.81	200	0.4	40	1	0.5R	0.93
47	0.81(2)	200	1.11	40	1	0	1.05
48	0.81(2)	200	1.11	40	1	0	1.04
49	0.81(3)	280	1.11	40	1	0	0.65
50	0.81(3)	280	1.11	40	1	0	0.86
51	0.81(1)(4)	280	1.56	40	1	0	0.82
52	0.81(1)(4)	280	1.56	40	1	0	0.82
53	0.81	360	1.11	40	1	0	0.66
Imperial College - (1/4 scale)							
54	3.53	170	1.11	20	3	0	0.87
	(1/20 scale)						
55	0.84	190	0.41	40	1	0	0.96
56	0.84	190	0.41	20	1	0	0.96
57	0.84	190	1.13	40	1	0	0.95
58	0.84	190	0.41	40	1	0.25R	0.78
59	0.84	190	0.41	20	1	0.4R	0.54
60	0.84	190	1.13	40	1	0.4R	0.51
Glasgow University (1/8-scale)							
61	6.0(6)	94	1.33	8	1	0	0.89
62	3.0(6)	200	1.10	30	3	0	0.69
63	2.0(6)	280	1.56	20	1	0	0.57

TABLE 4 - Geometry and Results of Stringer Stiffened Cylinders

Notes:
(1) Cylinders tested in earlier SRC-funded programme.
(2) Similar overall dimensions but with different stringer proportions.
(3) Similar overall dimensions but with different stringer proportions.
(4) Similar overall dimensions but with different stringer proportions.
(5) Nominal steel thickness.

DISCUSSION

C. Miller (Chicago Bridge and Iron Company): In the test program you described, how thin were the cylinders tested and what design curves were used for comparison with the unstiffened cylinder test results?

Dowling: The tubes had a wall thickness of 0.805 mm and were especially rolled by the British Steel Corporation to be typical of fabricated larger scale offshore tubular members. The design curves used in the comparison were those in the Twelfth edition of the API Code. These are not the currently recommended curves.

S. Sridharan (Washington University): Could you make any comments, based on your tests, about how well the API and/or DNV codes predict the behavior of stringer stiffened cylinders?

Dowling: There seems to be good agreement between our test results and the DNV predictions. This, however, is somewhat fortuitous since the DNV predicted results were for different buckling modes than were observed.

D. Faulkner (Conoco, Inc.): Could you comment on the intentions of the British Department of Energy (D.O.E.) with respect to their future codes?

Dowling: Bill Supple could comment better than I on this. However, it seems that D.O.E. intends to present more detailed information than it has in the past. In particular, it is intended to give formulae for various buckling modes and stiffener configurations.

RESEARCH IN NORWAY IN THE STABILITY OF CIRCULAR TUBES

G. Foss(M.Sc), Principal Surveyor Industrial and Offshore Division

INTRODUCTION

Extensive research in Norway into the instability failure modes of tubulars applicable for offshore structures started only in the beginning of the nineteen seventies - coinciding in time with the exploration of hydrocarbons on the Norwegian continental shelf.

The research effort has mainly been undertaken by "The Norwegian Institute of Technology" in Trondheim and by "Det norske Veritas (VERITAS)" acting originally as a ship classification society, however in the last ten to fifteen years also acting as a "certifying authority" or "verifying agency" for offshore structures on a world wide basis.

As a non profit organization working for safety at Sea, VERITAS is feeding approximately 10 percent of its gross budget into research - some of which is used in the research of instability of tubulars.

The main objective for the research is to improve the strength verification requirements as given in various Veritas Rules (34, 35, 36, 37).

In recent years the research in Norway has been sponsored by oil companies and contractors - thus enabling larger projects and increased international communication. It is hoped that this trend continues, broadening the mutual scientific understanding which may lead to harmonized rule requirements.

BEAM-COLUMNS AND SHELLS

Tubulars for offshore applications are generally classified as belonging either to the "beam-column" family or to the "shell" family.

In Fig. 1 the geometric proportions are shown which may be used for such a classification.

As may be seen a given tubular member may in many cases be classified either as "beam-column" or as "shell". The classification itself is not important as long as all possible failure modes are considered.

Generally this means that if a tubular is classified as a beam-column then susceptibility to local shell instability must be considered, e.g. by substituting the yield stress by the shell buckling resistance.

Or, if the tubular is classified as shell but is at the same time susceptible to beam-column buckling, the stresses obtained from geometric and material linear analyses must be corrected to take into account second order effects caused by beam-column behaviour.

Based upon above it would seem natural to treat beam-columns and shells as one type of structural element. However, in order to comply with current terminology, this presentation has been subdivided into two separate main chapters; Beam-columns and shells.

SUMMARY OF RECENT, ONGOING AND PROPOSED RESEARCH IN NORWAY INTO THE INSTABILITY FAILURE MODES OF TUBULARS

SHELLS

The summary is given in fig. 2 and 3.

The numbers in fig. 3 refer back to the reference list.

The instability failure modes are visualized in fig. 5 to 9.

Summaries from references are given in Appendix 1.1.

Commentary to proposed research is given in Appendix 1.2.

Shear loading is neglected in fig. 2 (less important in most offshore structures).

BEAM-COLUMNS

The summary is given in fig. 4.

The numbers in the matrix refer back to the reference list.

The instability failure mode is visualized in fig. 10.

Summaries from references are given in Appendix 2.1.

Commentary to proposed research is given in Appendix 2.2.

Acknowledgements

The author wishes to thank colleagues in Veritas and the Norwegian Institute of Technology for their help and contributions to this paper.

In chronological order: Mr. J. Amdahl, Mr. C. Eriksson, Mr. P. Gjerde, Dr. J. Odland, Mr. K. Olaisen, Mr. E. Steen, Prof. T. H. Søreide, Dr. S. Valsgård, Dr. P. Xirouchakris.

REFERENCES

1. Grove, T., Didriksen, T.: "Buckling Experiments on 4 Large Axial Stiffened and 1 Ring Stiffened Cylindrical Shell." Det norske Veritas Report No. 76-432 (1976)

2. Grove, T., Didriksen, T.: "Buckling Experiments on 4 Large Ring-Stiffened Cylindrical Shells Subjected to Axial Compression and Lateral Pressure." Det norske Veritas Report No. 77-431 (1977)

3. Odland, J.: "Buckling of Slightly Curved Panels Subject to Axial Compression." Det norske Veritas Report No. 78-679 (1978)

4. Odland, J.: "Buckling Resistance of Unstiffened and Stiffened Circular Cylindrical Shell Structures." Norwegian Maritime Research, 6 (3), 2-22 (1978)

5. Valsgård, S.: "Finite Difference and Finite Element Methods Applied to Nonlinear Analysis of Plated Structures." Div. of Marine Structure. The Norwegian Inst. of Technology Report No. UR-79-03 (1979)

6. Odland, J.: "General Buckling of Stringer Stiffened Cylindrical Shells." Det norske Veritas Report No. 79-0042 (1979)

7. Foss, G. and Horne, J. E.: "Buckling of Beam-Columns in Braced Structures." OTC 3903 (1980)

8. Carlsen, C. A., Shao, W. J., Fredheim, S.: "Experimental and Theoretical Analysis of Post Buckling Strength of Flat Bar Stiffeners Subject to Tripping." Det norske Veritas Report No. 80-149 (1980) (Progress Report No. 1)

9. Valsgård, S., Shao, W. J., Steen, E.: "Data Collection on Geometry and Design Loads for Stiffened Cylindrical Shells in Marine Structures." Det norske Veritas Report No. 80-0156 (1980) (Progress Report No. 2)

10. Carlsen, C. A.: "Torsional Buckling of Falt-Bar Stiffeners." Part 1, Progress Report No. 3, Det norske Veritas Report No. 80-0562 (1980)

11. Valsgård, S., Steen, E.: "Simplified Strength Analysis of Narrow Panelled Stringer Stiffened Cylinders under Axial Compression and Lateral Load." Progress Report No. 4, Det norske Veritas Report No. 80-0590 (1980)

OFFSHORE SESSION (Foss)

12. Valsgård, S., Shao, W. J.: "Computer Simulation Studies of Torsional Buckling of Flat Bar Rings in Pressurized Ring Stiffened Cylinders." Progress Report No. 5, Det norske Veritas Report 80-0605 (1980)

13. Kjeøy, H., Foss, G.: "Buckling Strength and Post Buckling Behaviour of Cylindrical Members Subject to End Moments and Axial Compressive Load." Progress Report No. 6, Det norske Veritas Report No. 80-0625 (1980)

14. Kjeøy, H., Foss,G.: "Pilot Test on the Compressive Strength of an Internal Ring Stiffener." Progress Report No. 7, Det norske Veritas Report No. 80-0021 (1980)

15. Valsgård, S., Eriksson, C.: "Collapse Test on Two Point Loaded Rings with Internal Flat Bar Stiffeners." Progress Report No. 8, Det norske Veritas Report No. 81-0515 (1981)

16. Valsgård, S., Foss, G.: "Buckling Research in Det norske Veritas." Presented at "Buckling of Shells in Offshore Structures", Imperial College of Science and Technology, London, 1981

17. Eriksson, C.: "Instability of Flat Bar Ring Stiffener Subjected to External Pressure." Progress Report No. 9, Det norske Veritas Report No. 81-0516 (1981)

18. Steen, E., Valsgård, S.: "General Buckling of Orthogonally Stiffened Cylindrical Shells under Various Load Conditions." Progress Report No. 10, Det norske Veritas Report No. 81-0611 (1981)

19. Maison, B. F.: "Analytical Study for the Determination of Tubular Joint Rotational Flexibility Coefficients." Progress Report No. 11, Det norske Veritas Report No. 81-0740

20. Eriksson, C.: "Flat Bar Internal Ring Stiffener Design Formula Proposals.: Progress Report No. 12, Det norske Veritas Report No......

21. Agelidis, N. A., Harding, J. E., Dowling, P. G.: "Buckling Tests on Stringer Stiffened Cylinder Models Subject to Load Combinations." Progress Report No. 13, Det norske Veritas Report No......

22. McCall, S. and Walker, A. C.: "Buckling Tests on Stringer Stiffened Cylinder Models Subject to Load Combinations." Progress Report No. 14, Det norske Veritas Report No......

23. Steen, E., Xirouchakris, P., Valsgård, S.: "Design Proposals for Buckling of Stringer Stiffened Cylinder under Axial Compression and External Pressure." Progress Report No. 15, Det norske Veritas Report No. 82-0268

24. Foss, G., Gjerde, P., Olaisen, K.: "Ultimate Load-bearing Capacity of Beam-Columns in Braced Frames." Progress Report No. 16, Det norske Veritas Report No......

25. Valsgård, S.: "Summary Report on Progress Reports 1 to 16." Progress Report No. 17, Det norske Veritas Report No......

26. Shao, W. J., Chen, T. Y.: "Effects of Large Deflections and Initial Imperfection on the General Plastic Stability of Circular Cylindrical Shell Reinforced by Closely Spaced Rings under Hydrostatic Pressure." Det norske Veritas Report No. 80-0130 (1980)

27. Amdahl, J., Søreide, T. H.: "Energy Absorption in Axially Compressed Cylindircal Shells with Special Reference to Bulbous Bows in Collision." Norwegian Maritime Research, Vol. 9, No. 4 (1981)

28. Odland, J.: "An Experimental Investigation of the Buckling Strength of Ring Stiffened Cylindrical Shells under Axial Compression." Norwegian Maritime Research, Vol. 9, No. 4 (1981)

29. Odland, J.: "On the Strength of Welded Ring Stiffened Cylindrical Shells Primarily Subjected to Axial Compression." The Norwegian Institute of Technology, Trondheim (1981)

30. Taby, J., Moan, T. and Rashed, S. M. H.: "Theoretical and Experimental Study of the Behaviour of Damaged Tubular Members in Offshore Structures." Norwegian Maritime Research, Vol. 9, No. 2 (1981)

31. Søreide, T. H., Moan, T., Amdahl, J. and Taby, J.: "Analysis of Ship Platform Impacts." To be presented at BOSS 1982 (MIT August 1982)

32. Søreide, T. H., Amdahl, J.: "Deformation Characteristic of Tubular Members with Reference to Impact Loads from Collision and Dropped Objects." To be presented in Norwegian Maritime Research, Vol. 10, No. 2 (1982)

33. The Norwegian Maritime Directorate: "Project on Design Against Accidental Loads on Mobile Platforms." Part project PP3, Residual Strength of Damaged Elements. To be performed during '82, '83.

34. Det norske Veritas. "Rules for Classification - Steel Ships." Oslo (1980)

35. Det norske Veritas. "Ship's Load and Strength Manual." Oslo (1978)

36. Det norske Veritas. "Rules for the Construction and Classification of Mobile Offshore Units." Oslo (1975) Reprint (1980)

37. Det norske Veritas. "Rules for the Design Construction and Inspection of Offshore Structures." Appendix C - Steel Structures. Oslo (1977) Reprint with corrections (1979)

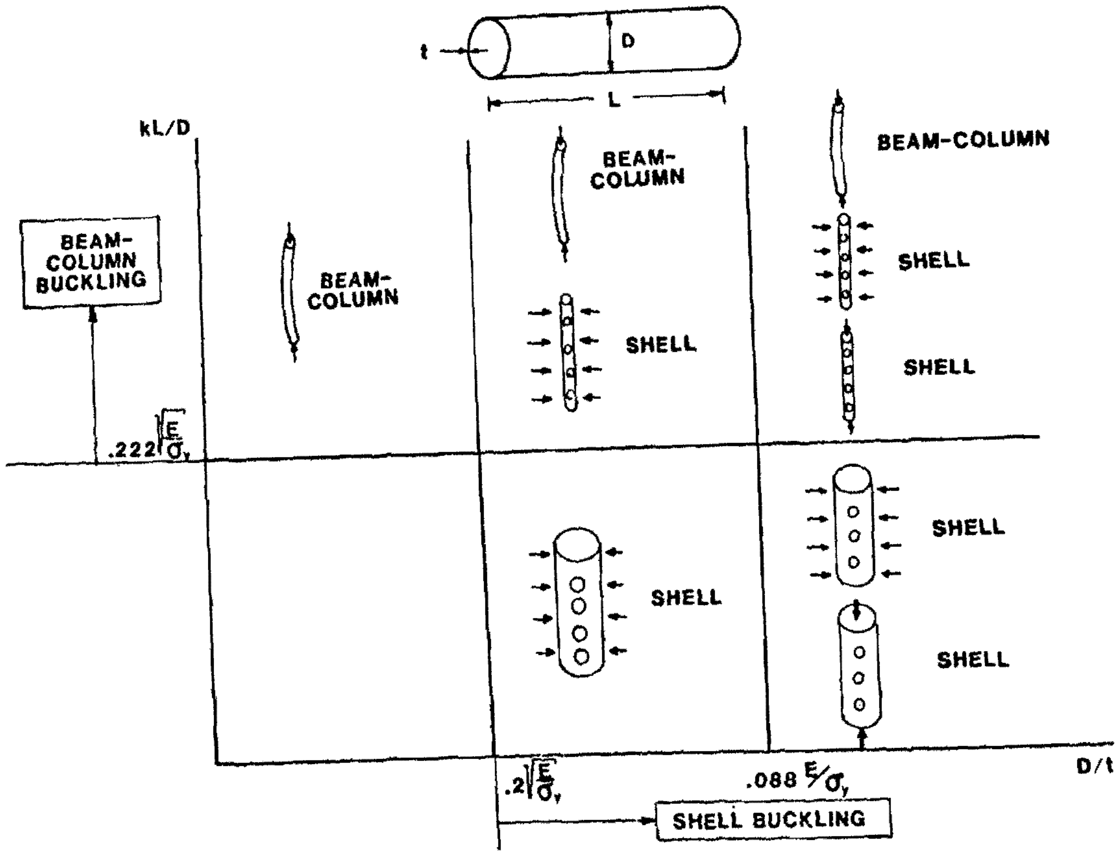


FIG1. INSTABILITY OF TUBULARS

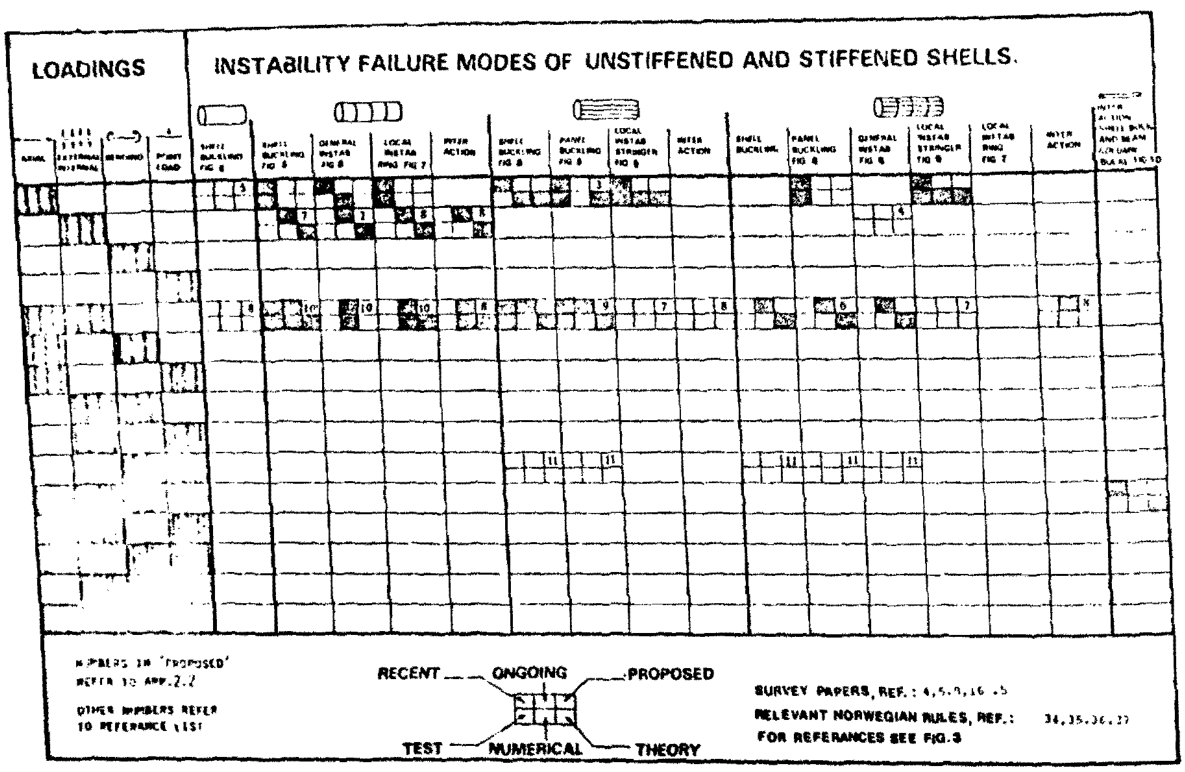


FIG.2: RECENT, ONGOING AND PROPOSED RESEARCH IN NORWAY INTO THE BEHAVIOUR OF STIFFENED AND UNSTIFFENED TUBULARS OF SHELL TYPE.

FAILURE MODES \ LOADING CASES	AXIAL	EXTERNAL	AXIAL + EXTERNAL	AXIAL+ EXTERNAL+ BENDING
SHELL BUCKLING				
SHELL BUCKLING	27,28,29	17	2,20	
GENERAL INSTAB	12	17	20,26	
LOCAL INSTAB RING	12,14,15	17	20	
INTER ACTION		17	20	
SHELL BUCKLING	3		21,22,23	
PANEL BUCKLING	1,6		11,21,22,23	
LOCAL INSTAB STRINGER	8,10			
INTER ACTION				
SHELL BUCKLING			18	
PANEL BUCKLING	1		18	
GENERAL INSTAB			18	
LOCAL INSTAB STRINGER				
LOCAL INSTAB RING				
INTER ACTION			18	
INTER-ACTION SHELL BUCKLING AND BEAM COLUMN BUCKL. FIG.				7

FIG.3 REFERENCES TO NORWEGIAN TUBULAR RESEARCH

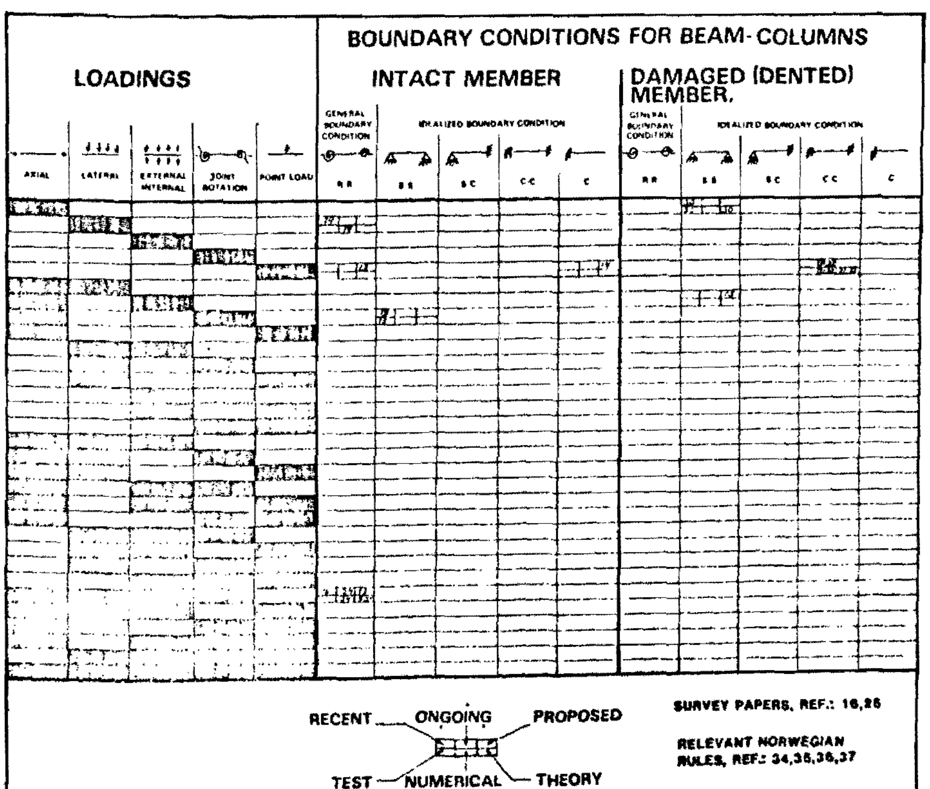


FIG.4: RECENT, ONGOING AND PROPOSED RESEARCH IN NORWAY INTO THE BEHAVIOUR OF TUBULARS OF BEAM-COLUMN TYPE

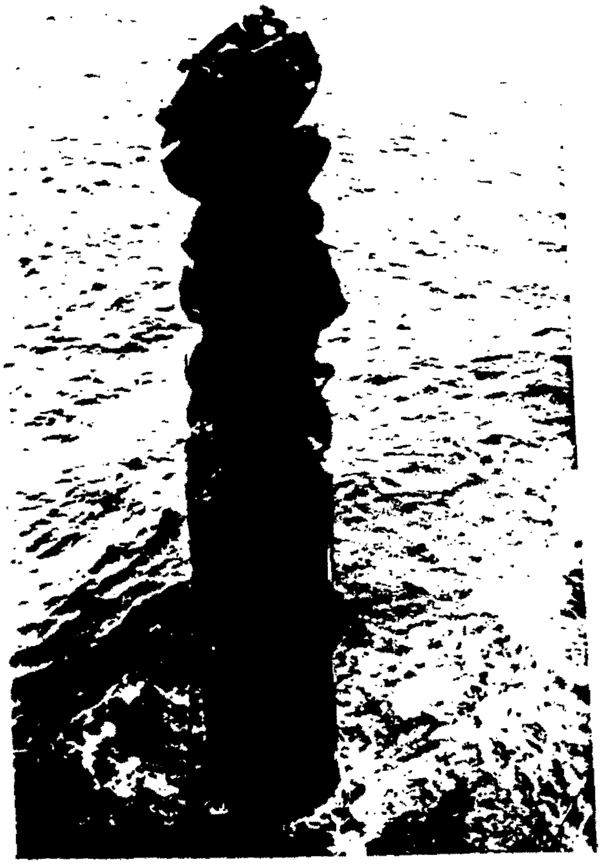


FIG.5 SHELL BUCKLING

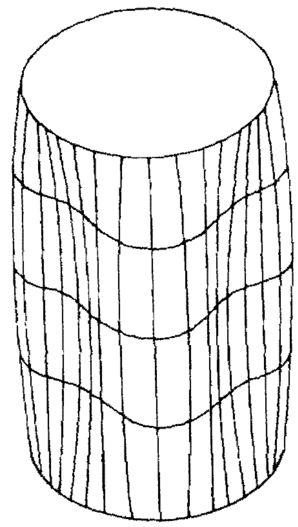


Fig.6 GENERAL BUCKLING

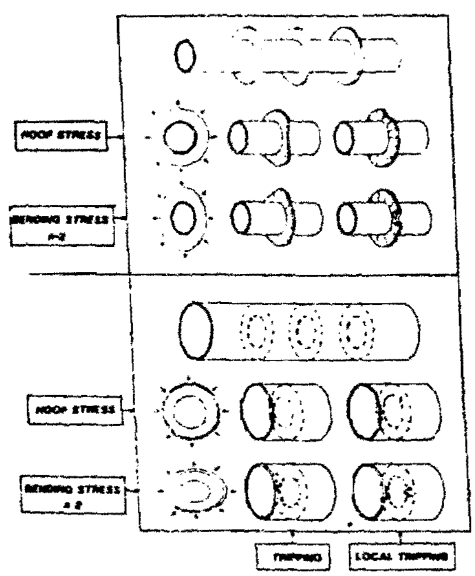


FIG.7 LOCAL INSTABILITY FAILURE MODES OF RINGSTIFFENERS

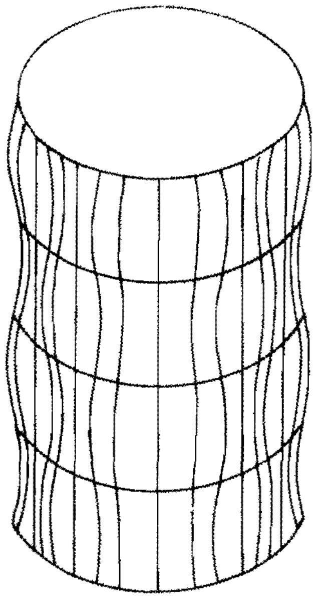


Fig.8 PANEL BUCKLING

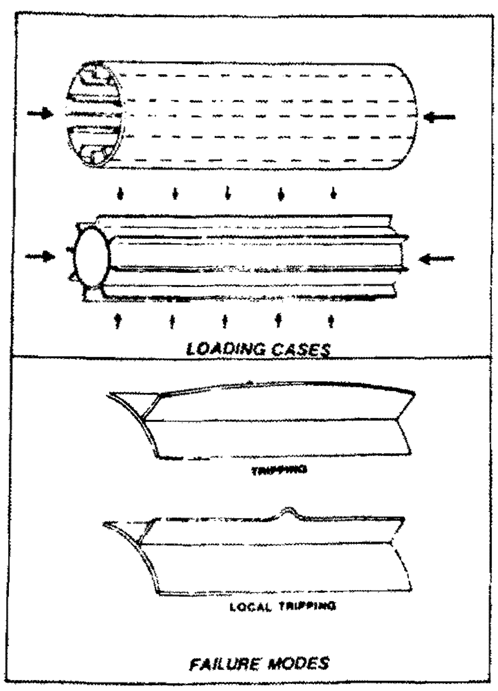


FIG.9 LOCAL INSTABILITY FAILURE MODES OF STRINGERS

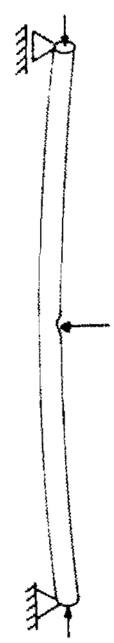


FIG.10 BEAM-COLUMN BUCKLING

APPENDIX 1: Shells and Beam-Columns; Summary Information on Recent and Ongoing Research

1. Tormod Grove and Thoralf Didriksen: Buckling experiments on 4 large axial stiffened and 1 ring stiffened cylindrical shell.

Summary

4 axial stiffened and 1 ring stiffened cylindrical models have been tested with axial compressive load. Radius and shell thickness of models are 1250mm and 3.0mm. The imperfections are measured and stored on tape:

The axial stiffened models showed buckling stresses of 50-65% of theoretical values.

The ring stiffened model showed buckling stress of about 95% of classical value or 80% of theoretical value.

2. Tormod Grove and Thoralf Didriksen: Buckling Experiments on 4 Large Ringstiffened Cylindrical Shells Subjected to Axial Compression and Lateral Pressure.

Summary

4 ring stiffened cylindrical models have been tested with combinations of axial compressive load and lateral pressure. Radius and shell thickness of models are 600mm and 2.5mm. The imperfections are measured and stored on tape.

The interaction for combination of axial compressive load and lateral pressure is shown to be linear.

The experiments with axial compression have shown that ring distances larger than the classical buckling length, will give buckling loads larger than associated with unstiffened cylindrical shells.

The experiments with lateral pressure showed buckling pressures from 0.65 to 1.0 of the theoretical pressure.

3. Jonas Odland: Buckling of slightly curved panel subject to axial compression.

Summary

The buckling resistance of curved panels has been investigated. The background for an existing design method is reviewed, and the results of a series of numerical computations are presented.

Due to the large number of parameters involved, the numerical study was too limited to serve as background for general conclusions.

4. Jonas Odland: Buckling resistance of unstiffened and stiffened circular cylindrical shell structures.

Summary

Buckling modes in unstiffened and stiffened cylindrical shells are reviewed. Classical theories for buckling of unstiffened shells are discussed, and the effect of shape imperfections as observed in experiments and as predicted by postbuckling theory is described.

Various loadcases are considered, and the behaviour under combined loading is also discussed.

Design principles for the stiffening system of cylindrical shells are reviewed, and it is described how ring frames can be proportioned conservatively to ensure that general instability is preceded by interframe buckling. The analogy with stiffened cylindrical panels is pointed out.

5. Valsgård, S.: Finite difference and finite element methods applied to nonlinear analysis of plated structures.

Summary

The present study deals with numerical discretizations for the nonlinear analysis of plated structures. Using two-dimensional Taylor series expansions the calculation of Finite Difference (FD) operators for irregular grids is discussed. This approach is capable of producing operators for irregular boundaries by including rotations along boundaries as free parameters. Truncation error terms are studied, and the influence on the error coefficients for various neighbour point configurations is indicated.

Large deflection formulations, nonlinear material modelling and incremental solution techniques are outlined. A two-dimensional FD scheme is developed, and the numerical formulation is shown for the nonlinear static analysis of thin shell structures.

Based on this scheme a linear shell program is developed. Comparisons are made with solutions from available FD-and Finite Element (FE) programs of test cases involving both membrane and bending behaviour. The obtained accuracy is discussed with reference to calculated truncation error coefficients for the particular meshes.

A rectangular flat shell element based in the Taylor series approach is developed and implemented in a existing FE shell program. The versatility of this formulation is then tested and compared to nonlinear solutions with available FD-and FE-codes. A parametric study of the capacity of imperfect plates with aspect ratio 3 is used as a test case. The interaction behaviour in biaxial in-plane compression is studied. The effect of increasing the longitudinal and transverse loading in a nonproportional way is investigated concluding that design data have to be based on proportional loading.

Using simple test examples with known analytical and/or experimental solutions three nonlinear shell analysis programs are compared with respect to accuracy and costs.

In nonlinear analysis it turns out that for a certain level of accuracy the simplest possible discretization procedure should be used regardless if this is a FD- or a FE-formulation. Moreover, due attention ought to be paid to the coding efficiency of out of core operations.

6. Jonas Odland: General buckling of stringer stiffened shells.

Summary

A simple method for calculation of the elastic general buckling stress has been proposed.

The elasto-plastic buckling stress can be estimated using one of the methods which are in use for plane panels. Due to lack of data it may be most reasonable to apply one of the most simple methods where the number of undetermined or random parameters is low.

7. Foss, G. and Horne, J. E.: Buckling of beam-columns in braced structures.

Summary

A refined method for checking instability of beam-columns, both isolated and as members of braced frames, is presented. Also, a simplified method as decribed in the nonmandatory Appendix C of Det norske Veritas Rules for Offshore Structures of 1977 is discussed. The method is based on calculation of first and second order bending stresses which are added to mean axial compressive stresses. The stresses thus obtained are then compared to the yield stress or to the local critical compressive stress, whichever approporiate.

Because special attention has been given to the inclusion of wave-current induced lateral loading in combination with frame action, the method is especially relevant to offshore structures. Also, the method specifically recognizes important parameters for jacket structures, such as local flexibility in tubular joints and the effect of hydrostatic pressure on submerged tubular members.

The reliability of the presented method has been evaluated through comparisons with results from non-linear frame analyses. The capacity estimates have been illustrated with examples and comparisons with other current methods.

The presented design method is believed to constitute a refined design method when compared to other relevant methods. It is anticipated that as a more reliable method it may ensure higher safety and at the same time reduce cost.

8. C. A. Carlsen, W. J., Shao and S. Fredheim: Experimental and theoretical analysis of post buckling strength of flatbar stiffeners subjected to tripping.

Summary

The report describes model tests on collapse and post failure strength of T struts simulating flat bar stiffeners in stiffened plates. The results are compared with two simple plastic hinge analysis methods, one neglecting and the other accounting for the effect of local tripping.

The results are of practical importance for defining local slenderness requirements in case of

a) plastic design methods assuming yield hinge mechanism,

b) local redistribution analysis in case of nonhomogeneous strength or loads, e.g. stiffened panels subjected to snearlag.

9. Sverre Valsgård, Shao Wen-jiao and Eivind Steen: Data collection on geometry and design loads for stiffened cylindrical shells in marine structures.

Summary

Geometry and Load data have been collected from drawing submitted to DnV for approval. Data for 38 cylinders have been collected and presented in tabular form.

Frequency data for geometrical relations are given in separate figures. Being a rather small sample, no final statistical conclusion can be drawn regarding mean value and standard deviation of the various geometrical relationships.

Some indications, however, emerge from the collected data.

10. C. A. Carlsen: Torsional buckling of flat-bar stiffeners, part 1.

Summary

Numerical analyses of torsional buckling of flat bar stiffeners are presented, and simplified design formulations are derived. The effect of imperfections are studied. Two different computer programs are compared.

The axial stiffness of the associated plate flange is included. The effect of rotational stiffness of the plate flange, however, will be considered in part 2 of the study and will be reported separately.

11. Sverre Valsgård and Eivind Steen: Simplified strength analysis of narrow panelled stringer stiffened cylinders under axial compression and lateral load.

Summary

A study of the strength of narrow panelled stringer stiffened cylinders between heavy rings or bulkheads is performed. Local buckling between stringers and mode interaction effects are commented on.

Based on energy formulations, simple formulas are developed for elastic buckling stresses. A beam-column on an elastic foundation is used as a simplified model for the total stringer/shell combination. First compressive outer fiber yield is taken as collapse criterion, and the effect of residual stresses are included.

For axially compressed shells a reasonable agreement between test results and the proposed design formulations is obtained. Compared to existing VERTIAS rules, the improvements are significant.

12. Sverre Valsgård and Shao Wen-jiao: Computer simulation studies of torsional buckling of flat bar rings in pressurized ring stiffened cylinders.

Summary

Ring stiffener collapse modes are characterized, and numerical modelling principles for ring stiffened cylinders described.

Using ring models and a simplified shell model, comparative studies of the ability of various computer models to describe torsional buckling of flat bar ring stiffeners have been performed.

The results show that applicable numerical models can be established. The approaches lack so far experimental verification on ring frames, but the general procedures are similar to those used for torsional buckling of straight flat bar stiffeners for which test results are available.

13. H. Kjeøy and G. Foss: Tests in buckling strength and post buckling behaviour of cylindrical members subjected to end moments and axial compressive load.

Summary

The report deals specifically with the significance of pure double curvature moments on the load bearing capacity of axial compressed cylindrical members.

A set of 14 steel columns have been tested, with variable slenderness and end moment ratios. The results indicate that the importance of end moments of pure double curvature on the loadbearing capacity of tubular beam columns have been overestimated in current design codes.

14. Helge Kjeøy and Gunnar Foss: Pilot test on the compressive strength of internal ring stiffener.

Summary

A pilot test in the compressive strength of internal ring stiffener has been performed. The model contains a sector of 90^{o} of the cylinder. An effective shell plating is included.

Collapse of the model occurred after the strains at inner fiber of the ring exceeded the yield strain, indicating that the presently used design formula to prevent side ways tripping is highly conservative for the geometry tested.

15. Sverre Valsgård and Christer Eriksson: Collapse test on two point loaded rings with internal flatbar stiffeners.

Summary

Two tests on rings with internal flatbar stiffeners have been carried out. The two rings were loaded with point loads. This gives a rough idea of what happens if external pressure is applied.

The behaviour of the rings up to and beyond failure was recorded and is discussed in this report. The failure loads are compared with analytical estimates based on outer fibre yield stress and plastic moment capacity of the rings showing quite reasonable correlations.

16. Valsgård, S. and Foss G.: Buckling research in Det norske Veritas.

Summary

The main objectives for buckling research within a classification society as Det norske Veritas is stated. Previous and present buckling research projects are outlined and the main findings summarized. Further, some areas of future research are pointed out. Much of this work has been performed because of a clearly felt need for better buckling design criteria for the various types of structural elements used within the marine environment.

Based on these efforts, published data, and practical engineering judgement, design codes and recommendations have been issued for marine structural elements different from those used in ships which traditionally have been the main concern of the society, (3), (4), (63). These recommendations cover for instance structural elements as tubular members, stiffened plates, unstiffened and stiffened cylinders, unstiffened speherical shells and spheres with openings.

17. Christer Eriksson and Sverre Valsgård: Instability of flatbar ring stiffeners subjected to external pressure.

Summary

Design formulas for elastic out-of-plane buckling are proposed for flat-bar ring stiffeners in pressurized cylindrical shells. The formulations are based on linear membrane buckling theory originating from relatively simple energy expressions for the ring and the cylindrical shell.

The effect of radial imperfection of ring stiffeners is included in the formulas. The report also contains a short discussion of plasticity effects.

18. Eivind Steen and Sverre Valsgård: General buckling of orthogonally stiffened cylindrical shells under various load conditions.

Summary

Based on a simplified energy approach, asymptotic formulas are developed for the elastic buckling strength of ring and stringer stiffened cylinders under axial compression, external pressure and shear loads. From these formulas minimum ring stiffness requirements are derived which ensure that general buckling is excluded as a possible failure mode.

Two separate methods for stability control of ring stiffeners are proposed. The first method is based on checking the relative strengths of elastic buckling modes whereas the second one uses applied stress criteria. These methods are easy to use and should be well applicable in a design code formulation.

19. B. F. Maison: Analytical study for the determination of tubular joint rotational flexibility coefficients.

Summary

The objective of this study is to provide a first step toward the development of rotational stiffness coefficient formulas for various different joint configurations. The present report contains the results of three tasks in which tubular joint rotational stiffness coefficients are analytically determined from finite element representations of tubular joints. Comparisons of the computed stiffness coefficients and the DnV Rules - Appendix C equation values are made. Eight T joint geometries were investigated in Task 1 with the purpose of verification of the analytical technique; and, calibration of an appropriate analytical model main chord length. Two different joint configurations were examined in Task 2 (single K joint analysis) and Task 3 (single triple T joint analysis). The purpose of these analyses is to determine the importance of additional in- and out-of-plane members on the local joint flexibility and to provide guidance for future studies.

OFFSHORE SESSION (Foss)

This report is the result of a subcontract issued under part project 1 "Ultimate Strength of Beam Columns" to J. G. Bouwkamp, Inc., Berkeley, Ca.

Nos. 20, 21, 22, 23, 24, 25, 26, ongoing.

27. Amdahl, J. and Søreide, T. H.: Energy absorption in axially compressed cylindrical shells with special reference to bulbous bows in collision.

Summary

The paper deals with plastic collapse of ring-stiffened cylinders. In an effort to study the collision behaviour of bulbous bows, analytical expressions for the energy absorption in cylindrical shells are presented. The various buckling models for cylinders are outlined and related to the successive development of plastic mechanism. Strain rate effects on the yield stress of steel are included to adjust for the dynamic loading in real collisions.

A series of collapse tests on ring-stiffened cylinders is described and related to the analytical formulae. The stiffener spacing is varied. Thus, axisymmetric as well as asymmetric collapse modes are obtained.

Application of the analytical models to a real tanker bulb is demonstrated. Alternative techniques for estimating the energy absorption are compared with test observations.

28. Odland, J.: An experimental investigation of the buckling strength of ringstiffened cylindrical shells under axial compression.

Summary

Stiffened and unstiffened circular cylindrical shells are extensively used as members in marine structures. Such members are often designed to carry an axial load, and the utlimate capacity is defined by a criterion for loss of stability. The difficulties in defining satisfactory design criteria are associated with initial shape imperfections and residual stresses. An experimental investigation of the buckling strength of welded and machined shells under axial compression is carried out. Before testing of the welded shells, initial shape imperfections and residual stresses were recorded. The results from the present investigation are compared to other relevant test data and design curves.

29. Odland, J.: On the strength of welded ring stiffened cylindrical shells primarily subjected to axial compression.

Summary

Stiffened cylindrical shells are extensively used as members in marine structures. The steel weight of such structures is to a large extent governed by buckling criteria. Current design methods which are basically empirical are not completely satisfactory. The difficulties are associated with initial shape imperfections and residual stresses.

OFFSHORE SESSION (Foss)

The theoretical basis of shell buckling analysis is discussed.

An experimental investigation of the buckling strength of cylindrical shells under axial compression is presented. Weld induced distortions were measured before testing.

A method for numerical simulation of weld induced distortions is established. The method is calibrated against empirical relations between basic welding and weld shrinkage parameters.

The welding simulation method is used in connection with numerical analysis of the ultimate strength of welded shells. Numerical results are first obtained for the tested shells. The established procedure is then used for systematic studies of certain effects by variation of parameters. The numerical analyses were carried out by means of an already existing computer program based on the variational finite difference technique. Some of the results are supported by simplified analyses.

30. Taby, J., Moan, T.: Theoretical and experimental study of the behaviour of damaged tubular members in offshore structures.

Summary

An efficient, simplified theoretical model of a damaged tubular member subjected to axial compressions is presented. The model is based on a yield line collapse mechanism of the dented shell. Consequent eccentricity of the load in the dented portion of the member is accounted for and ultimate strength is evaluated. The post-ultimate strength behaviour is traced based on a large deflection formulation and a plastic hinge inserted at the middle of the dented portion. An extensive experimental study is accomplished and comparisons of results are carried out. The presented model is suitable for use in analysis of large systems of tubular members.

31. Søreide, T. H., Moan, T., Amdahl, J., Taby, J.: Analysis of ship/platform impacts.

Summary

The paper deals with ship collision against platform. A general description of the collision mechanisms is given and various methods available for analyzing the collision problem are discussed. Models of energy absorption of steel platforms are identified and compared.

Simple computer programs based on plastic yield line theory have been developed for the analysis of local energy absorption, i.e. energy associated with deformation work in the vicinity of the point of impact. For studying energy absorption by beam bending of the impact element between adjacent joints yield hinge models accounting for axial restrictions as well as F.E.M. programs are applied. Constraints on the energy absorbing capability caused by local buckling, ovalization and tubular joint capacity are discussed.

Theoretical load-deformation predictions derived from the methods above are compared with experimental results from local denting and global bending of small scale tube models, showing reasonably well agreement.

The results of an introductory study with axial crushing of radially stiffened cylinders are presented with special reference to ship deformation characteristics in bow and stern collisions.

Comparison is given between analytical models and experiments on axial capacity of dented tubes. The main theory behind an efficient computer program for predicting post-damage strength of tubular members is described.

32. Søreide, T. H. and Amdahl, J.: Deformation characteristics of tubular members with reference to impact loads from collision and dropped objects.

Summary

The paper deals with impact loads on tubular members. General impact mechanics for the case of ship/platform collision are presented together with analytical and numerical methods for estimating the energy absorption capability of bracing elements. Reductions in load carrying capacity of a simple tubular member due to ovalization and local crippling are discussed and incorporated in the numerical models.

A series of tests on tubular members is performed primarily to study two effects, namely the influence from membrane forces and dynamic loading. Both horizontally free and full axially restrained members are tested, and the increase in energy absorption due to membrane forces is demonstrated. The effect from membrane action on the type of collapse is also investigated.

Dynamic tests corresponding to a real velocity of 1.0 - 2.0m per second are performed in order to study the influence from impact velocity on energy absorption capability.

33. Valsgård, S. and Kavlie, D.: Design against accidental loads on mobile platforms.

Summary

This report summarizes the recommendations for the project group on scope of work for the project "Design against Accidental Loads on Mobile Platforms". The project aims at improving present design codes and procedures and consists of six part projects dealing with:

PP1 Definition of Accidental Loads

PP2 Assessment of Resulting Damage

PP3 Residual Strength of Damaged Elements

PP4 Progressive Collapse of Platforms after Damage from Accidental Loads

PP5 Reliability of Damaged Platforms

PP6 Criteria and Design Procedures for Design against Accidental Loads

Det norske Veritas will be the main contractor with OTTER as a subcontractor on PP3 and PP4. The project is planned to be terminated medio 1984. The total cost for the project is estimated in 1981 prices to be NOK 8,2 mill.

34. Foss, G. and Edvardsen, G.: Energy Absorption during Ship-Impact on Offshore Steel Sturctures OTC 4217 (1982)

APPENDIX 2 - PROPOSED RESEARCH

2.1 Shells

1. Cylindrical shells with cut-outs.

For various reasons it may be necessary to have openings (doors, manholes) in shell structures.

Problem: When is edge reinforcement necessary and how should the reinforcement be designed and welded to the shell.

2. Design of ringstiffeners with restraints.

When internal longitudinal bulkheads are used, the ring stiffeners will have several points of radial support along the circumference.

Problem: Should ring-stiffeners be designed according to buckling criteria or only according to linear stress analysis.

3. General buckling of stringer stiffened shells with internal longitudinal bulkheads.

Problem: How much load is carried by the shell and how much is carried by the internal bulkhead.

4. Design of ring stiffeners in cylindrical shells with longitudinal stiffeners.

Design methods for rings in traditional pressure vessels have been developed. If longitudinal stiffeners are added, it must be expected that a greater part of the external pressure is transferred to the rings.

Problem: Stresses in ring stiffeners when longitudinal stiffeners are added.

5. Cylinder-cone transitions.

Tubular compression members may have transition pieces of slightly conical shape.

Problem: Is it always necessary to provide ringstiffeners at such transitions.

6.0 Effection of boundary conditions at rings.

Problem:

All existing simplified formulas for buckling strength of cylinders between heavy rings or bulkheads are based on the conservative assumption of simply supported boundary conditions. This may yield a very conservative estimate for certain ranges of geometries.

Method:

With a combined effort of analytical and numerical studies the effect of boundary conditions on load and type of failure mode can be obtained.

Objective:

To account for different boundary conditions present in real offshore structures, and to implement their effect on buckling behaviour through simplified correlated correction factors.

7.1 Effect of torsional stiffness of stringers on overall buckling behaviour

Problem:

The usual way of analyzing buckling strengths of composite elements is to separate the single construction elements and treat them independently. For the stringer stiffened cylinder the combined effect of shell and torsional buckling of stringers will influence the overall behaviour of the cylinder, and may in some cases drastically reduce the strength (mode interaction).

Method:

Simplified analytical models should be developed and correlated against numerical analysis. Test results do exist which clearly show the described behaviour.

Objective:

To get more knowledge about interactive buckling behaviour of stiffened cylindrical shells. The behaviour of single elements acting together is an important feature for the overall stability and should be included into design practice.

7.2 Explicit strength formulation for combined loads.

Problem:

The proposed formulations existing at DnV are expressed as implicit stability requirement in the case of combined loading on stiffened cylinders, i.e. the applied load levels are controlled against an upper "boundary".

Method:

Approximate analytical solutions to existing formulations could be obtained and verified against numerical analysis.

Objective:

To obtain explicit strength predictions for stiffened shells under combined loadings.

Both proportional loading and a single varying load will be of interest.

7.3 Effect of residual stresses and imperfections for buckling in the plastic range.

Problem:

The imperfection sensitivity of stiffened cylindrical shells buckling in the plastic range is little explored. Likewise the effect of residual stresses which always will be present in offshore structures should be related to ultimate load behaviours.

Method:

The basic tool for analysis will be numerical studies by general non-linear programs such as STAGSC and BOSORS.

Analytical theories are available which can verify the numerical analysis.

Objective:

To study the effect of imperfections and residual stresses for stiffened cylindrical shells in the loading cases and buckling modes where the influence is considered to be important. The results will be incorporated into design rules.

8. Strength of fabricated cylindrical shells.

Problem:

Theoretical work done on the strength of stiffened stocky cylinders have shown the importance of the imperfections; size and distribution. The present code does not consider the strength as a function of the actual imperfection level obtainable at the construction yard, but base itself on general tolerance requirements which do not directly correspond to strength as defined in the design codes.

Method:

Measurements on fabricated cylinders using an adequate method evaluated within the project. Numerical analysis of the data applying fourier series will be employed to correlate strength formulations already proposed within a recent project (A3).

Objective:

Design formulas that will optimize structural elements according to the possible level of quality that can be achieved at the actual construction yard.

9. Postbuckling behaviour of stringer stiffened cylinders.

Problem:

The postbuckling behaviour of stringer stiffened cylindrical shells subjected to axial compressive loading and external pressure will be investigated. The proposed work is of importance in estimating the energy absorption capabilities of stringer stiffened cylindrical shells against impact loads. This information is of importance to provide guidelines for the strength verification of offshore structures subjected to impacts (1).

Method:

In a recent article (2) a plastic analysis procedure was employed to analyze the postbuckling behaviour of stiffened cylinders under axial compression. A similar method is to be used to study the combined loading case (axial compression plus external pressure). Available test results will be reviewed so that suitable simplified collapse mechanisms can be assumed.

Objective:

A simple design procedure is to be developed so that the postbuckling behaviour of stringer stiffened cylindrical shells can be evaluated. The cylinders will be subjected to axial compressive loading and external pressure. The primary objective of the proposed investigation is to describe the postbuckling behaviour of cylinders in the basis of the failure mechanisms obtained from test observations.

REFERENCES

1. Foss, G., and Edvardsen, G.: Energy Absorption during Ship-Impact on Offshore Steel Structures.
Offshore Technology Conference 4217, Houston, Texas 1982.

2. Shao, W. J.: Analysis of Postbuckling Strength of Stringer Stiffened Cylindrical Shells under Axial Compression. Det norske Veritas, Report No. 81-0631. 1981.

10. Tests on ring stiffened cylinders.

Problem:

Internally ring stiffened cylindrical shells are frequently used in externally pressurized vessels such as submarine pressure hulls and buoyancy tanks. Flanged profiles may increase production costs as compared to flatbar ring stiffeners. The latter is therefore often preferred by European construction yards. With no sideways stringer support present, design codes conservatively require flanged ring stiffeners to prevent out-of-plane buckling.

Method:

Design formulas based on elastic stresses and using first outer fibre yield as collapse criteria in accordance with ideas by Donnell have been proposed. Numerical methods have been used to verify the proposed formulas. Tests are, however, needed to verify the behaviour of pressurized internally ringstiffened cylinders since this is the most realistic way to get reliable information of the complex behaviour of these structures (e.g. the sharp triangular tripping mechanism experienced in pressurized internal flatbar ring frames has been found very difficult to simulate numercially).

The theoretical and numerical work done so far in evaluating design formulas will be used in order to get the most possible information from the tests.

Objective:

Safe and reliable structures as a product of simple and adequate design formulas considering bifurcation, plasticity, residual stresses and imperfection.

11. Buckling of stiffened cylindrical shells under bending and point loads

Problem:

The buckling behaviour of ring, stringer and orthogonally stiffened cylindrical shells under bending and point loads will be investigated. Critical loads will be obtained by consideration of the interaction of the various buckling modes for cylinders subjected to bending loads. Furthermore interactive equations will be developed to obtain the critical combination of loading for stringer stiffened shells by consideration of the shell and panel buckling modes. Similarly, interactive equations will be derived for othogonally stiffened shells by consideration of the shell, panel and general buckling modes.

Method:

The method of approach will be similar to that employed in a recent article to study the buckling behaviour of stringer stiffened cylindrical shells subjected to axial compressive loading and external pressure. It is based on the same principles as the Perry-Robertson approach to column buckling. Simple equations will be developed to describe the load deflection path for deformations prior to failure. Subsequently a first yield criterion will be employed to derive estimates of the collapse loading.

Objective:

The main objective of the proposed work is to develop simple design procedures for the determination of the buckling strength of stiffened cylindrical shells subjected to bending and point loads. The type of loading considered is of importance for the design of stiffened cylindrical shells against impact loads.

REFERENCE

1. Steen, E., Xitouchakis, P. C., and Valsgård, S.: Design Proposal for Buckling of Stringer Stiffened Cylindrical Shells under Axial Compression and External Pressure. Det norske Veritas, Report No. 82-0268, (1982).

2.2 Beam-columns

12. Formulae for the ultimate load-bearing capacity of beam-columns in braced frames are being developed (ref. 2.4). Additional research work is needed to establish:

M-P-K relations at intermediate and end-point for varying D-t ratios, material properties and degree of external pressure.

Discretized M-P relations at tubular joints.

Influence of Excentrisities at tubular joints due to non-symmetrical nodes.

14. Tests to obtain discretized M-P relations at end of beam-columns.

15. The load bearing capacity of a dented tubular beam-column member subjected to the simultaneous action of axial compression and external hydrostatic pressure is sometimes encountered - the strength should be investigated.

OFFSHORE SESSION (Foss/Sherman)

DISCUSSION

R. Graham (U.S. Steel): How many specimens have been tested or what is the dollar value of the testing program at DNV?

Foss: Most of the recent research at DNV over the last three years has been sponsored by a consortium of oil companies and the dollar value of this sponsored research is about $600,000. For a 7-8 year period prior to this, the dollar value of sponsored research was about $100,000.

D. Lai (Amoco): Have tests performed by other organizations been considered when DNV proposes rules? It seems that DNV does just a few tests and then proposes new rules based on these tests.

Foss: Yes, of course DNV considers all available material when formulating rules. It is DNV's philosophy to try to provide design guides for all possible buckling modes, even when sufficient test data is lacking. DNV promotes research aimed at improving the rules especially where there is insufficient test data.

C. Miller (CBI): Is there a commentary available for comparing test results and proposed rules? For example, is there a report in which the rules for sizing rings are derived and discussed?

Foss: The background to the 1977 rules is available in Norwegian Maritime Research publications. Other sources are the reports associated with the ECCS and BS-5500 codes. However, the most recent research reports are confidential since typically such reports remain proprietary for about one year after their completion.

RESEARCH IN NORTH AMERICA ON THE STABILITY OF CIRCULAR TUBES

D. R. Sherman, University of Wisconsin-Milwaukee

INTRODUCTION

The cylindrical shape is used in a wide variety of structural applications involving considerable variation in size and proportions. Common examples of these are piping, vessels, tanks and structural elements of a framing system. Mechanical engineering and aerospace applications have provided the impetus for numerous research investigations and publications concerning the strength of cylinders under various loading conditions that include pressure, axial force, torsion flexure and combinations thereof. Only in relatively recent years, however, has the widespread construction of offshore platforms led to intensive research concerning the cylindrical tube as a member in a structure.

Tubes used as structural members typically have D/t ratios less than 100, lengths ranging from about 20 to 50 diameters, diameters from a few inches to perhaps 6 feet and are usually unstiffened. The primary loadings are axial and flexural with pressure being a secondary consequence of the enclosed nature of the shape rather than the primary reason for constructing the cylinder. Early criteria for designing tubular members were adaptations of structural specifications primarily intended for other rolled or fabricated shapes. Some modifications were based on the knowledge of the behavior of cylinders obtained from research for other applications, which generally involve much larger D/t than in structural members. The recent research on tubular members has led to design criteria specifically developed to gain the maximum benefit but safe use of the tubular properties.

The purpose of this paper is to review the course of research in North America on tubular structural members and how it has impacted design criteria. This will show the type of concerns that have faced designers as offshore construction has progressed to increasingly hostile environments. The types of problems currently being investigated will be reviewed and some conjecture on the course of future work will also be made.

Before discussing the research it may be well to recall some of the unique characteristics of a cylinder that distinguish it from other structural shapes. One of the most important is the rapid decrease in load carrying capacity after an elastic local buckle occurs. This characteristic makes a cylinder very sensitive to initial local imperfections so that real members buckle elastically at loads considerably less than theoretical predictions. Cylindrical members are also made by a number of different methods, such as hot formed seamless pipe, cold formed ERW or butt welded pipe, and fabrication using structural plates with longitudinal and girth welds. Tubes made by these different methods exhibit different stress-strain properties and residual stress patterns. Therefore, information obtained for one type of cylinder may not be directly applicable to another type.

EARLY TUBULAR RESEARCH

In papers discussing the buckling strength of structural tubes, one of the earliest references is to the work of Wilson and Newmark in the 1930's (28). This study involved the testing of fabricated cylinders in axial compression to determine the variation in the local buckling strength as a function of D/t. The results were scattered and some were quite low. The next major study on axial compression was not conducted in North America but had considerable influence on U. S. practice. This was the work Plantema did in 1946 (16) in which he developed an empirical expression for the critical local buckling strength as a function of D/t and the yield strength of fabricated tubes. More importantly for structural member applications, however, this study presented the D/t limit of $3300/F_y$ below which cylinders can develop the full compressive yield capacity without buckling locally. Several other empirical curves were also proposed to approximate or bound the data as is illustrated in 10.8 of the SSRC Guide (7).

A less widely known but important effort was the column testing conducted at Armco in 1950 (29). This data has not been greatly influencial because it involved small diameters and some sections that were really machanical tubing. However, it strongly indicated that cold formed tube data fell on a lower column curve than hot formed tubes as indicated in Figure 10.4 of the Guide (7).

In 1965, Schilling presented his landmark paper on the strength of structural tubes (18). In this paper, he summarized much of the preceding work and presented strength criteria for structural tubes. Also included in the paper were some of his own test results on beams. From this data, he recommended that the D/t limit of $3300/F_y$ could also be used to differentiate when a cylinder could develop its full plastic moment capacity.

Perhaps Schilling's paper can be thought of as marking the transition between two eras concerning tubular members. The few early studies had provided sufficient information on inelastic local buckling, column buckling and beam behavior to permit the adaptation of allowable stress design criteria to structural tubes (20). After this time, the research effort on structural tubes was greatly expanded and a variety of more specialized topics were considered. The impetus for the increased activity came from two sources. CIDECT became a significant factor in promoting the use of tubes in European construction and their research data filtered into North America. Canadian CIDECT members fostered research projects at some of the Canadian Universities. The second impetus was the interest in the U. S. offshore industry for developing refined criteria for offshore platforms. Much of the early offshore research was directed toward tubular connections. However, by 1970 it was possible to reorient some of this effort to the stability and ultimate strength characteristics of members.

RECENT TUBULAR RESEARCH

The primary question in the early 70's was whether plastic analysis concepts could be applied to cylindrical members. This was largely motivated by the desire in offshore industries to conduct risk analyses requiring a collapse prediction of highly redundant towers. Early analytical studies by Marshall (14) indicated high concentrations of curvature at the locations of the first plastic moments. Research was, therefore, undertaken by both offshore and land based industries interested in plastic design to determine if cylindrical members had the plastic rotational capacity to redistribute moments necessary to develop mechanisms (9, 19, 22). In addition, there was some flexural data related to pipeline investigations that could be used (3, 11).

Since a mixture of seamless, butt welded and ERW pipe was used in the various investigations, it is not surprising that scatter existed in the test data. However, it appeared conservative to establish a D/t limit of $1300/F_y$ below which plastic mechanisms could be developed for "normal" member proportions and loadings. Unfortunately, the investigations also cast doubt on some previous conclusions regarding tubular beams. Some tests with D/t less than $3300/F_y$ did not quite develop the full plastic moment. Some question was also raised as to whether the D/t limit should be inversely related to the yield strength. The linear inverse relation had its origins in elastic local buckling theory and the scatter in inelastic buckling data appears somewhat reduced when a different power of the yield strength is used (22). (This was also observed with respect to local buckling in axial compression (15).) The point is still open to question (8) but fortunately the variation in yield strength is not too great in tubular structural members so that D/t limits do vary greatly. In fact, API (1) had adopted a single limit for all yield strengths.

Since most members in offshore towers carry a combination of flexural and axial loads, attention was naturally directed toward beam-column studies. Several computer based analytical investigations were undertaken (19, 21, 26, 27). These involved generating moment-thrust-curvature relations, usually by subdividing the cross section and iterating to obtain a family of compatible values of moment and curvatures for fixed values of axial load. Newmark's method of numerical integration was then used to determine the load-deflection of beam column members. In some cases (21) modifications were made to include hinge lengths observed in the beam tests in order to obtain more realistic predictions of curvatures so that local buckling could be predicted. These analytical studies provided considerable information on the importance of residual stresses, effects of moment decay after local buckling, initial out-of-straightness and hydrostatic effects (23). The results of these studies formed the basis of the Marshall strut algorithm (12) which describes the overall axial load-deflection behavior which could be used for a member element in large frame analysis programs (10). Since these large frame programs involved inelastic time step dynamic analysis, it would be prohibitive to subdivide individual members. Therefore, the algorithm for member behavior is essential to obtain manageable programs.

As was noted, the beam-column algorithm was based on the results of analytical studies. A natural extention of this work was to conduct beam-column tests to verify the input assumption and conclusions of the analytical studies (24). By the end of 1980, the testing of forty cylindrical beam-columns monotonically loaded until their capacity was essentially exhausted had been completed. These tests verified the general behavior pattern of the algorithm and provided refinements for the input parameters.

The planning of platforms in seismically active regions made it apparent that knowledge of the cylic behavior of members was necessary. With the redundant nature of a tower, unrealistic behavior would be predicted under extreme conditions if members were removed when they reached their ultimate load conditions (12). It was necessary to maintain them in the structure with properly modified stiffnesses and cyclic load capacities if true responses were desired. The beam-column algorithm was capable of including cyclic behavior but proper descriptive input parameters (decay in stiffness and capacity) were open to speculation. This same problem was also being considered for land based construction in seismic regions and tests of rolled bracing members were initiated in the late 70's (6). In order to provide the hysteritic information on tubular members, 62 cyclic tests were included in the previously mentioned beam-column test program (24). In addition, two series of cross braced tubular frames subject to cyclic panel shear were undertaken (2, 17, 30). The results of these studies have provided considerable insight in the cyclic behavior of towers under extreme loads. They also provide the basis for refining the algorithms and checkpoints for the results of the INTRA program (10).

As design criteria were developed for offshore towers, the question was frequently raised as to whether the data base from testing pipe and tubing was directly applicable to fabricated members. Due to the known sensitivity of local and column buckling to residual stresses and imperfections, these were the first area of concern. Two major test programs on members fabricated according to standard offshore practice and API standards were undertaken. One involved local buckling of single cans (15) and the other concerned long members which would buckle as columns (5). Care was taken to measure the imperfections, stress-strain properties and residual stresses and to correlate these with the results. The local buckling results tend to verify Platema's conclusion for relatively heavy walls but did not indicate as severe a reduction in the critical stress for thin walls. The column tests indicate that data from fabricated compression members fell close to the old CRC curve and did not exhibit the reduction noted in the early tests of cold formed tubes.

One final topic that has come under investigation in recent years is the interaction between axial force and pressure. Members of offshore towers are inherently subject to incidental pressures, if not major pressures when they are not flooded. Interactions with both tensile and compressive loads in fabricated cylinders were studied in pipeline tests (4) and are currently part of an extensive investigation in an API project.

From this discussion, it is evident that in a little over a decade a number of topics concerning structural tubes have been intensively studied in North America. The major topics are the requirements for developing a plastic beam mechanism, post buckling and cyclic behavior of beam-columns, local and column buckling of fabricated members, and interaction between axial force and pressure. All these topics are of major concern to the offshore industries and some of them are also important in land based construction.

CURRENT AND FUTURE RESEARCH

At the present time, fabricated cylinders are the subject of most North American research. Tests of the interaction between axial load and pressure are continuing at Southwest Research Laboratories. Some very recent flexure and axial load tests at the University of Alberta involved 60 inch diameter fabricated pipe with D/t of 300. A program has been initiated at the University of Wisconsin-Milwaukee to reexamine the conclusions that were derived from the flexural tests of pipe. Speciments fabricated to offshore standards with D/t from 18 to 96 will be tested in bending. In addition to examining eight D/t ratios, two different yield strengths are being used and the influence of the longitudinal and girth welds will be observed. Another program at the University of Toronto is just getting underway to examine the local stability of fabricated beams and beam-column behavior in the post buckling regions and under cyclic conditions are continuing at Purdue University and Portland State University.

With the past and present studies, an excellent framework for understanding the behavior of cylindrical structural members will exist. Future work will probably concentrate on more detailed topics where extrapolations of exiting information has been made in a hopefully conservative manner to develop design guidelines. Some areas that have been mentioned at advisory meetings are:

1. Effects of shear and stiffening at connections on the moment and rotation capacities.

2. Capacities of members with D/t larger than 100, with and without stiffeners.

3. The effect of imperfections on inelastic local buckling of fabricated members.

4. Reinforced openings as they might exist in the large members now being proposed or used in offshore structures.

5. Beam-column behavior of fabricated members

6. The influence of load rates as it affects the yield strength in coupon tests, specimen tests and field conditions.

In all of these areas, it is necessary to not only determine the ultimate load conditions, but also the post buckling and perhaps cyclic behavior.

CONCLUSIONS

The results of the research projects referenced in this paper have had considerable effect on API RP2A (1) and on other specifications (20, 25) which include cylindrical members. Recent efforts to develop LRFD specifications have emphasized the need for more knowledge concerning the ultimate strength of cylindrical members. Other topics, such as cyclic behavior, go beyond member specification requirements but are essential for the sophisticated analyses required for predictions of structural collapse conditions. Although considerable information on member behavior is already available, it appears that more detailed studies particularly related to fabricated tubes are also necessary.

REFERENCES

1. American Petroleum Institute, "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms," API RPZA.

2. Briggs, M. J. and Maison, J. R., "Test of X-Braced Subassemblage," ASCE Preprint 3302, Chicago, Ill., October 1978.

3. Bouwkamp, J. G., "Buckling and Post-Buckling Strength of Circular Tubular Sections," OTC 2204, Proceedings 1975 Offshore Technology Conference.

4. Bouwkamp, J. G. and Stephen, R. M., "Large Diameter Pipe under Combined Loading," Transportation Engineering Journal, ASCE, Vol. 99, TE3, Paper 9907, August 1973.

5. Chen, W. F. and Ross, D. A., "Tests of Fabricated Tubular Columns," Fritz Engineering Laboratory Report 393.8, Lehigh University, September 1978.

6. Jain, A. K., Goel, S. C. and Hanson, R. D., "Hysteresis Behavior of Bracing Members and Seismic Response of Braced Frames with Different Proportions," UMEE 78R3, University of Michigan, July 1978.

7. Johnston, B. G., Guide to the Stability Design of Metal Structures, Chapter 10, 3rd Edition, John Wiley & Sons, 1976.

8. Koral, R. B., "Critical Buckling Strains of Round Tubes in Flexure," Int. J. Mech. Sci., Vol. 99, 1979.

9. Koral, R. B., "Inelastic Buckling of Circular Tubes in Bending," Journal of the Engineering Mechanics Division, ASCE Vol. 105, EM4, Paper 13944, August 1979.

10. Litton, R. W., Pawsey, S. F., Stock, D. J. and Wilson, B. M., "Efficient Numerical Procedures for Numerical Seismic Response Analysis of Braced Tubular Structures," ASCE Preprint 3002, Chicago, Ill., October 1978.

11. Manney, G. A. and McCarron, J. K., "Plastic Bending of Line Pipe," TPR 10-66, Shell Pipeline Corporation, August 1966. And update information, "Bending Characteristics of Line Pipe," April 1972.

12. Marshall, P. W., Gates, W. E. and Anagnostopoulus, S., "Inelastic Dynamic Analysis of Tubular Offshore Structures," OTC 2908, Proceedings 1977 Offshore Technology Conference.

13 Marshall, P. W., "Design Considerations for Offshore Structures Having Nonlinear Response to Earthquakes," ASCE Preprint 3302, Chicago, Ill., October 1978.

14. Marshall, P. W., "Design Criteria for Structural Steel Pipe" Proceedings Column Research Council, 1971.

15. Ostapenko, A. and Grimm, D. F., "Local Buckling of Cylindrical Tubular Columns Made of A36 Steel," Fritz Engineering Laboratory Report 450.7, Lehigh University, February 1980.

16. Plantema, F. J., "Collapsing Stress of Circular Cylinders and Round Tubes," Nat. Luchtraart Lab. Report, Amsterdam (1946); see Ref. 7.

17. Popov, E. P., Mahin, S. A. and Zayas, V. A., "Inelastic Cyclic Behavior of Tubular Braced Frames," ASCE Preprint 80-135, Portland, Ore., April 1980.

18. Schilling, C. G., "Buckling Strength of Circular Tubes," Journal of the Structural Division, ASCE, Vol. 91, ST5, Paper 4520, October 1965.

19. Sherman, D. R., "Ultimate Capacity of Tubular Members," Report CE-15, Shell Oil Company, August 1975.

20. Sherman, D. R., "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron & Steel Institute SP 604-876-7.5 M-MP, August 1976.

21. Sherman, D. R., "Analytical Study of Tubular Beam-Columns After Local Buckling," University of Wisconsin-Milwaukee Report to Shell Oil Co., April 1977.

22. Sherman D. R., "Tests of Circular Steel Tubes in Bending," Journal of Structural Division, ASCE, Vol. 102, ST11, Paper 12568, November 1976.

23. Sherman, D. R., Erzurumulu, H. and Mueller, W. H., "Behavioral Study of Circular Tubular Beam-Columns," Journal of the Structural Division, ASCE, Vol. 105, ST6, Paper 14627, June 1979.

24. Sherman, D. R., "Cyclic Inelastic Behavior of Beam-Columns and Struts," ASCE Preprint 3302, Chicago, Illo, October 1978.

25. "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," American Institute of Steel Construction, November 1978.

26. Toma, S. and Chen, W. F., "Analysis of Fabricated Tubular Columns," Journal of the Structural Division, ASCE, Vol. 105, ST11, Paper 14994, November 1979.

27. Wagner, A. L., Mueller, W. H. and Erzurumulu, H., "Ultimate Strength of Tubular Beam-Columns," Journal of the Structural Division, ASCE, Vol. 103, ST1, Paper 12670, January 1977.

28. Wilson, W. M. and Newmark, N. M., "The Strength of Thin Cylindrical Shells as Columns," University of Illinois Engineering Experiment Station Bull. 255, 1933.

29. Wolford, D. S. and Kebholz, M. J.,"Beam and Column Tests of Welded Steel Tubing with Design Recommendations," Bulletin 233, American Society of Testing Materials, October 1958.

30. Zayas, V. A., Popov, E. P. and Mahin, S. A., "Cyclic Inelastic Buckling of Tubular Steel Braces," Report UCB/EERC-80/16, University of California-Berkeley, June 1980.

DISCUSSION

R. Meith (Chevron): What size were the pipe tests you reported?

Sherman: They were 4in. - 6in. diameter electric resistance welded tubes.

Elgaaly (Bechtel, Inc.): It is my experience that tubes of high (D/t) ratios buckle before the plastic moment is reached due to a flattening (ovalizing) effect on the cross section. However, tubes of low (D/t) ratios reach plastic moment before buckling. What about tubes of intermediate (D/t) ratios?

Sherman: In these tests, with $D/t < 3300/F_y$, a large single buckle forms at buckling. Futhermore, such specimens did reach their plastic moment capacity before buckling. The focus of this investigation has been on bending of the tubes and only slight ovaling of the cross section was noted in the tests. The ovalizing reduces the section modulus less than 2% and thus has had little effect on these specimens reaching their plastic moment capacity.

J. Durkee (Consulting Structural Engineer): What effect has this research had? What design parameters have been altered as a result of it? It should be possible to determine a cost-benefit index for such a concentrated research effort conducted in such a short time.

Sherman: The impact on design specifications has been largely favorable. For example, since tubes in bending have such a large reliable shape factor, it should be possible to raise the allowable bending stress from 0.66 Fy to 0.75 Fy for such tubes. Futhermore, this should be achievable for (D/t) ratios up to 3300/Fy, whereas API currently specifies a maximum (D/t) of 60 for applying the higher allowable stress of 0.75 Fy.

I have not personally attempted a cost-benefit analysis, but I'm sure API has looked into this.

RESEARCH IN JAPAN IN THE STABILITY OF CIRCULAR TUBES

Y. Kurobane, Kumamoto University, Japan
T. Atsuta, Kawasaki Heavy Industries, Ltd., Japan
S. Toma, Hokkai-Gakuen University, Japan

INTRODUCTION

Many offshore structures mostly for oil drilling have been built in Japan by two major industry groups (Table 1); the shipbuilding companies mainly make mobile rigs such as semi-submersibles (Fig. 1) and jack-up platforms (Fig. 2), while the steel manufacturing companies fabricate fixed jacket type platforms (Fig. 3). Most of these offshore structures are owned by oil companies or oil drilling companies outside Japan and thus the design of offshore structures is based on the rules of foreign regulatory bodies of shipping such as American Bureau of Shipping, Lloyd's Register of Shipping, Det Norske Veritas, etc. Some rigs owned by domestic companies are built under the control of Japanese classification societies such as Nippon Kaiji Kyokai(1). Japan Society of Civil Engineers has also issued a design guide for offshore steel structures.(2)

Cylindrical tubular members are widely used in offshore structures for various reasons; the minimum hydro-dynamic force, the smallest outside surface to be corroded, large local strength against impact loading and large torsional rigidity. Above all, one of the biggest advantages is the large buckling strength in all directions both in local and overall. On the other hand, a joint of tubular members tends to have a complicated configulation, and its high stress concentration presents the problem of cumulative fatigue failure due to large cyclic wave forces during storms. For this reason, most research efforts on offshore structures in Japan have been directed to fatigue analysis of tubular joints by Kurobane, et al.(3) (4) and, subsequently, by Iida, Yoshida and Iwasaki et al. as summarized in References 23 and 24.

Concerning elastic stability of structural members, vast amount of studies have been done and sufficient data are available in handbooks such as the one by the Column Research Committee of Japan.(5) Most research is now directed to studies on stability in inelastic range. Stability of beam-columns is also studied by Atsuta(6) and Toma.(7) On the stability of offshore structures, however, very few research works have been published.(9) (10) In the present paper, the state-of-the-art of research in Japan on the stability of cylindrical tubes is summarized including the stability problems in tubular joints and offshore pipelines.

STABILITY OF LONG CIRCULAR TUBES

Most rules for offshore structures adopt the straight line interaction formula

$$(f_a/F_a) + (f_b/F_b) \leq 0 \tag{1}$$

as the stability check of a member subjected to both axial compressive stress, f_a, and bending compressive stress, f_b. The allowable stresses, F_a and F_b, are to be determined comsidering the stability in each loading condition and the factors of safety.

A circular tube whose diameter to thickness ratio satisfies the condition

$$D/t \leq E/(9\sigma_y) \tag{2}$$

is treated as a member not to exhibit local buckling. Most unstiffened tubular members in offshore structures have the ratio D/t of about 20 to 60 which satisfies Eq. 2. Thus, when axial loads are applied the design of these members is governed by the stability as a purely compressed long column which is given by the tangent modulus theory,

$$\sigma_{cr} = \pi^2 E_t/(KL/\gamma)^2 \tag{3}$$

in which γ (=0.354 D) is the radius of gyration of the section and KL is the equivalent length of the member depending on the boundary conditions. Determination of the tangent modulus is not straight forward. This is especially true for cold-formed seam welded tubes because stress distribution is not uniform depending on the residual stresses generated in the manufacturing process.

Kato[11] conducted a series of stub-column tests of electric-resistance-welded tubes and derived the emprical formula for tangent modulus,

$$E_t/E = 0.48/[1-0.52\{1-12(\sigma/\sigma_y)^{11}\}] \qquad (4)$$

The strengths of columns calculated by Eqs. 3 and 4 are plotted in Fig. 4. Also shown in the figure is the curve "a" of ECCS column curves and some test results on seamless hot-finished tubes and seam welded tubes. Substantial agreement is observed between the theoretical prediction curve and test results except for very short columns. It is interesting that the buckling strengths of cold-formed tube are slightly higher than those of seamless tubes.

Wakabayashi et al.[12] used a similar tangent modulus approach to calculate the strengths of electric-resistance-welded tubular columns with the following three conditions:

a. As weld condition (concentric loading)

b. Annealed condition (concentric loading)

c. Eccentric loading (eccentricity = radius of gyration of the section)

Both theoretical results and test results are plotted in Fig. 5. It is recognized that the effect of residual stress is large and that the strength can be predicted by tangent modulus theory based on stub-column tests for both cases of concentric loading and eccentric loading. In Fig. 5, the column curve of DIN4114 is also plotted for reference.

Concerning material properties and residual stresses of cold-formed manufactured tubular members, extensive tests were performed by Aoki, Fukumoto and Kato which afforded valuable stochastic data to be the bases for buckling analysis.[13],[14],[15]

STABILITY OF SHORT CIRCULAR TUBES

The ultimate strength of a short circular tube is governed by local buckling in elastic or plastic range depending on the diameter-to-thickness ratio, D/t. Since most tubular members used in offshore structures satisfy Eq. 2, their full plastic strengths can be expected. However, the deformation capacities of these members after yielding, which also vary with the D/t ratio, are important to develop both plastic capacity and hysteretic damping capacity of the structure subjected to dynamic loadings such as wave, wind and earthquake motions.

Tubular columns used in building structures are classified into four ranks depending on the plastic deformation capacity.[16] Table 2 shows recommended D/t ratios in each rank. As shown in the table, design story shear force for ductile structures can be reduced by the multiplier D_s, which is a measure of damping capacity for each story.[17] In order to increase the damping capacity of the structure, it is effective to decrease the D/t ratio such that the local buckling that controls the ultimate capacity takes place well within a strain-hardening range.

Kato[18] carried out extensive stub-column tests on circular hollow sections made by different manufacturing processes. Figures 6 and 7 summarize the test results where the following abbreviations are used to distinguish the manufacturing processes:

ER: tube cold-formed with a longitudinal ER - welded seam

UO: tube cold-formed by U-O press with two longitudinal SA-welded seams

RB: tube cold-formed by roll bender with a longitudinal SA - welded seam

SP: tube cold-formed helically with a spiral SA - welded seam

CS: tube made by centrifugal casting

SL: hot-finished seamless tube

All the base materials are of structural grade low-carbon steels.

These figures depict how the axial compressive stress and strain at the maximum load, denoted by σ_m and ε_m respectively, are related with dimensions and mechanical properties of stub-columns.

It is apparent in Fig. 6 that the maximum stress exceeds the yield stress as the parameter $\alpha = (c\sigma_y/E)\ (t/D)$ becomes greater than about five. This property of compact sections is attributable to strain-hardening of materials and plays an important role in increasing the energy-absorbing and damping capacity of the structure. Kato observes that no definite difference in column capacity exists between different manufacturing processes, if exception is made for the cast steel tube.

The deformation capacities of stub-columns fall into three groups as represented by three straight lines in Fig. 7. These are cast steel tube (CS), seamless tube (SL) and cold-formed tubes (ER, UO, RB, SP).

Attention should be paid to the fact that the seamless tube having a flat yield plateau in its stress-strain curve may buckle and reach the maximum load as soon as compressive stresses reach the yield stress. This happened in Kato's tests; the deformation capacity of the seamless tube decreases discontinuously when α becomes less than about fifteen. The same results have been reported by Suzuki et al.[19] concering tests on as-rolled and annealed circular hollow sections.

Kato et al.[20] examined the following semi-empirical equation derived from the Gerard's proposal based on the deformation theory:

$$\sigma_{cr} = \frac{2}{3}\sqrt{E_s\ E_t}\ \ \frac{t}{R} \tag{5}$$

where σ_{cr} denotes the critical stress for a column cross section. The secant modulus E_s and the tangent modulus E_t were given by the model stress-strain curves prepared for both columns with and without residual stresses. The critical stresses estimated by Eq. 5 were found to agree with observed σ_m for compact sections. Further, it was also possible to predict the strains at critical stresses.

Since both the critical stress and deformation capacity vary with the stress-strain relationships, it is more practical, rather than the expression 5, to establish the empirical formulae of σ_m and ε_m for the type of tube used in the design. Kato[18] evaluated the following prediction equations for the electric resistance welded tube:

$$\frac{\sigma_m}{{}_c\sigma_y} = \frac{\alpha}{1.18 + 0.777\ \alpha} \tag{6}$$

$$\frac{\varepsilon_m}{\varepsilon_y} = 0.525\ \alpha \tag{7}$$

for the range $5 \leq \alpha \leq 30$. Kato also proposed rotation capacity formulae for beam-cloumns derived on the assumption that the maximum bending stress was also given by Eq. 6.

Suzuki et al.[19] carried out a series of stub-column tests on cold-formed welded tubes (ER, UO and RB types) in high-strength steels and derived formulae similar to Eqs. 6 and 7. The types of steel used are high-yield low-alloy steel with the specified minimum UTS of 539 MPa and quenched and tempered low-alloy steel with the specified minimum UTS of 785 MPa. The observed σ_m and ε_m were found to be approximately represented by the following equations regardless of steel grades and manufacturing processes:

$$\frac{\sigma_m}{{}_c\sigma_y} = 0.0163\alpha + 0.929 \tag{8}$$

$$\frac{\varepsilon_m}{\varepsilon_y} = 0.272\alpha\ \ - 0.496 \tag{9}$$

for the range $5 \leq \alpha \leq 22$. It is to be noted that, although the values of $\sigma_m/{}_c\sigma_y$ are essentially the same as Kato's test results, Eq. 9 gives a significantly lower value of $\varepsilon_m/\varepsilon_y$ compared with Eq. 7.

Suzuki et al. also carried out a non-linear finite element analysis on various stub-columns. An example of load-deformation curves obtained by the analysis is illustrated with the corresponding test results in Fig. 8. The analysis was found to be a powerful tool for interpreting the test results in both qualitative terms. Some of the findings were:

a. The higher the yield stress, the lower the $\sigma_m/{}_c\sigma_y$ and $\varepsilon_m/\varepsilon_y$ ratios.

b. Increasing a strain-hardening modulus was more effective in increasing $\varepsilon_m/\varepsilon_y$ than $\sigma_m/{}_c\sigma_y$.

c. Residual bending stresses contained in the electric-resistance-welded tube decreased the carrying capacity of the stub-columns under test.

d. The column made of material with a sharp yield point demonstrated a smaller deformation capacity than the column made of gradually yielding material, as mentioned before in connection with annealed stub-columns.

The study of deformation capacity of pipelines during the laying operation is also a research subject today. As shown in Fig. 9, when a short stinger is used, the linepipe undergoes large bending moments at the upper bent and lower bent parts. The large curvature causes flattening of the section, which requires a non-linear analysis. Kimura et al.(21) studied this problem by the use of the computer program, PAPS, which bent pipe in both elastic and inelastic regions. Two of the typical results are shown in Fig. 10 together with the test results. It is seen that the moment carrying capacity is rapidly reduced by the effect of flattening of the section. This is especially true for pipes with a large D/t ratio. Authors emphasize that linepipes should be controlled by the curvature rather than the bending moment during being laid.

STABILITY OF STIFFENED CIRCULAR TUBES

Stiffened circular tubes are often used for the stabilizing columns of semi-submersible platforms. The stabilizing columns constitute not only the structural members to support the main deck but also the stabilizing components to maintain floating stability, which requires the columns to have a large diameter. Accordingly, these columns are stiffened by longitudinal stiffeners and transverse rings to reinforce buckling strength.

Fujita et al.(22) reported on the buckling strength of cylindrical shells reinforced with longitudinal stiffeners and transverse rings under the loadings of axial compression and pure bending. The characteristic equations in elastic region were formulated by applying Galerkin's method to Donnel's shell equation adding the reinforcing terms due to stiffeners and rings. After calculations of panel buckling, the buckling load of stiffeners was determined by the use of frame work model in which the effective width of stiffeners were computed iteratively. Some of the calculated results are shown in Fig. 11 and Table 3. It was found that the panel buckling takes place first and the frame instability occurs later; the effect of ring stiffener arrangements on the overall stability of the specimens used are not very large. The buckling loads of the panel can be predicted closely by the theory, but the buckling loads of the frame structure stiffened by the longitudinal stiffeners and transverse rings are slightly over-estimated than the test. It was also found that the buckling load of stiffeners was not very high compared with the panel buckling load.

SPECIAL TOPICS IN STABILITY OF CIRCULAR TUBES

Local Buckling in Tubular Joints

In trussed structures local buckling may occur at member ends adjacent to connections. Although the buckling strength varies with details of connections, data obtained so far on this subject is very limited. The Japanese building specification for tubular structures (9),(23) recommends to dimension the tube reducer and the tube-to-through-gusset plate joint under compression as shown in Figs. 12 and 13. These recommendations are applicable only to compact sections and are based on tests with cold-formed manufactured tubes.

When the D/t ratio is less than 50 for tubular members of mild steel, local buckling stresses for properly designed joints are in general sufficiently high such that a compression member could be designed based on the flexural buckling strength of the member. An important exception to this general rule is the local buckling of a compression brace in an overlapped K - joint. Mode of failure in the latter instance is illustrated in Fig. 14.

The tubular K - joint with two tension and compression braces reaches the ultimate capacity owing to out-of-plane bending deformations of the chord and brace walls in local areas where the compression brace is welded. So-called punching shear failure usually takes place after significant plastic deformation was produced in the local areas of the tube walls. Kurobane (23),(24) proposed the formula to calculate the resistance of the tubular K - joint P_u as shown in Table 4. This formula gives a greater value of P_u as the gap g between the braces decreases and as the braces overlap each other. Overlapping tension and compression braces is effective in stiffening the K - joint. Also in a fatigue performance, the past tests evidenced that K - joints with overlapping braces were superior to those with extended braces.(25)

The joints with overlapping braces, however, fail at a load lower than the value predicted by the formula in Table 4, when the brace diameter to thickness ratio d/t becomes greater than a certain value.

The compression brace walls of these joints sustain out-of-plane deformation as shown in Fig. 14 and lose stability.

The non-dimensionalized maximum stress on a compression brace, namely $P_u/(A_b\ \sigma_y)$, is plotted against $(E/\sigma y)(t/d)$ in Fig. 15 for all the overlapped K-joints for which these data are available, where A_b is the cross sectional area of the compression brace and σ_y is the yield strength of the compression brace measured with tensile coupons. The solid circles and squares in the figure denote the test results in which ultimate capacities were governed mainly by crushing of compression braces as described above. The open circles and squares indicate the test results where plastic deflection of chord walls was the primary factor in failures of the joints. The circular and square marks were used to distinguish between cold-finished and hot-finished tubes. It is to be noted that all the specimens failed in a mixed failure mode; those with solid marks were selected deliberately from the test results by Kurobane(26), Togo et al.(27), Koning et al.(28) and Ochi et al.(29)

The following conclusions may be drawn from Fig. 15. The capacity of the overlapped K-joint is controlled also by instability of compression brace walls in the way that $P_u/(A_b\sigma_y)$ decreases with $(E/\sigma_y)(t/d)$. Within the range of data available the relationships between them are represented by a straight line for cold-finished braces as shown in the figure. Although the data is still insufficient, the test results suggest that hot-finished tubes have lower compressive strength in relation to their yield stresses. The capacity of the compression brace is found to be far smaller than the capacity of the stub-column represented by Eq. 6, if comparison is made on the assumption that ${}_c\sigma_y$ is nearly equal to σ_y.

The local buckling of the compression brace in the K-joint with extended braces has not been found in the existing data. There are a few examples where local buckling was observed in chords of K-joints.(28),(30) The chords were under combined bending and compression and reached a maximum stress close to the stub-column strength. Further study on the local buckling in unstiffened tube-to-tube joints with compact sections is presently in progress in Kumamoto University.

Tube-to-tube joints by welding sustain severe stress concentrations due to plate bending and unavoidable sharp notches at the weld toes, which could cause premature local failures of the joints owing not only to instable tube walls but also to fatigue and fracture. A promising solution of these difficulties is the use of steel castings for the nodes.(31) An example of cast steel nodes for a jack-up rig is illustrated in Fig. 16.

Stability of Liner Tubes of Pipelines

As the depth of oil or gas well gets deeper, the crude oil and gas contain more amount of corrosive elements such as H_2S, CO_2 or brine under higher pressure and higher temperature conditions. Pipelines to transport these crude products are therefore faced with severe corrosion problems. In order to solve these problems, the use of pipes lined with stainless steel liner tubes are considered in several oil and gas fields.

When a metal lined pipe of this type is used for a high pressure gas pipeline, the thin liner tube may sustain a buckling failure called "implosion", an example of which is shown in Fig. 17. The mechanism of implosion is explained by Fig. 18; the high pressure gas leaks through an infinitesimally small crack in the liner into the interface between the two pipes; the interface pressure is accumulated after a period of time. Once the internal working pressure is shut down by some operational reasons, the interface pressure can not decrease rapidly and pushes the liner tube from the outside to cause implosion.

Yamamoto and Matsubara(31) calculated the critical buckling pressure of liner tubes by solving the non-linear contact problem in large deflection analysis. As shown in Fig. 19, the critical buckling stress σ_{cr} of liner tube is controlled by the looseness or tightness of two pipes; the σ_{cr}/σ_y value gets higher as the interface gap G/D gets smaller; further, it is recognized that the compressive fit-in stress in liner tube works effectively to prevent the implosion.

Yoshida et al.(33) developed a method to fit a liner tube with high compressive fit-in stress. In the method, a liner tube is first inserted in an outer pipe, which is then heated while the liner tube is cooled. A hydrulic pressure is next applied and the liner tube is expanded plastically to touch on to the outer pipe. After cooling the outer pipe, its thermal shrinkage produces a tight fit double pipe. In Fig. 19, some test results on implosion strength of these tight-fit pipes by Yoshida et al. are plotted, which show good agreement with the theoretical calculation by Yamamoto el al.

CONCLUSIONS

Research in the stability of circular tubes in Japan has been progressed in relation with tubular columns of building structures. Especially, inelastic stability of cold-formed seam welded tubular columns are extensively studied. In the meantime, many offshore structures are being constructed based on the rules of regulatory bodies outside Japan.

There is tendency for offshore structures to get bigger, and heavy sectional members of high quality material are getting used. For example, a jacket type structure planned to be used in the North Sea of 400 meter water depth will have high strength steel columns unstiffened of 10 meter in diameter and 150mm in wall thickness. Once the fabrication technology has been developed (34), it may practically be constructed. There is no way to know the actual stability strength of such huge tubular members by testing. Theoretical analyses are the only tools to predict the strength.

ACKNOWLEDGEMENTS

The authors wish to thank Professor Ben Kato, University of Tokyo, Dr. Suguru Sakamoto, Sumitomo Metal Industries, Ltd., Dr. Jaap Wardenier, Delft University of Technology and Professor Yoshiyuki Yamamoto, University of Tokyo for affording to them valuable materials for preparation of this report.

REFERENCES

1. Nippon Kaiji Kyokai, "Rules and Regulation for Construction and Classification of Ships, Section P: Mobile Offshore Units," in Japanese

2. Offshore Structures Research Committee, "Recommendations for the Design of Offshore Steel Structures (tentative)," Japan Society of Civil Engineers, 1973, in Japanese

3. Kurobane, Y., Mitsui, Y., Atsuta, T. and Toma, S., "Fatigue Design of Welded Joints in Trussed Legs of Offshore Jack-up Platform," 10th Congress of IABSE, Sept. 1976.

4. Atsuta, T., Toma, S., Kurobane, Y. and Mitsu, Y., "Fatigue Design of an Offshore Structure," OTC 2607, Offshore Technology Conference, May 1976

5. Column Research Committee of Japan, "Handbook of Structural Stability," Corona Publishing Co. Japan, 1971

6. Chen, W. F. and Atsuta, T., "Theory of Beam Columns," Vol. 1 and Vol. 2, McGraw Hill Book Co., 1976

7. Toma, S., "Analysis and Design of Fabricated Tubular Columns," Ph. D. Dissertation, Purdue University, West Lafayette, IN., Dec. 1980

8. Suhara, T., "Buckling and Ultimate Strength of Offshore Structures," Bulletin of the Society of Naval Architects of Japan, No. 569, Nov. 1976, in Japanese

9. Tubular Structures Committee, "Recommendations for the Design and Fabrication of Tubular Structures in Steel," Architectural Institute of Japan, Feb. 1980, in Japanese

10. Steel Reinforced Concrete Structures Committee, "Specifications for the Design of Steel Tubu-Concrete Composite Structures," Architectural Institute of Japan, Feb. 1980, in Japanese

11. Kato, B., "Column Curve for Cold-Formed and Welded Steel Tubular Members," Stability of Steel Structure, Preliminary Report, Second International Colloquium at Liege, ECCS, April 1977

12. Wakabayashi, M., Nonaka, Y., and Nishikawa, K., "Experimental Studies on Electric Resistance Welded Pipes," Annual Report of Disaster Research Institute, Kyoto University, No. 12A, Mar. 1969, in Japanese

13. Aoki, T., and Fukumoto, Y., "Stochastic Material Properties and Estimate of Residual Stresses of Cold-Formed-Welded Steel Tubular Members with Small Diameters," Proceedings of Japan Society of Civil Engineers, No. 314, Oct. 1981, in Japanese

14. Kato, B and Aoki, H., "Residual Stresses in Cold Formed Tubes," Journal of Strain Analysis, Vol. 13, No. 4, 1978

15. Kato, B., "Mechanical Properties of Cold-Formed Structural Steel Tubes," Document to Com. 8, ECCS

16. Structural Standard Committee, "Ultimate Capacity and Deformation Capacity of Building Structures," Architectural Institute of Japan, June 1981, in Japanese

17. Kato, B., and Akiyama, H., "Seismic Design of Steel Buildings," Preprint 81-099, International Convention of ASCE, New York, N. Y., May 1981

18. Kato, B., "Local Buckling of Steel Circular Tubes in Plastic Region," Proc. Int'l Colloquium on Stability of Structures under Static and Dynamic Loads, SSRC/ASCE, Washington, D. C., March 1977

19. Suzuki, T., Kokawa, T., Kato, M., Warisawa, T. and Kurimoto, T., "Study on Ultimate Behavior of High-Strength Steel Tube under Axial Compression, Part 1 ---Buckling Strength of Stub and Long Columns, Part 2 ---Large Deformation Elastic Plastic Analysis of Axisymmetric Shell," Summary Papers, Annual Conference of Architectural Institute of Japan, Fukuoka, Sept. 1981, in Japanese

20. Kato, B., Akiyama, H. and Suzuki, H., "Plastic Local Buckling of Structural Tubes Subjected to Axial Compression," Trans. Architectural Institute of Japan, No. 204, Feb. 1973, in Japanese

21. Kimura, T., Itogaki, S., Takada, K., and Fujita, Y., "Experimental and Analytical Studies of the Elastoplastic Behavior of Offshore Pipelines During Laying," OTC 3737, Offshore Techonolgy Conference, Houston, May 1980

22. Fujita, Y., Nakata, Y., Sawayanagi, M., and Matsumoto, Y., "Buckling Strength of Stiffened Cylindrical Shells," Journal of the Society of Naval Architects of Japan, No. 135, June 1974, in Japanese

23. Kurobane, Y., "New Developments and Practices in Tubular Joint Design," IIW Doc. XV-488-81, XIII-1004-81, May 1981

24. Kurobane, Y., "Recent Developments in Tubular Joint Design," Preprint 81-002, International Convention of ASCE, New York, N. Y., May 1981

25. Otake, F., Sakamoto, S., Nakazato, T. and Minoshima, T., "Fatigue Strength of Overlapping Tubular N-Joints," Proc. OTC, Paper 3964, May 1981

26. Kurobane, Y., "Welded Truss Joints of Tubular Structural Members," Memoirs, Faculty of Engineering, Kumamoto University, Vol. XII, No. 1, Dec. 1964

27. Togo, T., "Experimental Study on Mechanical Behavior of Tubular Joints, "Doctoral Dissertation, Osaka University, Jan. 1967, in Japanese

28. Koning, C.H.M. de and Wardenier, J., "The Static Strength of Welded CHS K-Joints," Stevin Report, No. 6-81-13, Delft University of Technology, July 1981

29. Ochi, K., Kurobane, Y. and Makino, Y., "Further Tests on CHS K-Joints with Various d/D Ratios," Technical Report, Kumamoto University, Vol. 30, No. 2, Oct. 1981

30. Wardenier, J. and Koning C.H.M. de, "Investigation into the Static Strength of Welded Warren Type Joints made of Circular Hollow Sections," Stivin Report, No. 6-77-5, Delft University of Technology, July 1977

31. Ohba, H., Susei, S., Sakai, Y., Atsuta, T., Ohkuma, Y., Ohminami, R. and Tamura, A., "The Development of Casting Leg Nodes for a Jack-up Rig," Journal of Petroleum Technology, July 1980

32. Yamamoto, Y. and Matsubara, N., "A Proposal of Design Formula for Liner Tubes Subjected to External Pressure," Suimon-Tekkan, No. 127 1981, in Japanese

33. Yoshida, T., Matsui, S., Atsuta, T. and Itoga, K., "Corrosion Problems of Pipeline and a Solution," OTC 3891, Offshore Technology Conference, May 1980

34. Kanatani, F., Matsui, S., Atsuta, T., Nagai, H. and Nakayama, S., "New Welding Technologies for Thick Plate of Offshore Structures," OTC 4199, Offshore Technology Conference, May 1982

Table 1 Offshore Rigs Built in Japan (as of 1982)

Firms		Semi-sub	Jack-up	Jacket
Shipbuilding Companies	Mitsubishi Heavy Ind.	15	5	1
	Mitsui Zosen	7	15	7
	Hitachi Zosen	4	18	3
	Kawasaki Heavy Ind.	1	3	1
	Ishikawajima Harima Heavy		1	3
	Sumitomo Heavy Ind.	1	4	
Steel Manufacturing Companies	Nippon Kokan		6	4
	Nippon Steel Corp.			29
	Kawasaki Steel Corp.			2
Others		1		
Total (Japan)		29	49	50
Total (World)		120	337	396

Table 2 Maximum D/t Ratio Recommended for Building Structures [16]

Rank of Structure		I	II	III	IV
Yield Stress (MPa)	235	50	70	100	no limit
	324	36	50	73	no limit
Shear force factor Ds		0.3 - 0.45	0.3 - 0.55	0.3 - 0.6	0.4 - 0.7

Table 3 Compressive Buckling Stresses of Stiffened Circular Shells (Fujita, et al. [22])

Specimen		Panel Buckling			Frame Buckling			
Dimension	No. of Ring	critical stress kg/mm^2 Test	critical stress kg/mm^2 Theory	No. of waves (theory)	critical stress kg/mm^2 Test	critical stress kg/mm^2 Theory	Reduction rate %	No. of waves (theory)
$\frac{R}{t} = \frac{500}{0.6}$	1	18.51	18.55	72 x 8	19.65	25.22	22.1	0 x 2
	2	18.72	18.75	72 x 9	19.22	25.57	24.8	0 x 3
$\frac{R}{t} = \frac{1000}{1.2}$	1	14.72*	18.55	72 x 8	17.35*	25.22	31.2	0 x 2
	2	18.41	18.75	72 x 9	20.06	25.57	21.4	0 x 3

(* Flat plate buckling)

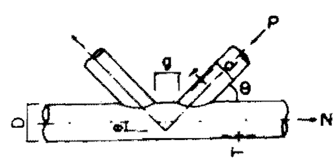
Joint Type	K-Joint  $n = N/(A_c \sigma_y)$ $A_c = \pi(D-T)T$ g is given in mm and takes a negative value when braces overlap.
Ultimate Strength Formula	$P_u = f_d f_{gT} f_\theta f_n f_y T^2 \sigma_y$ $f_d = 2.11(1+5.66\frac{d}{D})$ $f_{gT} = [1+\frac{0.00904(\frac{D}{T})^{1.24}}{\exp(0.508\frac{g-3.04}{T}-1.33)+1}](\frac{D}{T})^{0.209}$ $f_\theta = \frac{1-0.376\cos^2\theta}{\sin\theta}$ $f_n = (1+0.305n-0.285n^2)$ $f_y = (\frac{\sigma_y}{\sigma_u})^{-0.723}$
Applicable Range	$0.19 \leq \frac{d}{D} \leq 1.0$ when $15 \leq \frac{D}{T} \leq 60$ $0.19 \leq \frac{d}{D} \leq 0.6$ when $60 < \frac{D}{T} \leq 102$ $-30 \leq \frac{g}{T} \leq 40$

Table 4 Ultimate Strength Formula for K - Joint (Kurobane [23, 24])

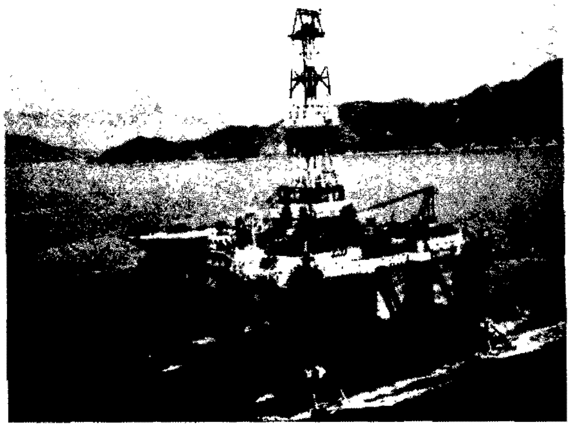


Fig. 1 Semisubmersible Drilling Rig
(Courtesy of Mitsubishi Heavy Ind.)

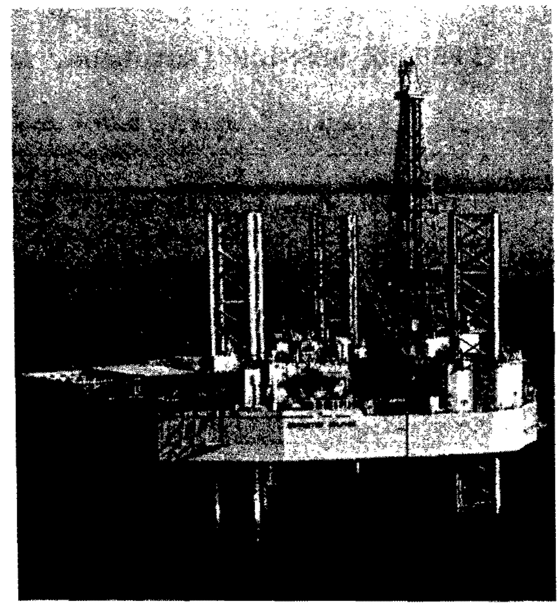


Fig. 2 Jack-up Rig
(Courtesy of Mitsui Zosen)

Fig. 3 Jacket Structure
(Courtesy of Nippon Kokan)

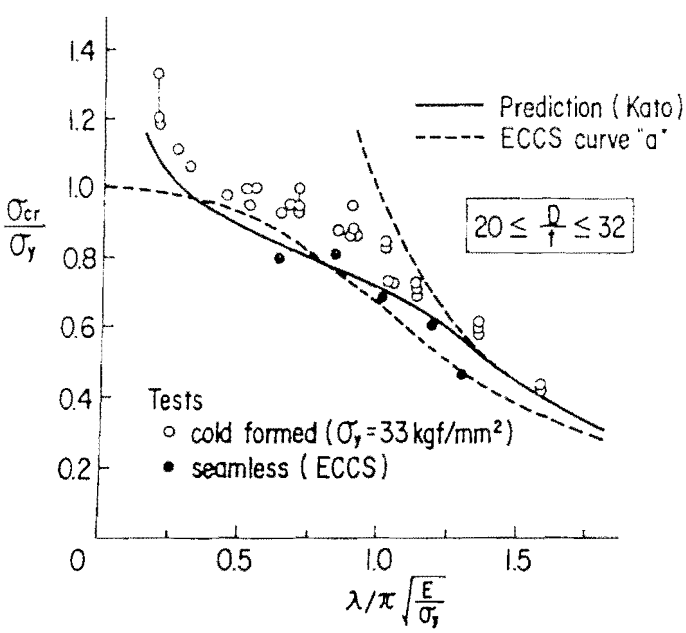


Fig. 4 Prediction of Tubular Column Strength (Kato [11])

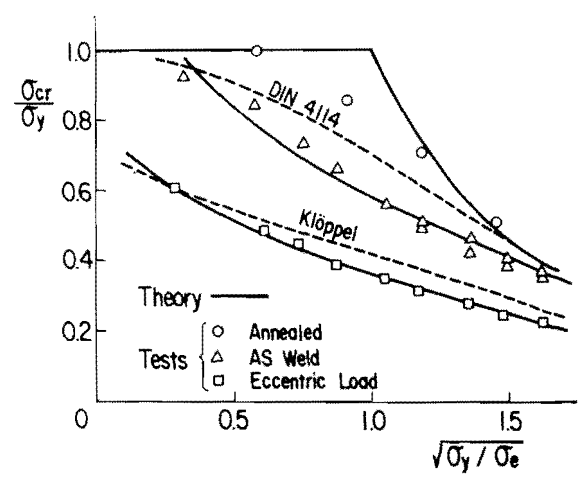


Fig. 5 Strength of Electric-Resistance-Welded Tubular Col
(Wakabayashi, et al. [12])

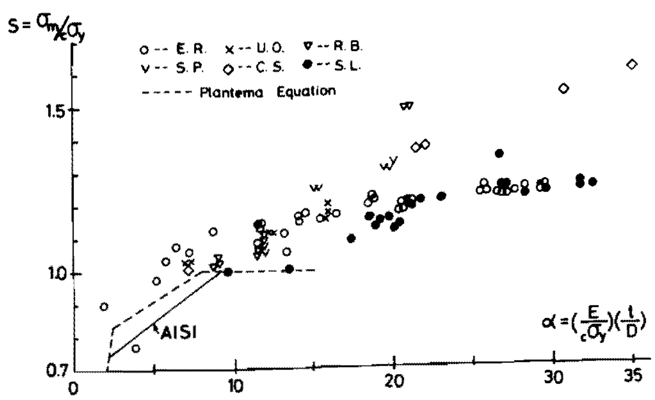


Fig. 6 Local Buckling Strength of Circular Tubes with Different Manufacturing Processes (Kato [18])

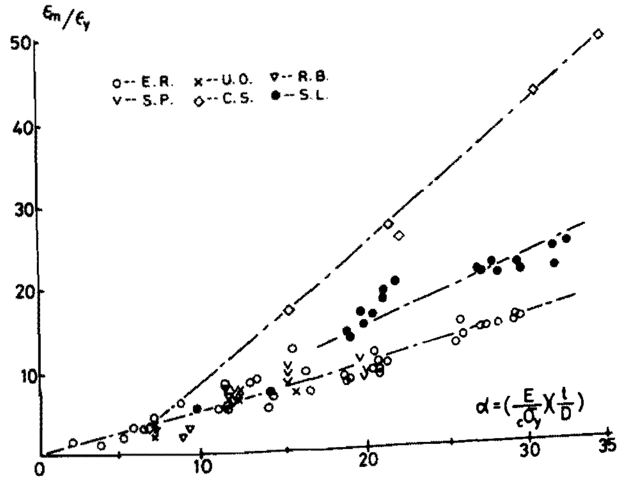


Fig. 7 Difarmation Capacity of Circular Tubes with Different Manufacturing Processes (Kato [18])

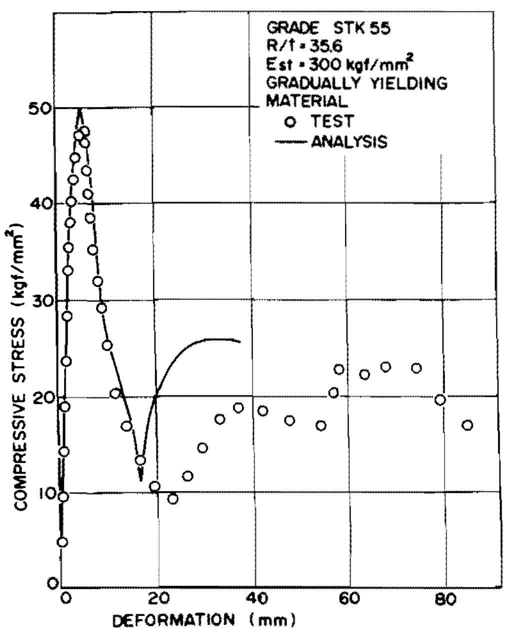


Fig. 8 Large Deformation Analysis of Stub-Column Compared with Test Results (Suzuki et al. [19])

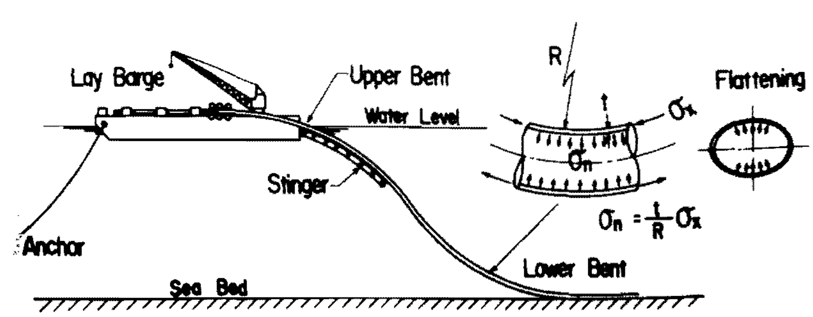


Fig. 9 Offshore Pipeline under Laying Operation

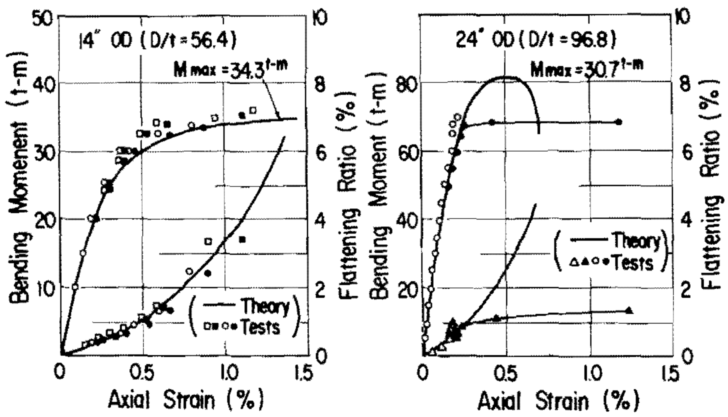


Fig. 10 Behavior of Pipeline under Bending (Kimura, et al. [21])

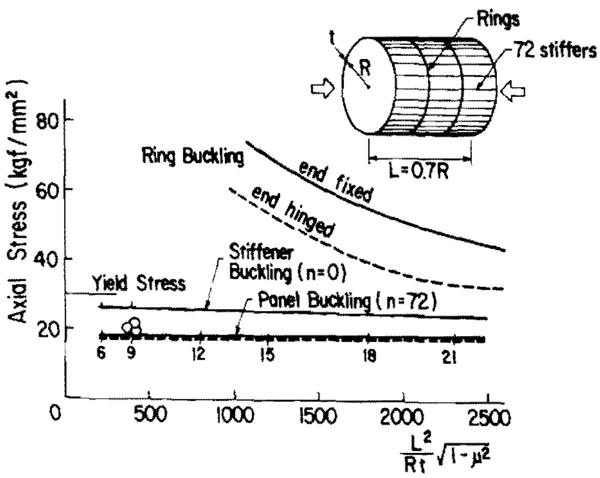


Fig. 11 Stability of Stiffened Circular Shell (Fujita, et al. [22])

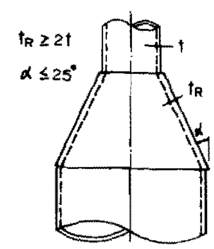


Fig. 12 Details of Tube Reducer (Mitsui [9])

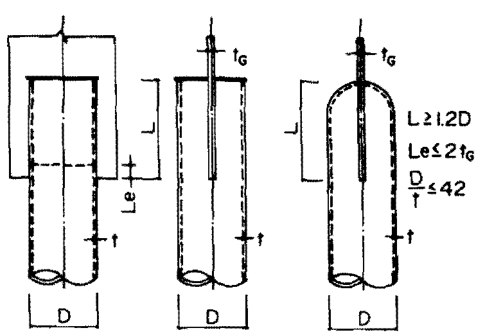


Fig. 13 Details of Tube-to-Through-Gusset Plate Joints (Kurobane [23, 24])
The allowable tensile or compressive force to the joints is equal to 85% of the gross sectional strength of tubular members.

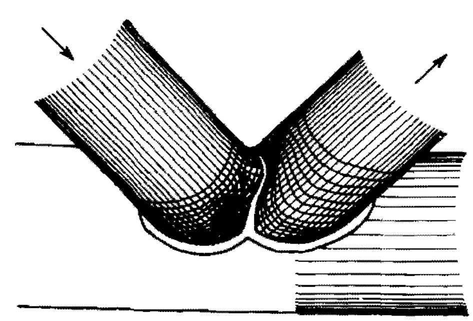


Fig. 14 Failure Mode of K - Joint with Instable Brace Walls

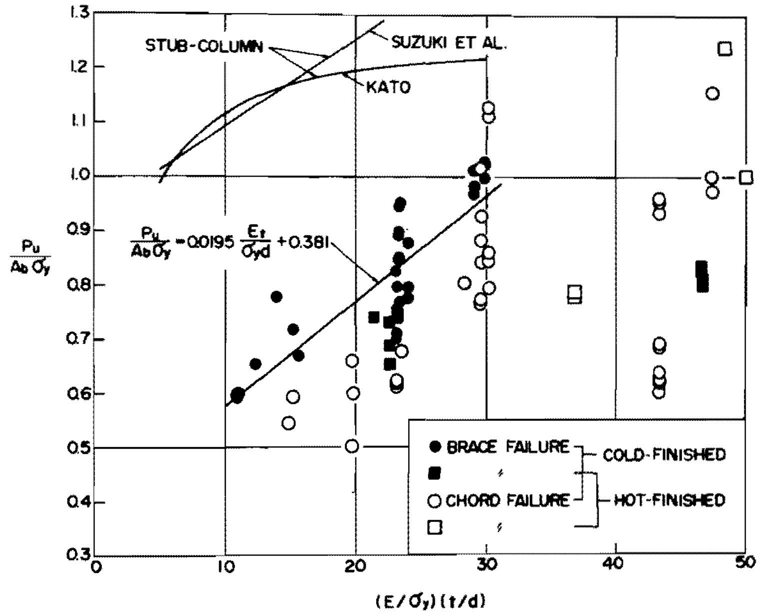


Fig. 15 Ultimate Strength of Compression Brace in Overlapped K - Joints

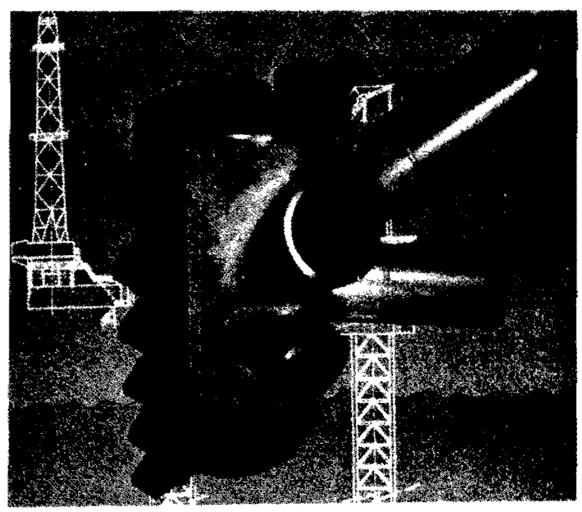


Fig. 16 Precast Leg Node for Jack-up Rig (Ohba, et al. [31])

Fig. 17 Implosion of Liner Tube of Pipeline (Yoshida, et al. [33])

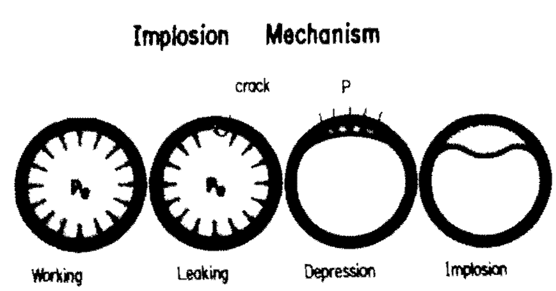


Fig. 18 Mechanism of Implosion (Yoshida, et al. [33])

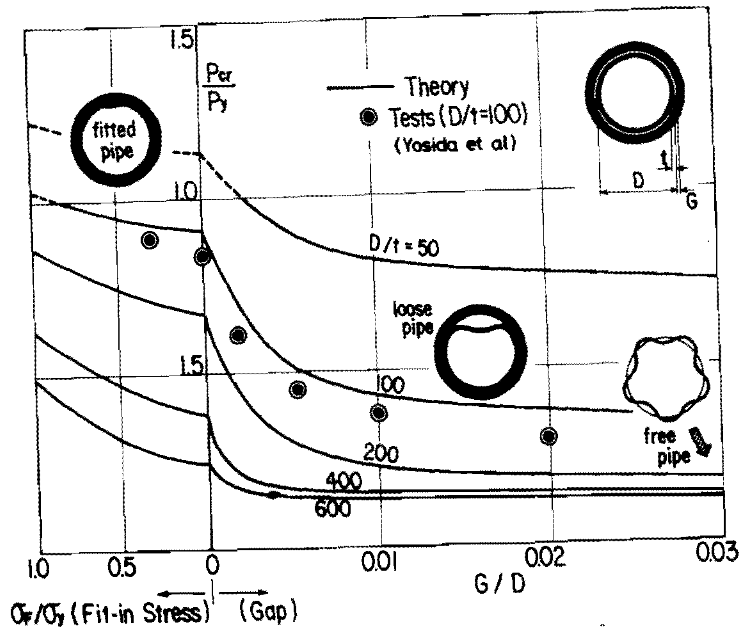


Fig. 19 Buckling Strength of Liner Tube (Yamamoto, et al. (32))

DISCUSSION

S. Sridharan (Washington University): The panel buckling tests you report appear to give good agreement between test and theory. How can this be when such members are supposedly highly imperfection sensitive?

Atsuta: We conducted four series of tests, and I have selected the best results. Not all of the experimental values are in good agreement.

S. Sridharan: The lower bound for panel buckling strength can be taken the same as that for a flat plate. See Koiter (1944).

C. Miller (Chicago Bridge and Iron Company): Could you describe the models used in the ring and stringer stiffened tests under axial load? Were the specimens rolled and welded?

Atsuta: The specimens had a 1 in. diameter and were rolled and welded.

SHELL BUCKLING RESEARCH IN ISRAEL AND ITS APPLICATION TO OFFSHORE STRUCTURES

J. Singer, Israel Institute of Technology

ABSTRACT

A summary report on shell buckling research in Israel with applications to offshore structures is presented. Some earlier studies on conical and cylindrical shells are briefly reviewed, but the emphasis is on the influence of boundary conditions and imperfections on closely stiffened shells, and on vibration correlation methods. Panel buckling, inelastic buckling, and dynamic buckling are also reviewed.

LIST OF SYMBOLS

A_1, A_2	Cross sectional area of stringer and ring, respectively
b_1, b_2	Distance between centers of stringers and rings, respectively.
E	Young's modulus of shell and stiffeners
e_1, e_2 $\bar{e}$	Stringer or ring eccentricity (outside -ve) eccentricity of loading (distance from shell middle surface to point of application of load (outside +ve)
f	frequency
f_{SS4L}	frequency predicted by VIBUL for SS4 B. C.'s
$f_{SS4+\bar{e}}$	frequency predicted by BOSOR 4 in presence of load eccentricity.
h	thickness of shell
k_1, k_4	nondimensional axial and rotational elastic restraints, respectively.
L	length of shell
$M_x, M_\phi, M_{x\phi}, M_{\phi x}, N_x, N_\phi, N_{x\phi}$	moment and membrane force resultants
m	number of longitudinal half waves
n	circumferential wave number
P_{cr}	calculated buckling load

P_{exp}	experimental buckling load
P_{sp}	calculated axial buckling load for shell with axial or rotational restraint.
P_{SS3}, P_{SS4}, P_{C4}	calculated axial buckling loads for shell with SS3, SS4, C4 boundary conditions, respectively
$P_{SS3\ imp}$	predicted buckling load for an imperfect shell
P_{extrap}	buckling load predicted directly from vibration tests
q	frequency exponent for direct prediction method
R	radius of middle surface of cylindrical shell
R_1, R_2	radius of small and large end of cone
u, v, w	nondimensional displacements (displacements divided by R)
Z	$(1-\nu^2)^{\frac{1}{2}}/2(L/R)^2(R/h)$ Batdorf shell parameter
α	cone angle
ν	Poisson's ratio
ρ_{extrap}	P_{exp}/P_{extrap}
ρ_{sp}	P_{exp}/P_{sp}
ρ_{th}	$P_{SS3\ imp}/P_{SS3}$
ρ_{ave}	average radius of curvature of cone $(R_1+R_2)/\cos\alpha$
ψ	taper ratio of cone $1-(R_1/R_2)$

NOTATION FOR BOUNDARY CONDITIONS

$$\begin{array}{ll}
SS1 & : w = M_x = N_x = N_{x\phi} = 0 \\
SS2 & : w = M_x = u = N_{x\phi} = 0 \\
SS3 & : w = M_x = N_x = v = 0 \\
SS4 & : w = M_x = u = v = 0 \\
C1 & : w = W_{,x} = N_x = N_{x\phi} = 0 \\
C2 & : w = W_{,x} = u = N_{x\phi} = 0 \\
C3 & : w = W_{,x} = N_x = v = 0 \\
C4 & : w = W_{,x} = u = v = 0
\end{array}$$

1. INTRODUCTION

Shell buckling research in Israel began about twenty five years ago. Originally it was entirely aerospace oriented. Its center was therefore the Technion Aircraft Structures Laboratory, and its results were primarily applied in the U.S., European, and Israeli aerospace industries. Some nonaeronautical application had already appeared in the mid-sixties, but only in the last decade has research interest seriously broadened to include civil and marine engineering structures, as reported in my survey lecture at BOSS 76 [1].

This summary report will briefly review the earlier studies on conical and cylindrical shells, and the extensive theoretical and experimental work on closely stiffened and orthotropic shells. But the emphasis will be on the more recent studies on the influence of boundary conditions, and in particular on the methods developed for better definition of these effects, such as vibration correlation techniques and imperfection measurements. Load interaction studies, which are of importance in offshore structures, panel buckling, and inelastic buckling will also be reviewed in more detail, and the report will conclude with an assessment of the potential applications of the results to the design of future offshore structures. In order to keep the list of references to a reasonable length, only some of the references have been cited, the choice being of those most likely to relate to interest in offshore structures. Further references can be found in those listed.

2. CONICAL SHELLS

The early studies focused on conical shells. Practical methods of analysis for isotropic conical shells under different loading conditions were developed and verified by extensive experimental studies, which also included combined loading (see for example [2-8]). These theoretical and experimental investigations were then extended to orthotropic and stiffened conical shells [9-12].

The results of these and other studies cited in [2-12] showed that the method of analysis based on the solution of Donnell type stability equations in the presence of slightly relaxed in-plane boundary conditions (with verification that the effect of the implied elastic restraints is negligible) is as satisfactory for conical shells as similar linear theory Donnell solutions are for cylindrical shells. With properly chosen cone parameters: the taper ratio $\psi = 1 - (R_1/R_2)$ and the average radius of curvature of the cone $\rho_{av} = (R_1 + R_2/\cos\alpha$, where α is the cone angle and R_1 and R_2 the small and large end radii respectively, the critical pressures, axial loads and torques of the conical shells can also be conveniently correlated with those of cylindrical shells. The same also applies to theoretical as well as empirical interaction curves in the case of combined loading and to orthotropic and stiffened conical shells.

3. STIFFENED AND ORTHROTROPIC CYLINDRICAL SHELLS

The orthotropic theory for conical shells [10,11] was also applied to analysing the general instability of closely stiffened conical shells [9, 13]. A more accurate, though still simple "smeared stiffener" theory, in which the stiffeners are "smeared" or distributed over the entire shell, in a manner that takes into account the eccentricity of the stiffeners, was then derived, first for conical shells [14, 15], and then for cylindrical shells [16-19].

The analysis is presented in detail in [18] for the case of axial compression and in [16, 17] for hydrostatic pressure, and is outlined in [19, 20]. Linear stability equations are employed (usually Donnell theory, though other theories have also been used, for example a Flügge-type theory [21]) in which the "smeared" rings and stringers are introduced through force and moment expressions representing them as a cut layer. For example, internal rings are replaced by a layer of many parallel rings covering the whole inside of the shell, touching each other but not connected. The main assumptions implied by the model are given in [16, 18, and 20]. This model for smeared stiffeners has also been applied to nonlinear theories and employed in multi-purpose computer codes. Smeared stiffener theory demonstrated clearly the important influence of the eccentricity of the stiffeners, (whether they are on the outside or inside of the shell) on general instability, which was a central theme of shell buckling research in the sixties, and was also studied extensively at the Technion [17-20]. This eccentricity effect was shown to depend both on stiffener and shell geometry, as well as on the type of loading, and the inversion of the eccentricity effect at a certain value of the shell geometry parameter Z was emphasized.

Since smeared stiffener theory is valid only if the discreteness of the stiffeners can be neglected, discreteness effects have been investigated by a linear "discrete" stiffener theory, in which the stiffeners are considered as linear discontinuities represented by the Dirac delta function, instead of being smeared, and have been shown to be usually negligible for stringers, but significant for rings in shells with large Z subjected to hydrostatic pressure [22, 23]. Conclusive experimental evidence is required for acceptance of results based on linear theory, and therefore extensive tests, summarized briefly in Section 5, have been carried out, which confirm the applicability of linear theory as a first approximation for closely stiffened shells, and have yielded bounds for this applicability (see [1, 20, 24-26]).

The buckling of laminated orthotropic cylindrical shells has also been studied at Technion [28-31]. Buckling under axial compression was analysed by an extended Reissner-type theory for axisymmetric deformations [28]. The effects of heterogeneity and orthotropy were studied, and the dependence of the buckling load on shell lamination and material was shown. The buckling load was also derived by the kinetic method [30], and the effects of layup and fiber direction were demonstrated for double and triple-layered shells. A more general method of solution, based on a complex finite Fourier transform, was then developed for stability and vibration analysis of compressed aeolotropic composite cylindrical shells, according to Love-type theory [31]. The critical buckling loads of a variety of single and multilayered clamped and simple supported shells were computed with this method, employing the kinetic stability criterion. For single-layered shells an optimum winding angle was found, for which a higher frequency response and critical buckling load is attainable, and for a bilayer shell of a certain fiber-reinforced material, in which each layer is wound at a different angle, a factor of as much as eight was found.

The structural efficiency of stiffened shells has been the subject of many Technion studies [1, 17-19, 24, 25, 32] which amplify the conclusions of earlier optimization studies carried out by the US aerospace industry in the mid-sixties, emphasizing the superiority of closely-stiffened shells over equivalent-weight isotropic ones. Linear smeared stiffener theory was employed in structural efficiency studies for different types of single loadings (axial compression and external pressure) [19], as well as for shells with non-uniform stiffeners [19, 32]. Non-uniform rings or stringers which were shown [32] to yield weight savings of 10-25%, with much higher gains in structural efficiency, for both external pressure and axial compression loadings, are a promising direction which should be further explored in welded offshore shells, where non-uniform stiffeners may be feasible.

The structural efficiency studies were also related to test-results for stiffened and unstiffened shells, and indeed the significantly higher knockdown factors for closely stiffened shells are one of the main reasons for their high efficiency (see [1, 20, 24, 25]). The many stiffened shells tested at the Technion confirm the expected structural efficiencies above unity, even for some steel shells that exhibited early inelastic effects [20, 24, 25].

Some buckling, postbuckling, and optimization studies have also been carried out for geometrically imperfect stiffened cylindrical shells, employing numerical soultions of Donnell type nonlinear equations [34-39]. For the postbuckling behavior a special numerical procedure was used [39]. These nonlinear analyses employ assumed imperfection shapes, whereas other nonlinear studies using measured imperfections are discussed in Section 7.

4. BOUNDARY CONDITIONS AND LOADING EFFECTS

Initial imperfections have generally been accepted as the main reason for the large discrepancies between theoretical predictions and experimental buckling loads of thin shells. For stiffened shells, and in particular closely stiffened shells (for which local panel buckling is rarely critical), the effect of geometric imperfections is less pronounced. Hence the reduction in predicted buckling loads and scatter of test results is less severe, provided the boundary conditions are adequately accounted for. However, the influence of boundary conditions, and in particular that of in-plane boundary conditions, becomes predominant, and this fact has motivated extensive studies of these boundary effects [24, 40-42].

Some work on the influence of in-plane boundary conditions of isotropic shells was already initiated in the beginning of the sixties (see for example [43]) before the more spectacular effects were discovered by Ohira, Hoff, Almroth, Sobel and others. Later, the analyses for isotropic conical shells were refined to include the effect of in-plane boundary conditions, for example [44], but the significant results were those obtained for stiffened shells [40-42]. Their salient features are: In ring-stiffened shells under axial compression the boundary condition effects are similar to those in isotropic shells [41]. The same also applies for external pressure, except in certain ranges of ring and shell geometries. In stringer-stiffned cylindrical shells, however, the effects differ appreciably from that of unstiffened ones, and the in-plane boundary conditions become predominant (see [20, 40, 42]). Under axial compression, for example, axial restraint ($u=0$, SS2 or SS4 boundary conditions) may raise the buckling load by 50% or more, if the shell is long. It should be noted that these effects depend on shell and stiffener geometry, and therefore good definition of the actual boundary conditions is important. For laminated shells the influence of in-plane boundary conditions is also strongly dependent on the shell heterogeneity and orthotropy [29].

The effect of nonlinear prebuckling deformations has also been studied, using the BOSOR 4 program. For ring-stiffened shells the effect is similar to, and usually smaller than for isotropic shells [40], whereas for stringer-stiffened ones the effect is relatively small, except for short shells (say $Z < 300$), see [25, 40]), for which large increases in buckling loads are predicted (with BOSOR 4, verified by computations with another multipurpose program SRA).

It should be pointed out that for stiffened cylindrical shells the boundary conditions have similar effects on the lower natural frequencies of vibrations whose shapes resemble the buckling modes (see for example [1, 21, 25, 40]). Correlation between vibration and buckling was therefore studied, and yielded a nondestructive experimental tool for the definition of the boundary conditions, - the vibration correlation technique to be discussed in Section 6.

In stringer-stiffened shells, the eccentricity of loading (the radial distance between the line of axial load application and the shell midskin) is another very important boundary effect which was investigated at some research centers in the late sixties. At Technion parametric studies and many tests were carried out [45], which showed that reductions in buckling load of up to about 50% can occur in practical configurations, and also emphasized the sensitivity of the effect to the end joint details. The importance of load eccentricity on vibration correlation warranted additional studies, which are discussed in Section 6.

Nonuniform loading is another loading effect that was subject to serious study at Technion [46-49]. Nonuniformity in axial loading of isotropic cylindrical shells, which occurs in many pratical applications (such as overall nonuniformity due to applied bending moments, or local nonuniformities due to local stiffening, or the presence of large openings, or due to partial loading) was studied within the scope of linear stability analysis and simply supported boundary conditions. It was rigorously proven [46] that, as $(h/R) \rightarrow 0$, the buckling load approaches that of a cylindrical shell subjected to a uniform load which is equal to the maximum intensity of the nonuniform loading. A parametric numerical study was made in [48] to establish the effects of finite thickness. Based on these results and on further analytical studies, a more complete solution for the buckling of shells of finite (h/R) subjected to general nonuniform axial loads was presented in [49]. The results are strictly valid only where a membrane prebuckling state is a reasonable approximation of the actual state. In long shells a redistribution takes place, and the results of [46-49] then represent a conservative estimate of the true buckling loads.

A numerical solution of classical buckling for shells of revolution subjected to nonuniform loads, both meridionally and circumferentially nonuniform, was also developed [50].

5. BUCKLING TESTS

In the early stages of shell research in Israel it had already been realized that for verification and confidence in the results, theoretical studies must be accompanied by careful experiments. Hence many early studies were experimental (see for example [3-5, 7-9, 12, 13]). In investigations on stiffened shells that followed, buckling tests moved into the foreground [20, 21, 24-27], and with the emphasis on boundary conditions, vibration correlation, and imperfections in the last decade, experiments have become even more important.

Fabrication of specimens has always been one of the main problems in shell experimentation, and also at the Technion Aircraft Structures Laboratory much effort has been devoted to this. Of particular interest are the integrally machined ring-and-stringer stiffened cylindrical shells. The alloy steel (AISI 4130) and aluminum alloy (7075-T6) specimens are machined from relatively thick-walled tubes in stages. In the final stages the shells are mounted on special mandrels, steel specimens on a liquid air cooled aluminum mandrel, and aluminum specimens on an oil heated steel mandrel (see [24, 26, 27], or for more details [51, 52]). These techniques, combined with care in machining have resulted in precise specimens, the worst deviation in wall thickness for ring-stiffened shells being 5% and usually less than 2.5%, and for stringer-stiffened shells 10% at worst, and usually less than 5%. For steel ring-stiffened conical shells [12] another interesting process was employed: hydrospinning of a blank onto an accurate mandrel, grinding of the internal surface, and then machining the outside rings on another mandrel, with special stress relieving between stages.

In two decades of experimental work considerable efforts have also been devoted to improvements in measurement techniques for precision, to facilitate better understanding of buckling behavior, to precise loading conditions, and better definition of boundary conditions (see for example [3-5, 7, 8, 12, 25-27, 52]). In recent years two special topics, vibration correlation and imperfection measurements, have received particular attention, and they are discussed in the sections that follow.

The major experimental effort has been devoted to testing different series of integrally machined stiffened cylindrical shells. To date 51 ring-stiffened (18 steel and 33 aluminum alloy) and 197 stringer-stiffened shells (88 steel and 117 aluminum alloy) have been tested, mostly under axial compression, and the remainder under external pressure and combinations of axial compression and external pressure [20, 21, 24-27, 45, 51-60]. Each series differed in either boundary or loading conditions or both. In addition a series of 20 annealed aluminum alloy ring-stiffened shells was tested for plastic buckling [61], and a test series of larger spot-welded stringer stiffened shells has been initiated [62].

One of the purposes of these extensive tests programs was examination of the adequacy of linear theories. Since initial imperfections and other degrading factors are most pronounced for axial compression, the correlation of theory with experiments deals primarily with this loading case, which has also been extensively studied at other research centers. The applicability of theory is conveniently expressed by the ratio of the experimental buckling load P_{exp} to that predicted by linear theory P_{cr}, usually called "knock-down factor" $\rho = (P_{exp}/P_{cr})$, though the author prefers the term "linearity" since, for stiffened shells as opposed to unstiffened ones, the ratio is usually closer to unity.

The correlation of ring-stiffened shells is presented in Fig. 1 (from [1]) versus the prime geometric parameter, the area ratio (A_2/b_2h). On the results of steel and aluminum alloy shells, covering a wide range of shell geometries $(R/h) = 400\text{-}710$, tested at the Technion and at other centers in the USA (Caltech [63], and Almroth), Hysol epoxy shells of $(R/h) = 170\text{-}210$, with prescribed periodic axisymmetric imperfections (a shape known to have a pronounced degrading effect) tested by Tennyson in Canada, are superimposed. Fig. 1 (from [1]) shows that for integrally ring-stiffened cylindrical shells under axial compression there is a $\pm$ 10 - 15% scatter; about $\rho = 0.95$ for $(A_2/b_2h) > 0.3$ or $\rho = 0.8$ applies roughly. For weaker rings one could alternatively state that $\rho = 0.6$ is a lower bound for $(A_2b_2h_2) > 0.15$.

For stringer-stiffened shells boundary conditions are very important, and correlation is therefore sometimes presented separately for simply supported and clamped shells. In Fig. 2 (from [1]), however, the results for the steel and aluminum alloy shells, covering shell geometries $(R/h) = 410\text{-}760$ and $(L/R) = 0.2\text{-}2.9$ are summarized together, though the nearly clamped shells are related to predictions for complete clamping ρ_{C4}. Again, most of the results are from Technion tests, some from tests in the USA (Milligan, Katz, Card, Caltech [63]) and results of 4 epoxy shells with $(R/h) = 130\text{-}160$ (Tennyson, Canada) are superimposed on them.

The scatter in Fig. 2 for stringer-stiffened shells is larger than that in the case of ring-stiffened ones. The trend is less clearly defined, but the stiffener area ratio is again a primary parameter. The shell geometry parameter Z is, however, also a governing factor for stringer stiffened shells with a clear trend towards larger ρ for $Z > 1000$ (see [27]). The "knock-down factor" ρ of the shells in Fig. 2 - excepting some of those correlated to full clamping, and a few shells where early local buckling was observed (Katz) - is above 0.65 even for weak stringers, and for $(A_1/b_1h) > 0.5, \rho > 0.7$. The large scatter of $\pm$ 20-30% in Fig. 2 is due partly to incomplete definition of the boundary conditions, since these, and in particular the in-plane boundary conditions, considerably affect the buckling loads of stringer-stiffened shells, as has already been mentioned.

Apart from the buckling load ratios ρ, which verify the adequacy of linear theory as a first approximation, the experimentally observed buckling modes are a significant criterion for applicability of the theory. As can be seen in Fig. 3 (from [64]), which shows the postbuckling pattern for three AB shells, the observed postbuckling and buckling patterns of stringer-stiffened cylindrical shells very closely resemble those predicted by linear theory, contrary to the observations in isotropic shells. This supports the above conclusions on the adequacy of linear theory, and also facilitates correlation with vibrations, discussed in the next section.

A preliminary comparison of results of some riveted and welded closely stiffened cylindrical shells with those obtained on integrally stiffened shells carried out in 1976 [1] indicated good agreement in cases of close stiffening, for which general instability dominates. More recent tests confirm this good agreement, which indicates that the results of the extensive studies on integrally stiffened shells are relevant to offshore structures.

6. VIBRATION CORRELATION METHODS

The similarities in the influence of the boundary conditions and of various other parameters on the buckling of stiffened shells and their free vibrations have motivated extensive studies at Technion [1, 25, 40, 54-60, 64, 65] attempting to correlate these two phenomena, and draw from the nondestructive vibrations conclusions about the destructive buckling behavior of these shells. These vibration correlation methods can be divided into two groups: a) those for determination of boundary conditions, and b) those for direct determination of buckling loads.

The vibration correlation technique for determination of boundary conditions consists essentially of an experimental determination of the lower natural frequencies for a loaded shell, and evaluation of equivalent elastic restraints representing the actual boundary conditions. It is based on the similarity between the strong influence of axial and rotational restraints on free vibrations of stiffened shells, in particular for the lower natural frequencies whose mode shapes resemble the buckling modes [21, 25, 55], and that observed for buckling [18, 27, 42]. The method was first developed for axial compression loading (see [1, 25, 40, 55, 56, 64]) and then extended to external pressure and combined loading (see [58, 60, 65]). It is briefly reviewed here for a typical aluminum alloy stringer-stiffened cylindrical shell, loaded in axial compression RO-32 [53, 64] similar to the ones shown in Fig. 3, except that it is supported on laboratory type boundary conditions, a "V" groove approximating simple supports. (The main geometrical properties of the shell are: $(R/h) = 467$, $(L/R) = 1.0$, $(A_1/b_1h) = 0.78$). It is nominally simply supported, and the real boundary conditions can be represented by an axial elastic restraint k_1 between SS3 and SS4. The influence of such elastic axial restraints on buckling and vibrations can be calculated with VIBUL [55], a special purpose program, employing linear smeared stiffener theory [16, 17] and Flügge equations. Figure 4 (reproduced from [64]) shows the variation of buckling load P_{cr} between SS3 and SS4 for this shell. The variation between SS3 and SS4 is obtained by the axial spring k_1(the other B.C.'s are $w=0$, $M_x=0$, $v=0$), the stiffness of which is zero for SS3 and infinity for SS4 B.C.'s. In Fig. 5 the influence of elastic axial restraints on the frequency squared of the vibration mode m=1, n=12, at an axial load of 1600 kg is shown in the same manner as the buckling load in Fig. 4. This mode of vibration was chosen because it represents the buckling mode in most of the range of the springs. The shape of the two curves is indeed similar. Hence by measuring the natural frequency at a relatively low load (1600 kg here is about 1/3 - 1/4 of the expected buckling load) the effective boundary conditions can be determined. For clamped boundary conditions a similar procedure can be employed by introduction of a torsional spring k_4 between SS4 and C4 (see for example [25, 40, 53, 55]).

The frequencies squared for shell RO-32 predicted with the VIBUL program are plotted versus the axial load P in Fig. 6 (from [64]). The experimental results are also plotted in the figure. They were obtained in a test set up (see for example Fig. 6 of [40]), which resembles the more recent one shown in Fig. 7. The test set up and procedure are described in detail in [21, 40, 56, or 64]. The experimental technique consists of vibrating the loaded shell by an exciter, followed by detection of resonance frequencies and mode shapes by an outside scanning microphone.

The experimental values in Fig. 6 show a clear trend to a certain value of k_1. Examination of similar plots for other vibration modes in the vicinity m=1, n=7-11 show trends to practically the same value of k_1. This insensitivity to the exact mode of vibration, confirmed in the many tests, is important for the reliability of the method, as in practice one can only hope to excite vibration modes close to that corresponding to the buckling mode, and not always exactly. The study of Fig. 6 and similar curves yielded for this shell k_1=6 (see also Fig. 5), after which the buckling load for this k_1 could be predicted with VIBUL (see Fig. 4).

The method was applied to shells of different Technion test series for axial compression loading [21, 53, 55-58]. Figure 4 (reproduced from [65]) presents the results for 31 shells, and shows a significant reduction in scatter as a result of the experimental determination of the boundary conditions.

Since in the vibration tests an actual imperfect shell is measured, the initial imperfections are indirectly included or "lumped" in the correlation. Theoretical studies of the influence of imperfections on the vibrations of unstiffened cylindrical shells [66,67], and of stiffened shells [68], have shown that imperfections have a strong influence on the frequency of vibrations, similar to that on the buckling of cylindrical shells, not only at high compressive loads, but also at zero axial loads. In fact, the shape of plots of the square of the frequency versus the imperfections amplitude μ resemble those of critical stress versus μ, though for vibrations the reduction is less severe. The extensive parametric studies carried out, and their comparison with similar buckling studies added confidence in the soundness of vibration correlation techniques. It is clear, however, that as the effect of the same imperfections on the vibrations is smaller than that on the buckling load, only part of the effect is taken into account by the vibration correlation technique. Further studies, combining direct imperfection scanning techniques and vibration correlation are discussed in Section 7.

The vibration correlation method was also applied to realistic boundary conditions, to shells with end supports simulating joints employed in actual engineering construction [56]. Such joints may also have load eccentricity, which may not be well defined a priori, and may depend on the tolerances and behavoir of the joints under load. Now whereas the "lumping" of the influence of boundary conditions and imperfections inherent in the vibration correlation technique give no reason for concern, as their behavior in vibrations and buckling is similar, the effects of load eccentricity differ for vibrations and buckling, and can therefore obscure the correlation [54, 57].

A thorough study of the data of AB shells [56] eventually revealed a salient property which distinguishes vibrations **in the presence** of significant load eccentricities. This important property is the large increase in the frequency ratio squared $(f/f_{SS4L})^2$ with the number of circumferential waves n of the vibration pattern, which does not occur in the absence of load eccentricity. Figure 9 (reproduced from [56]) shows a plot of this ratio versus n for a typical stiffened shell AB-5 on the practical missile joint type boundary conditions discussed in [56] (see Fig. 10f). The frequency ratio here is that of the experimentally observed frequency to that predicted by linear theory VIBUL for SS4 boundary conditions. If one now plots the corresponding theoretical frequency ratio squared for some likely values of load eccentricity, where $f_{SS4+\bar{e}}$ is the frequency computed with BOSOR 4 (which considers nonlinear prebuckling deformations) for SS4 boundary conditions in the presence of load eccentricity $\bar{e}$, the same property becomes evident. One can easily find a load eccentricity $(\bar{e}/h)$ which has the same slope as the experimental one. Here the apparent load eccentricity is $(\bar{e}/h) = 2.0$.

This frequency slope property presents a tool for nondestructive identification of significant unknown load eccentricity, and once the load eccentricity has been identified a modified vibration correlation method, it was verified on shells of prescribed load eccentricity [54, 57].

The vibration correlation method for definition of boundary conditions has been applied to a variety of boundary conditions shown in Fig. 10. Laboratory type edges: "simple" supports (a) [21. 53, 54, 57] or (e) [60], clamped edges (d) [2], 53, 55, 58, 60], or edges with prescribed load eccentricity as already mentioned [54, 57]; and practical boundary conditions which simulate actual joints, as in the AB shells (f) [56] and a recently initiated test series - the DK shells (g), designed for variation of boundary conditions during the test. A series of tests on larger shells of welded and riveted construction under axial compression (see Fig. 12) has also been initiated in cooperation with the Technical University of Aachen, in order to increase confidence in the method, and transform it into an industrial tool [62].

Vibration correlation can also be employed for direct prediction of buckling loads. At Technion, the same experimental technique used to define boundary conditions was extended to direct prediction of buckling loads. The method is essentially curve fitting to the experimental points of the frequency squared versus axial load, but using only those points below 50-60% of the buckling load, to make the procedure truly nondestructive. The curve fitting is actually carried out with respect to a straight line, representing the experimental points to a certain exponent q, obtained from previous vibration and buckling tests on similar shells [59]. This exponent was related to the dominant geometric properties of stringer-stiffened cylindrical shells, the Batdorf shell parameter Z and the stringer area ratio (A_1/b_1h), and a functional relation with two empirical constants was found [59]. Being empirical, this relation is obviously limited to the range of geometries tested, and requires many more tests for wider applicability.

In Fig. 11 (from [59]) buckling load ratios for direct prediction ρ_{extrap} of all the shells studied are summarized and compared with the theoretical predictions for effective boundary conditions found with the vibration correlation technique ρ_{sp}. The scatter of ρ_{extrap} is found to be about $^2/3$ that of ρ_{sp}.

7. IMPERFECTION MEASUREMENTS AND BUCKLING PREDICTIONS

Though stiffened shells are significantly less imperfection sensitive than unstiffened ones (see for example [20, 25]), the imperfections are among the important parameters influencing their buckling loads. In the Technion Aircraft Structures Laboratory it has therefore become general practice to measure the initial imperfections of every shell tested. The shells tested recently in vibration correlation studies have therefore been measured in a special test system for scanning imperfections and their growth, Fig. 12 [69-73]. The records are stored in the Haifa Branch of the International Imperfection Data Bank [71, 73], and are later also transmitted to the Delft Branch. The designs of the new test rigs for the larger shells and curved panels at the Technion therefore also include systems for measurement and recording of initial imperfections. A new multipurpose scanning and measurement system has recently been developed [62, 74], in which the same noncontact probe is used to measure both vibrations and geometric imperfections of cylindrical shells. Figure 13 (from [62]) shows the complete system (a) without the shell, showing the scanning system and the noncontact probe and (b) with a spot-welded shell AAC-1 in position for imperfection and vibration measurements.

Once the initial imperfection shapes are known and the boundary conditions well defined, analytical tools and computer codes are available for calculation of the buckling load, though improvements are still necessary. In [71] and [73] the buckling loads for recently tested stiffened shells of the AB, KR, SN, RS, and DUD series were computed from the reduced imperfection data with the MIUTAM program (developed by Arbocz and Babcock at Caltech and Delft), which is based on the Multimode Analysis. The resulting theoretical buckling load ratios

$$\rho_{th} = \frac{P_{SS3\ imp}}{P_{SS3}}$$

where $P_{SS3\ imp}$ is the buckling load computed from the measured imperfections by the Multimode Analysis and P_{SS3} is the classical linear theory buckling load for SS3 boundary conditions, are compared with the experimental buckling load ratios

$$\rho_{sp} = \frac{P_{exp}}{P_{sp}}$$

where P_{exp} is the experimental buckling and P_{sp} is the linear theory prediction for boundary conditions with equivalent elastic restraints, obtained experimentally by the vibration correlation method. In Table 1 (from [65]) this comparison is presented for 7 of the shells.

The comparison seems to indicate that boundary conditions are indeed fairly well defined by the vibration correlation technique. ρ_{th}, which evaluates the influence of initial imperfections only, and compares quite well with the knockdown factor ρ_{sp} referred to the experimentally determined boundary conditions, which represents the actual degradation due to imperfections if the B.C.'s are properly defined.

Table 1: Comparison between buckling loads predicted from measured initial imperfections and experimental results.

Shell	Predicted ρ_{th}	Experimental ρ_{sp}	
AB-6	0.72	0.75	-0.03
KR-1	0.84	0.68	0.16
SN-1	0.95	0.91	0.04
SN-4	0.88	0.82	0.06
DUD-2	0.86	0.63	0.23
DUD-3	0.70	0.58	0.12
RS-36	0.91	0.76	0.15

For shells KR-1, DUD-2 and DUD-3, which are clamped, the comparison is only fair. The main reason for the discrepancy between experiment and theory is probably that the initial imperfections were measured before being fixed in their final boundary conditions [65, 70, 71]. This deficiency is removed in the new unified system [74].

From the imperfection measurements carried out at the Technion on stiffened shells, some characteristic patterns could be distinguished [71, 73]. These characteristic patterns, or "characteristics", could be correlated with the manufacturing process, and were also observed in studies of other shells manufactured elsewhere by similar processes. As the data accumulates in the branches of the International Imperfection Data Bank, more general comparisons and conclusions will become possible.

8. COMBINED LOADING

Since combinations of external pressure and axial compression are a very important loading condition for stiffened shells, in particular for offshore applications, renewed interest and considerable research effort have been directed to combined loading. At Technion this motivated the initiation of a program extending the vibration correlation technique to combined loading [1, 58] in order to obtain improved interaction curves. The first test program [58] involved 4 steel shells and 6 aluminum alloy shells, of geometries similar to the RO and AB shells tested under axial compression, and established the applicability and usefulness of the technique as a nondestructive method for improved definition of boundary conditions also for external pressure and combined loading. The results were also used to evaluate theoretical interaction curves, and to obtain improved curves by referring to the effective boundary conditions found with the aid of vibration correlation. In order to obtain better interaction curves, an additional series of tests [60], with nominally clamped integrally stringer-stiffened shells (as in Fig. 10d) DUD-2 and DUD-3 shells, as well as similar shells on nominal simple supports (as in Fig. 10e) DUD-4 to DUD-7, were carried out.

Significant improvement in the interaction curve is obtained by the vibration correlation technique. It should be pointed out that these improved interaction curves are obtained by a series of definitely non-destructive tests on the same shell at different combinations of axial compression and external pressure, taking care not to exceed about half the theoretical (linear theory) buckling load at each load combination. In the more recent tests of DUD shells [60], the use of a single specimen for the entire range of load combinations also for buckling tests, which is desirable (see [75]), but was found difficult to achieve in stiffened shells [58], has been successfully performed. Additional tests of stiffened shells under combined loading are in progress.

9. CYLINDRICAL PANELS

Since cylindrical panels are important structural elements, and as local buckling of stiffened shells is usually governed by panel buckling, the stability of unstiffened and stiffened panels has also been the subject of many studies in Israel. The earlier work [76-78] was primarily concerned with the effect of boundary conditions and nonuniform loading on unstiffened panels, but then interest shifted to stiffened panels [79-81], again mainly to boundary conditions.

One study [80] considered the effects of in-plane boundary conditions along the straight edges on buckling of stiffened panels, using smeared stiffener theory, in order to provide the necessary information for correct simulation of a stiffened cylinder by its representative panel.

This study was motivated by large scale buckling tests on stiffened curved panels of complete cylinders in the interest of economy in specimens, and because of limitation in load capacity of test systems. The study revealed a "sensitivity" to certain in-plane boundary conditions which may, for some types of panels, lead to unexpected non-conservative buckling load predictions. The necessity for precise definition of the boundary conditions was thus emphasized, motivating development of vibration correlation techniques for curved panels.

Similarities in the influence of boundary conditions on buckling and free vibrations in stiffened shells also extend to stiffened panels, and therefore vibration correlation studies appear feasible. For theoretical predictions of vibrations and buckling of stiffened curved panels "smeared-stiffener" theory is again employed, and for correlation studies the first step is again linear theory. The theory used in the VIBUL program for stiffened-cylindrical shells [55] has therefore been extended to curved panels, with different boundary conditions along the straight edges, including elastic restraints; and a computer program, VIBUPAL [81], has been written. Check calculations have been made, and some correlation studies have been performed. Some pilot tests have been carried out, and theoretical studies on the influence of imperfections on panels have been initiated.

10. INELASTIC BUCKLING

As a result of the aerospace bias, the emphasis in shell buckling research in Israel has been on elastic buckling. In the experimental studies, however, inelastic effects had often to be accounted for, even for relatively thin shells (see for example [20, 51]). But though there was considerable research activity in the theory of plasticity, only recently has attention turned to plastic buckling of shells. A test program on plastic buckling of ring-stiffened cylindrical shells, made of annealed 3003-0 aluminum alloy, under axial compression [61] was started a few years ago. Simultaneously, a simple analytical solution based on rate equations, as proposed by Batterman, was developed and correlated with the tests and with computations using the BOSOR 5 program. These studies continue.

Very recently, more extensive theoretical studies have been initiated by Durban. Elastoplastic buckling of circular cylindrical shells and panels, subjected to nonuniform axial loads have been investigated with the framework of small strain plasticity using a Donnell type shell theory. Material behavior is modelled both by J_2 flow theory and J_2 deformation theory. Pure bending loading and "line load" have been considered, leaning somewhat on earlier elastic solutions (as for example [46-47, 77]) and buckling loads are obtained in an almost closed form. The analysis extends over the entire elastoplastic range, including the transition zone, and is continuing vigorously.

11. DYNAMIC BUCKLING AND PROBABILISTIC METHODS

Dynamic buckling of shells has scarcely been considered in Israel. An early study of parametric instability of stiffened cylindrical shells [82] extended the smeared stiffener theory [16-18] from static buckling to dynamic stability of the type extensively studied by Bolotin in the USSR. Later smeared stiffener theory was also employed in the study of dynamic buckling of closely stiffened imperfect shells under axial impact [83], and then to similar shells with assumed statistical distributions of initial imperfections [84]. Very recently some experimental studies of shell buckling under impact have also been initiated in the framework of a broader test program of dynamic buckling of structures under impact loading.

One different approach, stochastic stability analyses, carried out as part of more general reliability studies, has also recently been applied to shell buckling (see for example [85]), and more work in this direction is in progress.

12. POTENTIAL APPLICATIONS TO OFFSHORE STRUCTURES

As pointed out in [1], the experience accumulated in aerospace practice with closely spaced cylindrical shells can be utilized for offshore structures in part of the (R/h) range. For the relatively thin stiffened shell elements, where elastic instability dominates, the correlations with tests on integrally stiffened shells are applicable, and have generally been verified by tests on fabricated and welded closely stiffened shells. Hence the results of the extensive theoretical and experimental studies carried out in Israel in the last two decades can also be widely applied in offshore structures. This is to some extent pertinent to the studies on conical shells, orthotropic shells, and nonuniform loading. The more recent emphasis on boundary conditions and imperfections, and in particular on vibration correlation methods, bring the results from Israel even closer to offshore structures.

The potential of the vibration correlation methods to offshore structures can be evaluated as in my London survey [65].

It should be remembered that the primary requirement for successful application of vibration correlation methods is that the likely buckling mode of the structure is unique, and at least approximately known beforehand. It must also be feasible to vibrate the structure in this mode. On account of optimization, larger sizes, and improved automation of welding processes, the trend in shells for offshore structures is to closely stiffened shells. These are governed by general instability, and satisfy the primary requirement of vibration correlation. Furthermore, this general instability failure is very sensitive to boundary conditions, and therefore warrants a better definition of those conditions, which is indeed provided by the vibration correlation technique.

In passing, it should again be pointed out that, as demonstrated by many tests of integrally stiffened shells and shells with riveted stiffeners, closely stiffened shells are also less sensitive to the effects of imperfections occurring in practice. Furthermore, on account of their unique buckling mode, closely stiffened shells are much less sensitive to local imperfections than shells with widely spaced stiffeners, where panel buckling resembling that of isotropic shells occurs.

For closely stiffened cylindrical shells in future offshore structures, the vibration correlation technique for definition of boundary has therefore considerable potential, though much work is still required to make it a reliable industrial tool. The joint test program with the RWTH Aachen on larger spotwelded, riveted, and corrugated shells is a step in this direction.

The direct prediction method may also find its place, but, being semi-empirical, it requires extensive testing of shells resembling those used in practice.

Now since the stiffened shells in offshore structures often buckle in the elastic-plastic range, an extension of vibration correlation to that range should be explored. Since, however, the effect of boundary conditions on plastic buckling of shells has not yet been fully determined, its study, which has already commenced, is of importance.

When more experience with vibration correlation methods for improved buckling predictions of offshore shells has been acquired, additional applications will become possible. For example, vibration tests in situ are already in use in some structures for assessment of possible degradation. Excitation in modes that approximate buckling modes may eventually also permit direct evaluation of the resultant reduction in buckling strength.

For shells with widely spaced stiffeners, vibration correlation has smaller potential. Assessment of actual boundary conditions at the straight edges of panels is one example which may develop from the studies that have been initiated.

The measurement of initial imperfections and their analysis is becoming an important tool for the study of the buckling behavior of shells. As the International Imperfection Data Bank develops and contributes new design criteria and recommendations, including those for offshore shells, the Haifa branch will figure prominently in its output.

The other recently initiated shell stability research efforts discussed, such as dynamic buckling, inelastic buckling, and probabilistic method, may also find various applications in offshore structures as their results crystallize.

12. CLOSING REMARKS

In the references, only those papers describing shell buckling research in Israel have been included. Other references, though they relate directly to the topics discussed, and may be even more important, have been omitted, to make the list of references in itself a summary of the shell research in Israel. However, as there has been close co-operation with many groups in other countries, some references have joint authors, and I would like to indicate the affiliations of the non-Israeli authors mentioned:

G. J. Simitses and J. Giri are from Georgia Institute of Technology, Atlanta;
S. C. Batterman is from the University of Pennsylvania, Philadelphia;
C. D. Babcock is from the California Institute of Technology, and J. Arbocz from Delft University of Technology.

Finally, I would like to point out that though research on the buckling of shells in Israel has primarily been carried out at the Technion, Israel Institute of Technology in Haifa, (mostly in Aeronautical Engineering, and recently some in Civil Engineering), there was some activity at the School of Engineering of Tel-Aviv University, and a vigorous shell program has been initiated at Ben-Gurion University of the Negev, Beer-Sheva (see for example [86]), but these activities are not related to offshore structures.

ACKNOWLEDGEMENTS

The support of the DFG in cooperation with the RWTH Aachen for part of this study is gratefully acknowledged. The author also wishes to thank Mrs. M. Seitelbach and Mrs. R. Minkoff for the typing of the manuscript.

REFERENCES

1. Singer, J., "Buckling, Vibrations and Postbuckling of Stiffened Metal Cylindrical Shells", Proc. of BOSS 76 (First International Conference on Behavior of Off-Shore Structures) Norwegian Inst. of Technology, Trondheim, Norway, Aug. 1976, pp. 765-786.

2. Singer, J., "Buckling of Circular Conical Shells under Axisymmetric External Pressure", Jour. of Mechanical Engineering Science, Vol. 3, No. 4, December 1961, pp. 330-339.

3. Singer, J., and Eckstein, A., "Experimental Investigations of the Instability of Conical Shells under External Pressure", Proc. of the 4th Annual Conference on Aviation and Astronautics, Bulletin of the Research Council of Israel, Vol. 11C, April 1962, pp. 97-122.

4. Singer, J., and Eckstein, A., "Recent Experimental Studies of Buckling of Conical Shells under Torsion and External Pressure", Proc. of the 5th Israel Annual Conference on Aviation and Astronautics, February 1963, pp. 135-146.

5. Berkovits, A., and Singer, J., "Buckling of Unstiffened Conical Shells under Combined Torsion and Axial Compression or Tension", Proc. of the 7th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 3, No. 1, February 1965, pp. 15-24.

6. Singer, J., and Baruch, M., "Buckling of Circular Conical Shells under Combined Torsion and External or Internal Pressure", Topics in Applied Mechanics, E. Schwerin Memorial Volume, Elsevier, Amsterdam, Holland, 1965, pp. 65-88.

7. Berkovits, A., Singer, J., and Weller, T., "Buckling of Unstiffened Conical Shells under Combined Loading", Experimental Mechanics, Vol. 7, No. 11, November 1967, pp. 465-467.

8. Singer, J., and Bendavid, D., "Experimental Investigation of Buckling of Electroformed Conical Shells under Hydrostatic Pressure", AIAA Journal, Vol. 6, No. 12, 1968, pp. 2332-2337.

9. Singer, J., "Buckling of Orthotropic and Stiffened Conical Shells", Proc. of NASA Symposium on Instability of Shell Structures, October 1962, NASA TN D-1510, December 1962, pp. 463-479.

10. Singer, J., "Donnell Type Equations for Bending and Buckling of Orthotropic Conical Shells", Jour. of Applied Mechanics, Vol. 30, No. 2, June 1963, pp. 303-305.

11. Singer, J., and Fersht-Scher, R., "Buckling of Orthotropic Conical Shells under External Pressure", The Aeronautical Quarterly, Vol. 15, Part 2, May 1964, pp. 151-168.

12. Weller, T., and Singer, J., "Experimental Studies of Buckling of Ring-Stiffened Conical Shells under Axial Compression", Experimental Mechanics, Vol. 10, No. 11, November 1970, pp. 449-457.

13. Singer, J., "On the Buckling of Unstiffened Orthotropic and Stiffened Conical Shells", presented at the 7th A.F.I.T.A.E. International Aeronautical Congress, Paris, June 1965, pp. 1-22.

14. Baruch, M., and Singer, J., "General Instability of Stiffened Circular Conical Shells under Hydrostatic Pressure", The Aeronautical Quarterly Vol. 26, Part 2, May 1964, pp. 187-204.

15. Baruch, M., Singer, J., and Harari, O., "General Instability of Conical Shells with Non-uniformly Spaced Stiffeners under Hydrostatic Pressure", Proc. of the 7th Israel Annual Conference on Aviation and Astronautics, February 1965, Israel Journal of Technology, Vol. 3, No. 1, pp. 62-71.

16. Baruch, M., and Singer, J., "The Effect of Eccentricity of Stiffeners on the General Instability of Stiffened Cylindrical Shells under Hydrostatic Pressure", Journal of Mechanical Engineering Science (England) Vol. 5, No. 1, March 1963, pp. 23-27.

17. Singer, J., Baruch, M., and Harari, O., "Inversion of the Eccentricity Effect in Stiffened Cylindrical Shells Buckling under External Pressure", Journal of Mechanical Engineering Science (England), Vol. 8, No. 4, December 1966, pp. 363-373.

18. Singer, J., Baruch, M., and Harari, O., "On the Stability of Eccentrically Stiffened Cylindrical Shells under Axial Compression", International Journal of Solids, Vol. 3, No. 4, 1967, pp. 445-470.

19. Singer, J., and Baruch, M., "Recent Studies on Optimization for Elastic Stability of Cylindrical and Conical Shells", Aerospace Proceedings 1966 Proc. of the 5th International Congress of the Aeronautical Sciences, London, September 1966, Macmillan, pp. 751-782.

20. Singer, J., "Buckling of Integrally Stiffened Cylindrical Shells - a Review of Experiment and Theory", Contributions to the Theory of Aircraft Structures, Delft University Press, 1972, pp. 325-357.

21. Rosen, A., and Singer, J., "Vibrations of Axially Loaded Stiffened Cylindrical Shells," Journal of Sound and Vibration, Vol. 34, No. 3, June 1974, pp. 357-378.

22. Singer, J., and Haftka, R., "Buckling of Discretely Ring-Stiffened Cylindrical Shells", Proceedings of the 10th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 5, No. 1-2, February 1968, pp. 125-137.

23. Singer, J., and Haftka, R., "Buckling of Discretely Stringer-Stiffened Cylindrical Shells and Elastically Restrained Panels", AIAA Journal Vol. 13, No. 7, July 1975 (synoptic),.pp. 849-850.

24. Singer, J., "The Influence of Stiffener Geometry and Spacing on the Buckling of Axially Compressed Cylindrical and Conical Shells", Proceedings of the 2nd IUTAM Symposium, Theory of Thin Shells, Copenhagen, September 1967, Springer-Verlag 1969, pp. 239-263.

25. Singer, J., and Rosen, A., "Design Criteria for Buckling and Vibration of Imperfect Stiffened Shells," ICAS Proc. 1974, Proc. of the 9th Congress of the International Council of the Aeronautical Sciences, Haifa, Israel, August 1974, Weizmann Science Press of Israel, Jerusalem 1974, pp. 495-517.

26. Weller, T., and Singer, J., "Further Experimental Studies on Buckling of Integrally Ring-Stiffened Cylindrical Shells under Axial Compression", Experimental Mechanics, Vol. 14, No. 7, July 1974, pp. 267-273.

27. Weller, T., and Singer, J., "Experimental Studies on the Buckling under Axial Compression of Integrally Stringer-Stiffened Circular Cylindrical Shells", Journal of Applied Mechanics, Vol. 44, No. 4, December 1977, pp. 721-730.

28. Stavsky, Y., and Friedland, S., "Stability of Heterogeneous Orthotropic Cylindrical Shells in Axial Compression", Israel Journal of Technology, Vol. 7, Nos. 1-2 1969, pp. 111-119.

29. Stavsky, Y., and Friedland, S., "Free-Edge Buckling of Heterogeneous Cylindrical Shells in Axial Compression", International Journal of Mechanical Sciences, Vol. 11, No. 3, 1969, pp. 217-223.

30. Greenberg, J.B., and Stavsky, Y., "Buckling and Vibration of Orthotropic Composite Cylindrical Shells", Acta Mechanica, Vol. 36, Nos. 1-2 1980, pp. 15-29.

31. Greenberg, J.B., and Stavsky, Y., "Stability and Vibration of Compressed Aeolotropic Composite Cylindrical Shells", presented at 15th International Congress of Applied Mechanics, Toronto, August 1980 (to be published in Journal of Applied Mechanics)

32. Harari, O., Singer, J., and Baruch, M., "General Instability of Cylindrical Shells with Non-uniform Stiffeners", Proceedings of the 9th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 5, No. 1, February 1967, pp. 114-128.

33. Baruch, M., Singer, J., and Weller, T., "Effect of Eccentricity of Stiffeners on the General Instability of Cylindrical Shells under Torsion", Proceedings of the 8th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 4, No. 1, February 1966, pp. 144-154.

34. Sheinman, I., and Simitses, G. J., "Buckling Analysis of Geometrically Imperfect Stiffened Cylinders under Axial Compression", AIAA Journal, Vol. 15, No. 3, March 1977, pp. 374-382.

35. Simitses, G. J., and Sheinman, I., "Optimization of Geometrically Imperfect Stiffened Cylindrical Shells under Axial Compression", Computers and Structures, Vol. 9, 1978, pp. 377-381.

36. Sheinman, I., and Simitses, G. J., "Buckling of Imperfect Stiffened Cylinders under Destabilizing Loads Including Torsion", AIAA Journal, Vol. 15, No. 12, December 1977, pp. 1699-1703.

37. Simitses, G. J., Sheinman, I., and Giri, J., "Nonlinear Stability Analysis of Pressure Loaded Imperfect Stiffened Cylindres", Journal of Ship Research Vol. 23, No. 2, June 1978, pp. 123-126.

38. Simitses, G. J., Giri, J., and Sheinman, I., "Optimization of Imperfect Stiffened Cylinders under Destabilization Loads", Acta Astronautica Vol. 7, 1980, pp. 197-207.

39. Sheinman, I., and Simitses, G. J., "Buckling and Postbuckling of Axially-Loaded Imperfect Cylindrical Shells". (Submitted for Publication AIAA Journal).

40. Singer, J., and Rosen, A., "Influence of Boundary Conditions on the Buckling of Stiffened Cylindrical Shells", Buckling of Structures, Proceedings of IUTAM Symposium on Buckling of Structures, Harvard University, Cambridge, USA, June 17-21, 1974, Springer-Verlag, Berlin 1976, pp. 227-250.

41. Weller, T., Baruch, M., and Singer, J., "Influence of In-Plane Boundary Conditions on Buckling under Axial Compression of Ring Stiffened Cylindrical Shells", Proceedings of the 5th Annual Conference of Mechanical Engineering, Israel Journal of Technology, Vol. 9, No. 4, 1971, pp. 397-410.

42. Weller, T., "Combined Stiffening and In-Plane Boundary Conditions Effects on the Buckling of Circular Cylindrical Stiffened-Shells", Journal of Computers and structures, Vol. 9, July 1978, pp. 1-16.

43. Singer, J., "The Effect of Axial Constraint on the Instability of Thin Circular Cylindrical Shells under External Pressure", Journal of Applied Mechanics, Vol. 27, December 1960, pp. 737-739.

44. Singer, J., Baruch, M. and Reichenthal, J., "Influence of In-Plane Boundary Conditions on the Buckling of Clamped Conical Shells" Proceedings of the 13th Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 9, Nos. 1-2, March 1971, pp. 127-139.

45. Weller, T., Singer, J., and Battermann, S. C., "Influence of Eccentricity of Loading on Buckling of Stringer-Stiffened Cylindrical Shells." Thin Shell Structures, Theory, Experiment and Design. Prentice-Hall, Englewood Cliffs, N. J., 1974, pp. 305-324.

46. Libai, A. and Durban, D., "A Method for Approximate Stability Analysis and its Application to Circular Cylindrical Shells under Circumferentially Varying Edge Loads", Journal of Applied Mechanics, Vol. 40, No. 4, Dec. 1973, pp. 971-976.

47. Durban, D. and Libai, A., "Buckling of Short Cylindrical Shells under Axial Compression", AIAA Journal, Vol. 12, No. 7, 1974, pp. 909-914.

48. Durban, D. and Libai, A., "Influence of Thickness on the Stability of Circular Cylindrical Shells Subjected to Nonuniform Axial Compression", Proceedings 18th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 14, 1976, pp. 9-17.

49. Libai, A. and Durban, D., "Buckling of Cylindrical Shells Subjected to Nonuniform Axial Loads", Journal of Applied Mechanics, Vol. 44, No. 4, Dec. 1977, pp. 714-719.

50. Sheinman, I., and Tene, Y., "Buckling in Segmented Shells of Revolution Subjected to Symmetric and Antisymmetric Loads", AIAA Journal Vol. 12, No. 1, January 1974, pp. 15-20.

51. Weller, T., Singer, J., and Nachmani, S., "Recent Experimental Studies on Buckling of Integrally Stiffened Cylindrical Shells under Axial Compression", TAE Report 100, Technion Research & Dev. Foundation, Ltd. Haifa, Israel, Feb. 1970.

52. Weller, T., and Singer, J., "Experimental Studies on the Buckling of 7075-T6 Aluminum Alloy Integrally Stringer-Stiffened Shells", TAE Report 135, Technion Research & Dev. Foundation, Haifa, Israel, Nov. 1971.

53. Rosen, A. and Singer, J., "Further Experimental Studies of Vibrations of Axially Loaded Stiffened Cylindrical Shells", TAE Report 210, Technion-Israel Inst. of Technology, Dept. of Aeronautical Eng., Haifa, Israel, May 1975.

54. Rosen, A., and Singer, J., "Vibrations and Buckling of Eccentrically Stiffened Cylindrical Shells," Experimental Mechanics, Vol. 16, No. 3, March 1956, pp. 88-94.

55. Rosen, A., and Singer, J., "Vibrations and Buckling of Axially Loaded Stiffened Cylindrical Shells with Elastic Restraints," International Journal of Solids and Structures, Vol. 12, No. 8, 1976, pp. 577-588.

56. Singer, J., and Abramovich, H., "Vibration Techniques for Definition of Practical Boundary Conditions in Stiffened Shells", AIAA Journal, Vol. 17, No. 7, July 1979, pp. 762-769.

57. Singer, J. and Segal, Y., "Further Experimental Studies on Vibrations and Buckling of Eccentrically Loaded Stiffened Cylindrical Shells", TAE Report 331, Technion, Israel Inst. of Technology, Dept. of Aeronautical Eng., Haifa, Israel, Dec. 1978.

58. Abramovich, H., and Singer, J., "Correlation Between Vibration and Buckling of Stiffened Cylindrical Shells under External Pressure and Combined Loading", Israel Journal of Technology, Vol. 16, Nos. 1-2, 1978, pp. 34-44.

59. Singer, J. and Segal, Y., "Direct Prediction of the Buckling Load of Stiffened Cylindrical Shells from Vibration Tests", TAE Report 405, Technion, Israel Inst. of Technology, Dept. of Aeronautical Eng., (to be published).

60. Abramovich, H., Singer, J., and Grunwald, A., "Nondestructive Determination of Interaction Curves for Buckling of Stiffened Shells", TAE Report 341, Technion, Israel Inst. of Technology, Dept. of Aeronautical Eng., Haifa, Israel, Dec. 1981.

61. Weller, T., Singer, J., and Yaffe, R., "Axisymmetric Plastic Buckling of Closely Ring-Stiffened Shells under Uniform Axial Compression", ASL Report 116, Technion, Israel Inst. of Technology, Dept. of Aeronautical Eng., Haifa, Israel, May 1981 (in Hebrew).

62. Singer, J., "Die Bestimmung der Randbedingungen von orthotrop-versteiften Schalen und Teilschalen anhand von Schwingungsversuchen", Final Report 1.10.79-30.9.81, Joint Research Project RWTH Aachen-Technion, ASL Report 121, Technion, Israel Inst. of Technology, Dept. of Aeronautical Eng., Haifa, Israel, October 1981.

63. Singer, J., Arbocz, J. and Babcock, C. D., "Buckling of Imperfect Stiffened Cylindrical Shells under Axial Compression," AIAA Journal, Vol. 9, No. 1, January 1971, pp. 68-75.

64. Singer, J., "Recent Studies on the Correlation Between Vibration and Buckling of Stiffened Cylindrical Shells", Zeitschrift fuer Flugwissenschaften und Weltraumforschung, Vol. 3, No. 6, Nov./Dec. 1979, pp. 333-344.

65. Singer, J., "Vibration Correlation Techniques for Improved Buckling Predictions of Imperfect Stiffened Shells", presented at International Conference on Buckling of Shells in Offshore Structures, Imperial College London, April 1981, (to be published in the proceedings).

66. Rosen, A., and Singer, J., "Effects of Axisymmetric Imperfections on the Vibrations of Cylindrical Shells under Axial Compression", AIAA Journal, Vol. 12, No. 7, July 1974, pp. 995-997.

67. Rosen, A., and Singer, J., "Influence of Asymmetric Imperfections of the Vibrations of Axially Compressed Cylindrical Shells", Proceedings of the 18th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 13, Nos. 1-2, 1976, pp. 23-36.

68. Singer, J., and Prucz, J., "Influence of Imperfections on the Vibrations of Stiffened Cylindrical Shells", TAE Report 369, Dept. of Aeronautical Eng., Technion, Israel, Inst. of Technology, Haifa, Israel, Feb. 1980 (to be published in Journal of Sound & Vibration 1982).

69. Singer, J., Abramovich, H. and Yaffe, R., "Initial Imperfection Measurements of Stringer Stiffened Shells", TAE Report No. 330, Dept. of Aeronautical Eng., Technion, Israel Inst. of Technology, Dec. 1978.

70. Singer, J., Abramovich, H. and Yaffe, R., "Initial Imperfection Measurements of Stiffened Shells and Buckling Predictions", Proceedings 21st Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology Vol. 17, 1979, pp. 324-338.

71. Abramovich, H., Yaffe, R. and Singer, J., "Evaluation of Stiffened Shell Characteristics from Imperfection Measurements", Collection of Papers, 23rd Israel Annual Conference on Aviation and Astronautics, Feb. 1981, pp. 114-122.

72. Yaffe, R., Singer, J. and Abramovich, H., "Further Initial Imperfection Measurements of Integrally Stringer-Stiffened Cylindrical Shells - Series 2", TAE Report 404, Dept. of Aeronautical Eng., Technion, Israel Inst. of Technology, Haifa, Israel, July 1981.

73. Abramovich, H., Singer, J. and Yaffe, R., "Imperfection Characteristics of Stiffened Shells - Group 1", TAE Report 406, Dept. of Aeronautical Eng., Technion, Israel Inst. of Technology, September 1981.

74. Rosen, A., Singer, J., Grunwald, A., Nachmani S. and Singer J., "Unified Noncontact Measurement of Vibrations and Imperfections of Cylindrical Shells", to be presented at the 7th International Conference on Experimental Stress Analysis, Haifa, Israel, August 23-27, 1982.

75. Singer, J., "On Experimental Technique for Interaction Curves of Buckling and Shells", Experimental Mechanics, Vol. 4, No. 9, Sept. 1964, pp. 279-280.

76. Singer, J., Meer, A. and Baruch, M., "Buckling of Cylindrical Panels under Lateral Pressure", The Aeronautical Journal, Vol. 73, No. 698, 1969, pp. 169-172.

77. Durban, D. and Singer, J., "Buckling of Cylindrical Panels under Nonuniform Axial Compression", Proceedings of the 15th Israel Annual Conference on Aviation and Astronautics, Israel Journal of Technology, Vol. 11, Nos. 1-2, March 1973, pp. 9-16.

78. Simitses, G. J. and Sheinman, I., "Accurate Prediction of Critical Conditions for Shear-Loaded Panels", AIAA Journal, Vol. 14, No. 5, May 1976, pp. 683-685.

79. Simitses, G. J. and Sheinman, I., "Minimum Weight Design of Stiffened Cylindrical Panels under Combined Load", Journal of Aircraft, Vol. 14, No. 5, May 1977, pp. 419-421.

80. Weller, T., "Effects of Inplane Boundary Conditions and Stiffening Combinations on Buckling of Eccentrically Stringer-Stiffened Cylindrical Panels", TAE Report 378, Technion, Israel Inst. of Technology, Haifa, Israel, March 1980 (to be published in Computers and Structures).

81. Singer, J., Rand, O. and Rosen, A., "Vibrations of Axially Loaded Stiffened Cylindrical Panels with Elastic Restraints", TAE Report 439, Dept. of Aeronautical Engineering, Technion, Israel Inst. of Technology, Haifa, July 1981.

82. Baruch, M., "Parametric Instability of Stiffened Cylindrical Shells", Israel Journal of Technology, Vol. 7, No. 4, 1969, pp. 297-301.

83. Maymon, G. and Singer, J., "Dynamic Elastic Buckling of Stringer-Stiffened Cylindrical Shells under Axial Impact," Israel Journal of Technology, Vol. 9, No. 6, 1971, pp. 595-606.

84. Maymon, G. and Libai, A., "Dynamics and Failure of Cylindrical Shells Subjected to Axial Impact", AIAA Journal, Vol. 15, No. 11, Nov. 1977, pp. 595-606.

85. Elishakoff, I. and Arbocz, J., "Buckling of Shells with General Random Imperfections", presented at the 15th International Congress of Theoretical and Applied Mechanics, Toronto, Canada 17-23 August, 1980.

86. Shilkrut, D., "Solutions of Some Stability Problems in the Theory of Geometrically Nonlinear Shells", Collention of Papers, 22nd Israel Annual Conference on Aviation and Astronautics, March 12-13, 1980, pp. 107-114.

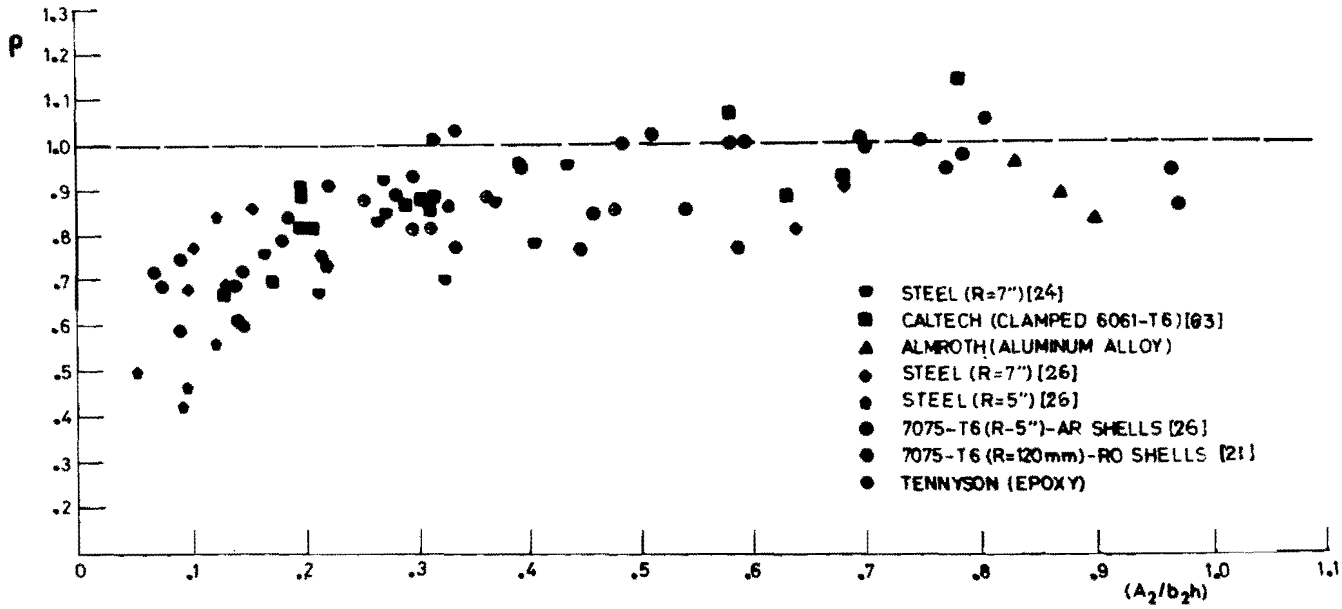


Fig. 1 "KNOCK-DOWN FACTOR", OR "LINEARITY", ρ OF RING-STIFFENED CYLINDRICAL SHELLS UNDER AXIAL COMPRESSION AS FUNCTION OF RING AREA PARAMETER (from [1]).

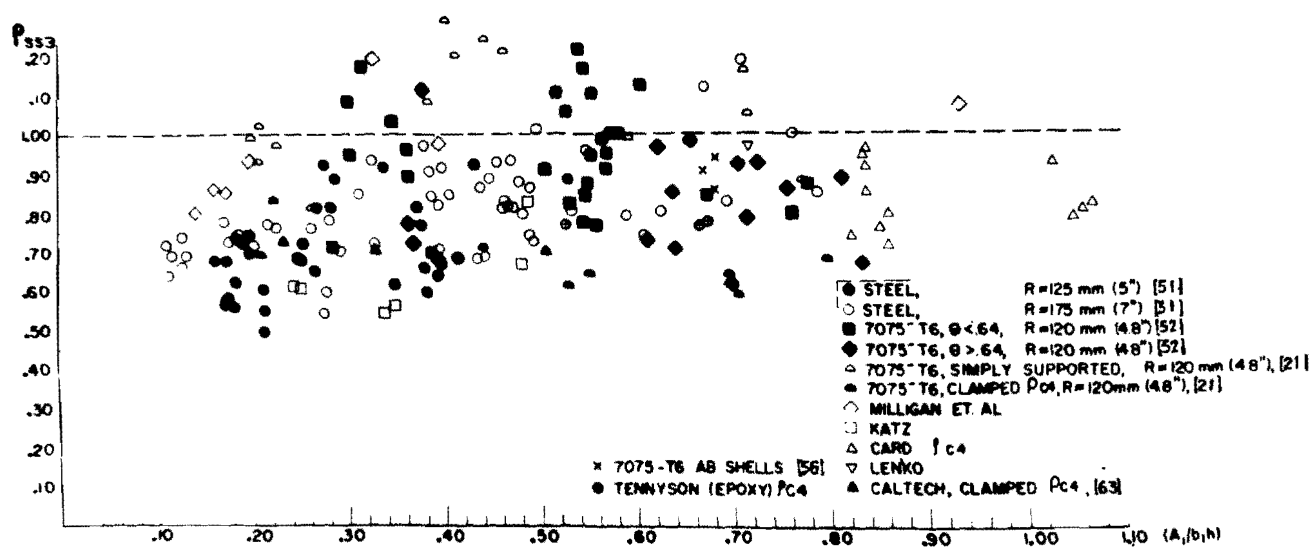


Fig. 2 "KNOCK-DOWN FACTOR", OR "LINEARITY", ρ OF STRINGER-STIFFENED CYLINDRICAL SHELLS UNDER AXIAL COMPRESSION AS FUNCTION OF STRINGER AREA PARAMETER (from [1]).

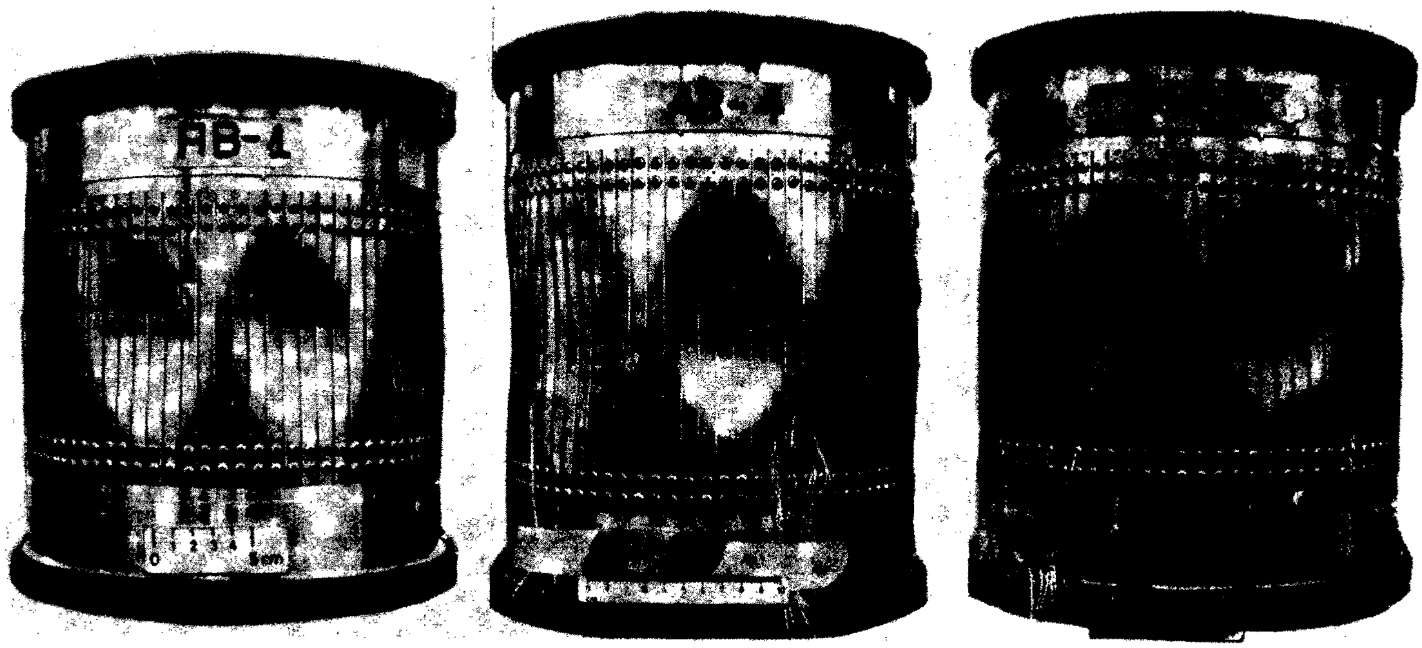


Fig. 3 TYPICAL POST BUCKLING PATTERNS FOR AB SHELLS (from [64]).

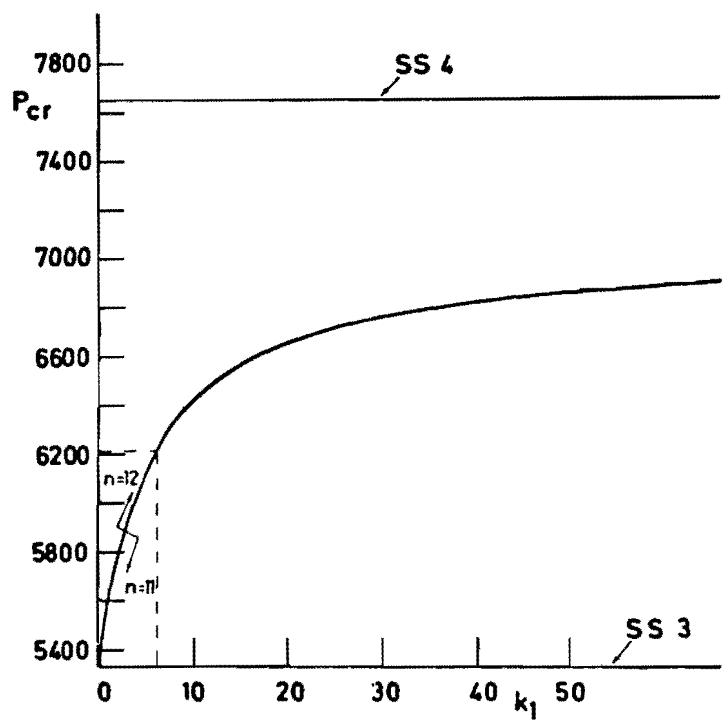


FIG. 4 INFLUENCE OF ELASTIC AXIAL RESTRAINT ON THE BUCKLING OF SHELL RO-32 (from [53])

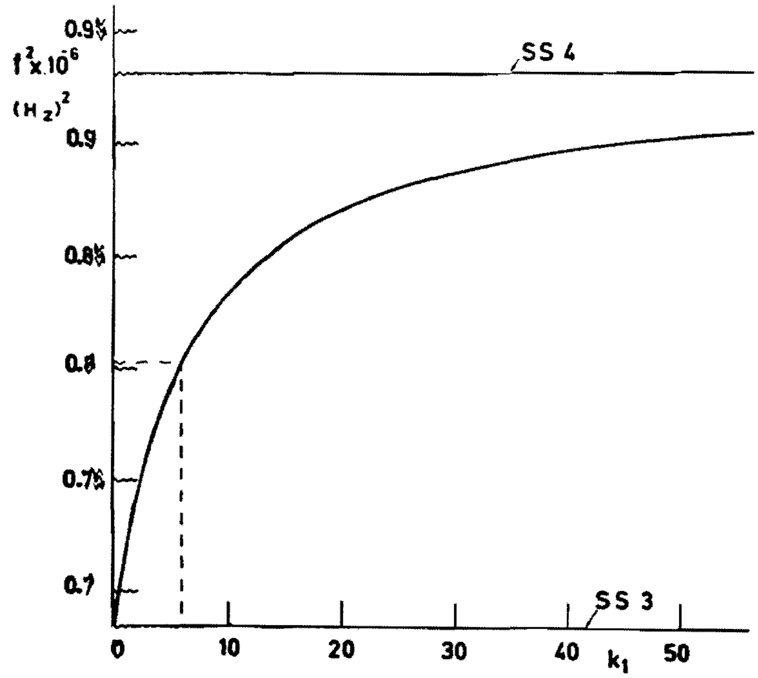


FIG 5 INFLUENCE OF ELASTIC AXIAL RESTRAINT ON THE VIBRATIONS OF SHELL RO-32 (P=1600 kg, m=1, n=12; from [53]

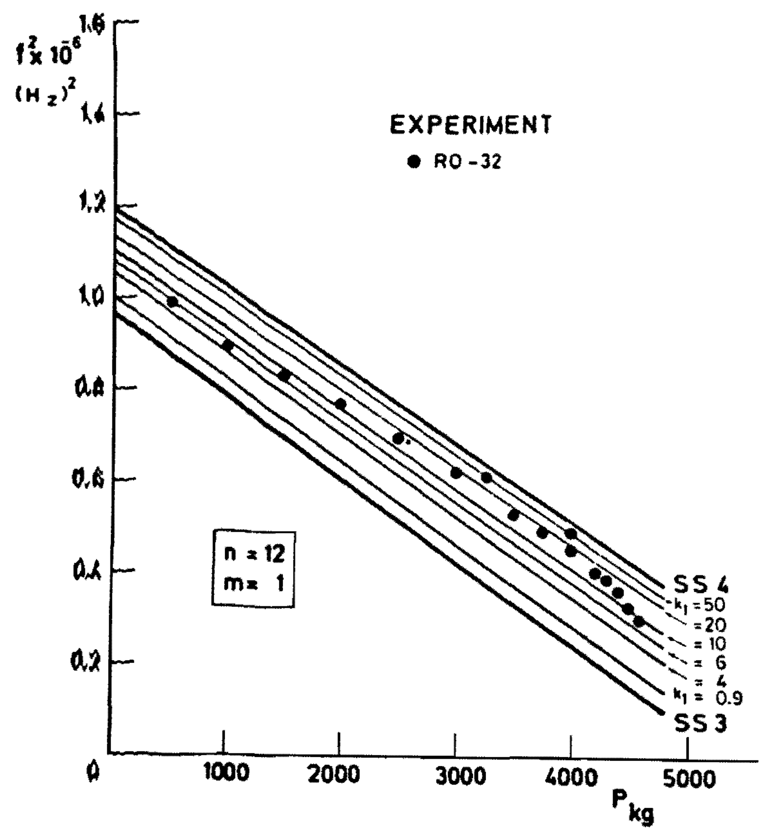


FIG 6 FREQUENCY SQUARED VS. AXIAL LOAD - SHELL RO-32 (from [53])

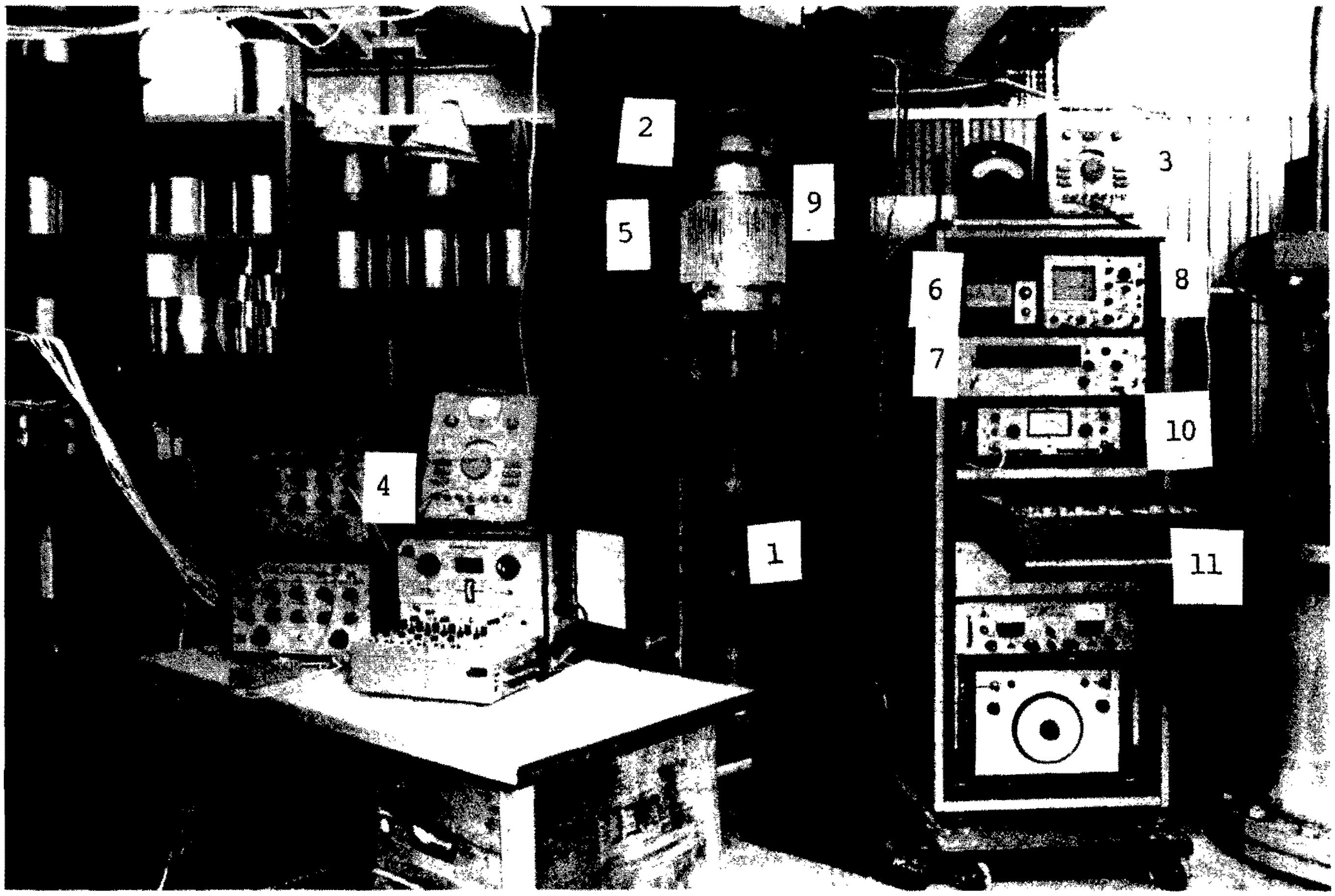


Fig. 7 TEST SET-UP FOR AXIAL COMPRESSION LOADING WITH SHELL DK1 (from [65]).

1 - SCREW JACK
2 - LOAD CELL
3 - LOAD CELL STRAIN INDICATOR
4 - SWITCHING & BRIDGE CIRCUIT
5 - SHAKER
6 - OSCILLATOR
7 - DIGITAL COUNTER
8 - OSCILLOSCOPE
9 - MICROPHONE
10 - AMPLIFIER
11 - X-Y RECORDER

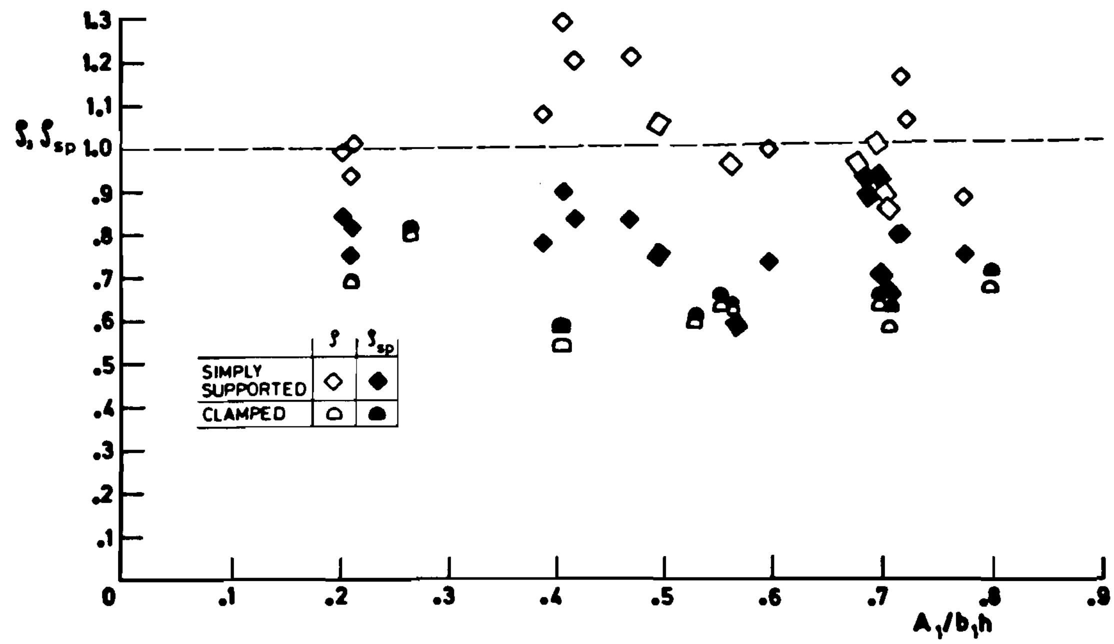


Fig. 8 "KNOCK-DOWN FACTOR" OF 31 SIMPLY SUPPORTED AND CLAMPED STRINGER-STIFFENED SHELLS OF RO, AB, KR, SN, AND RS SERIES, CORRECTED FOR EXPERIMENTALLY (NON-DESTRUCTIVE-LY) DETERMINED BOUNDARY CONDITIONS. ρ_{sp}, THE CORRECTED "KNOCK-DOWN FACTOR" IS COMPARED TO THE ORIGINAL ρ_{SS3} AND ρ_{C4} VALUES (from [65]).

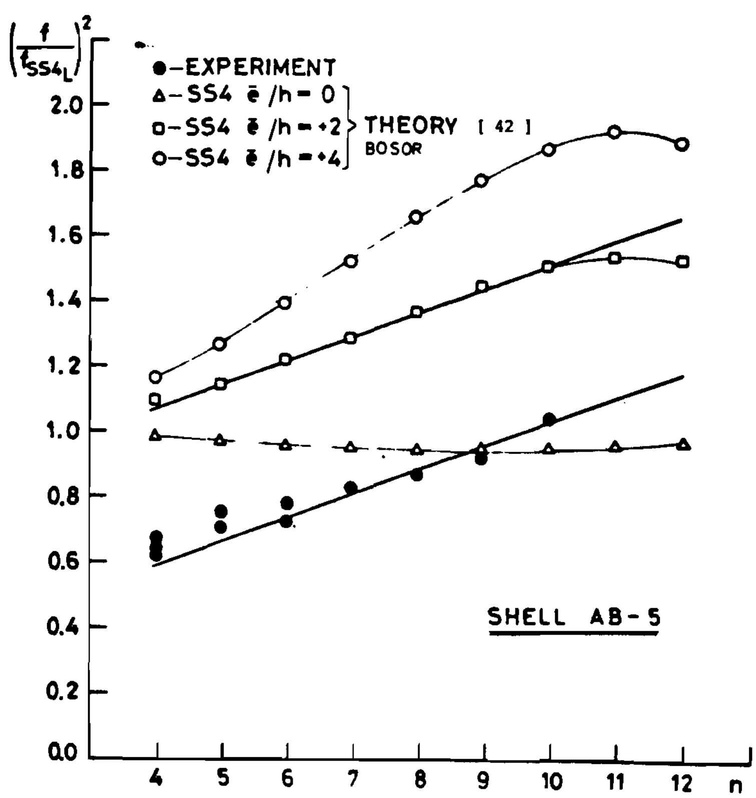


FIG 9 VARIATION OF FREQUENCY RATIO SQUARED WITH CIRCUMFERENTIAL WAVE NUMBER n, AT P=1000kg - SHELL AB-5 (from [56])

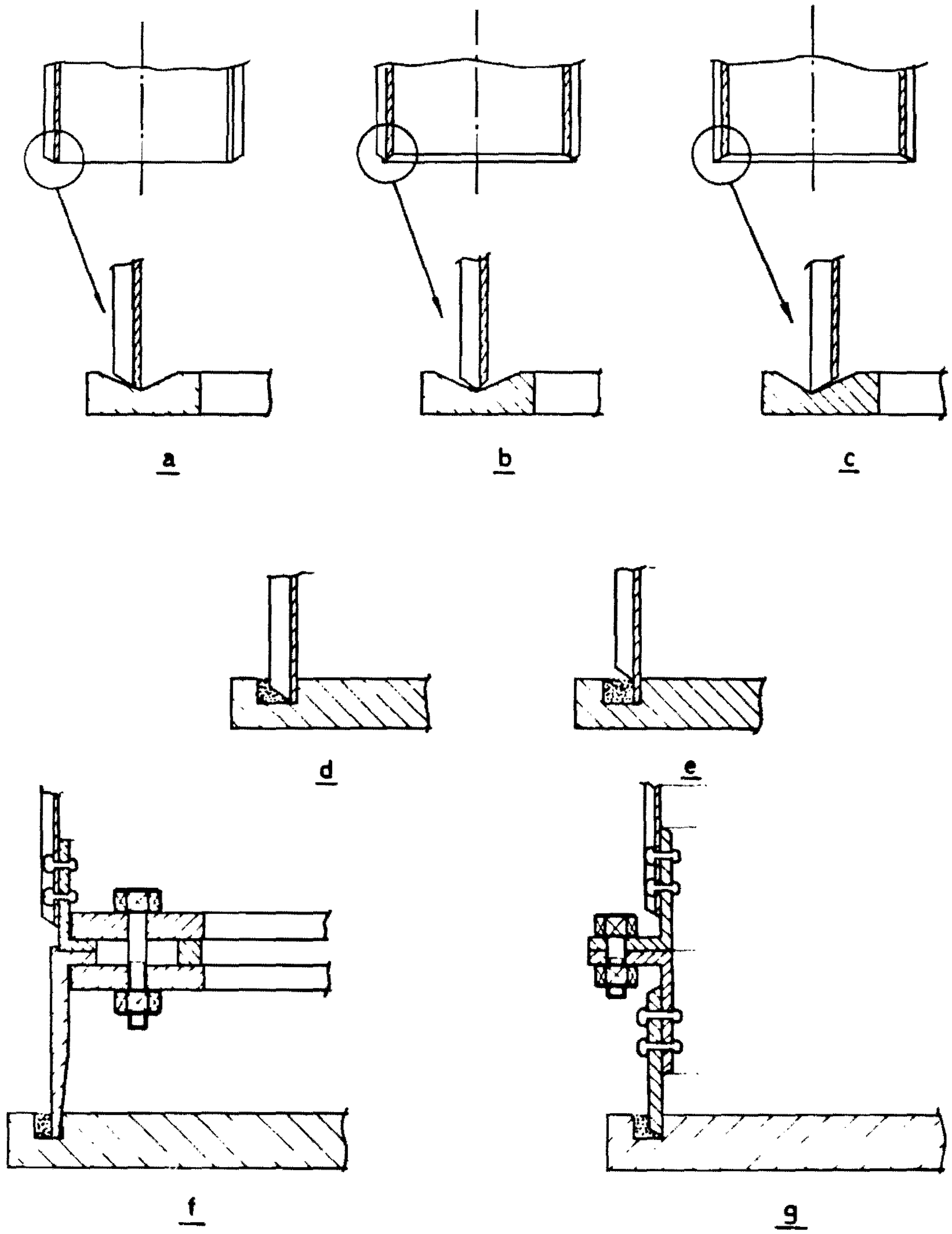


Fig. 10 BOUNDARY CONDITIONS OF SHELLS TESTED IN THE VIBRATION CORRELATION STUDIES (from [65]): a,b,c - RO AND SN SHELLS; d - RO, ST, KR AND DUD (2 AND 3) SHELLS; e - DUD (4 AND 5) SHELLS; f - AB SHELLS; g - DK SHELLS.

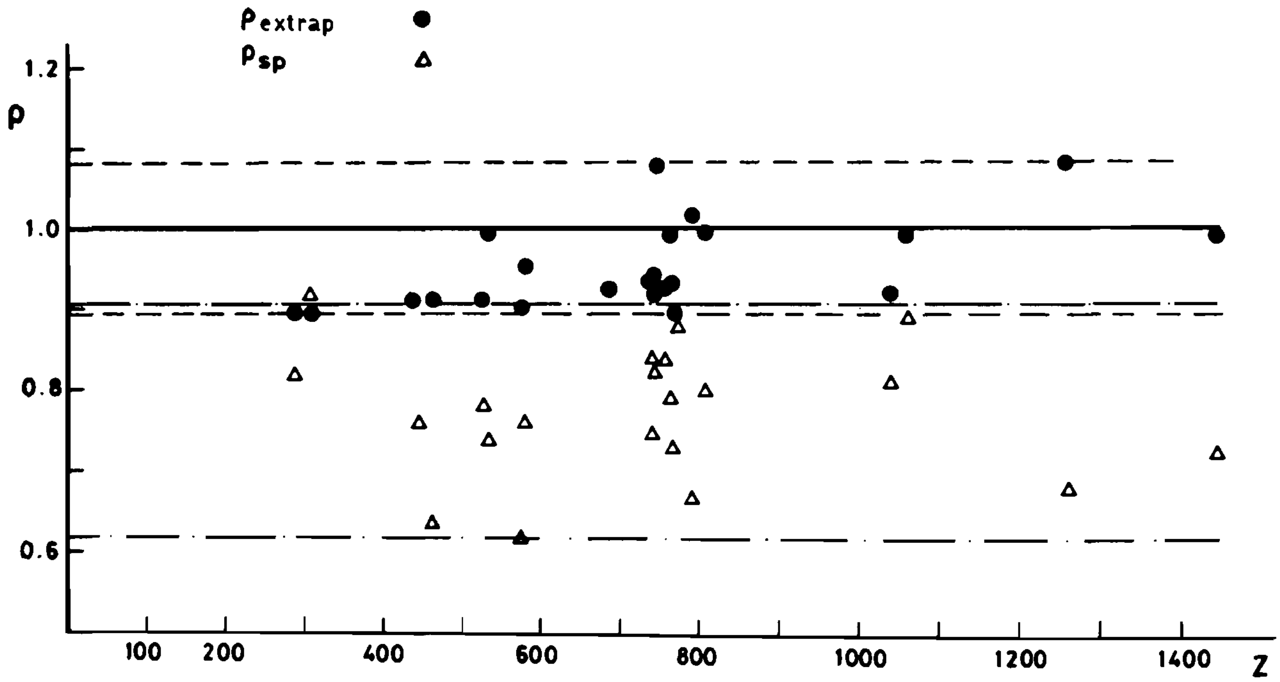


FIG 11 PREDICTION OF BUCKLING LOAD WITH DIRECT METHOD, COMPARISON WITH RESULTS FROM VIBRATION CORRELATION METHOD (from [59])

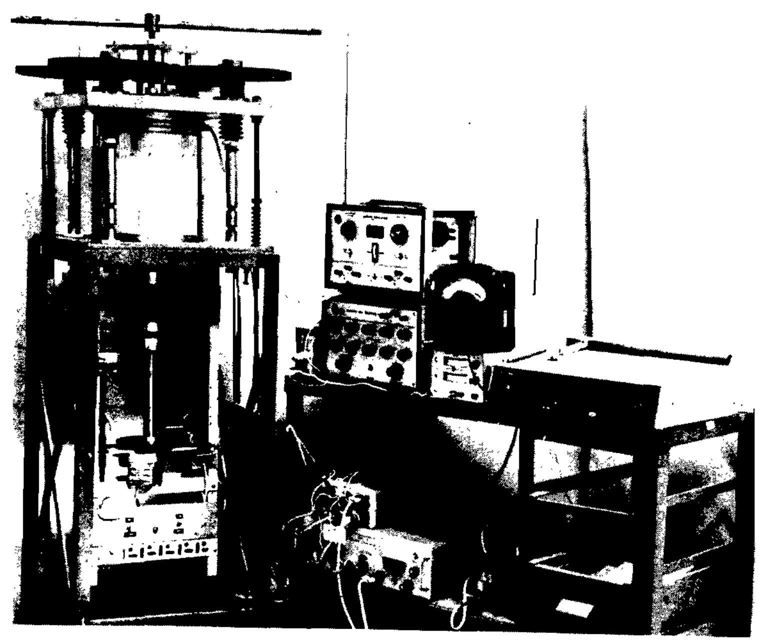


FIG 12 TEST RIG FOR SCANNING OF IMPERFECTIONS AND THEIR GROWTH UNDER AXIAL LOAD FOR SHELLS WITH R-120 mm (from [1])

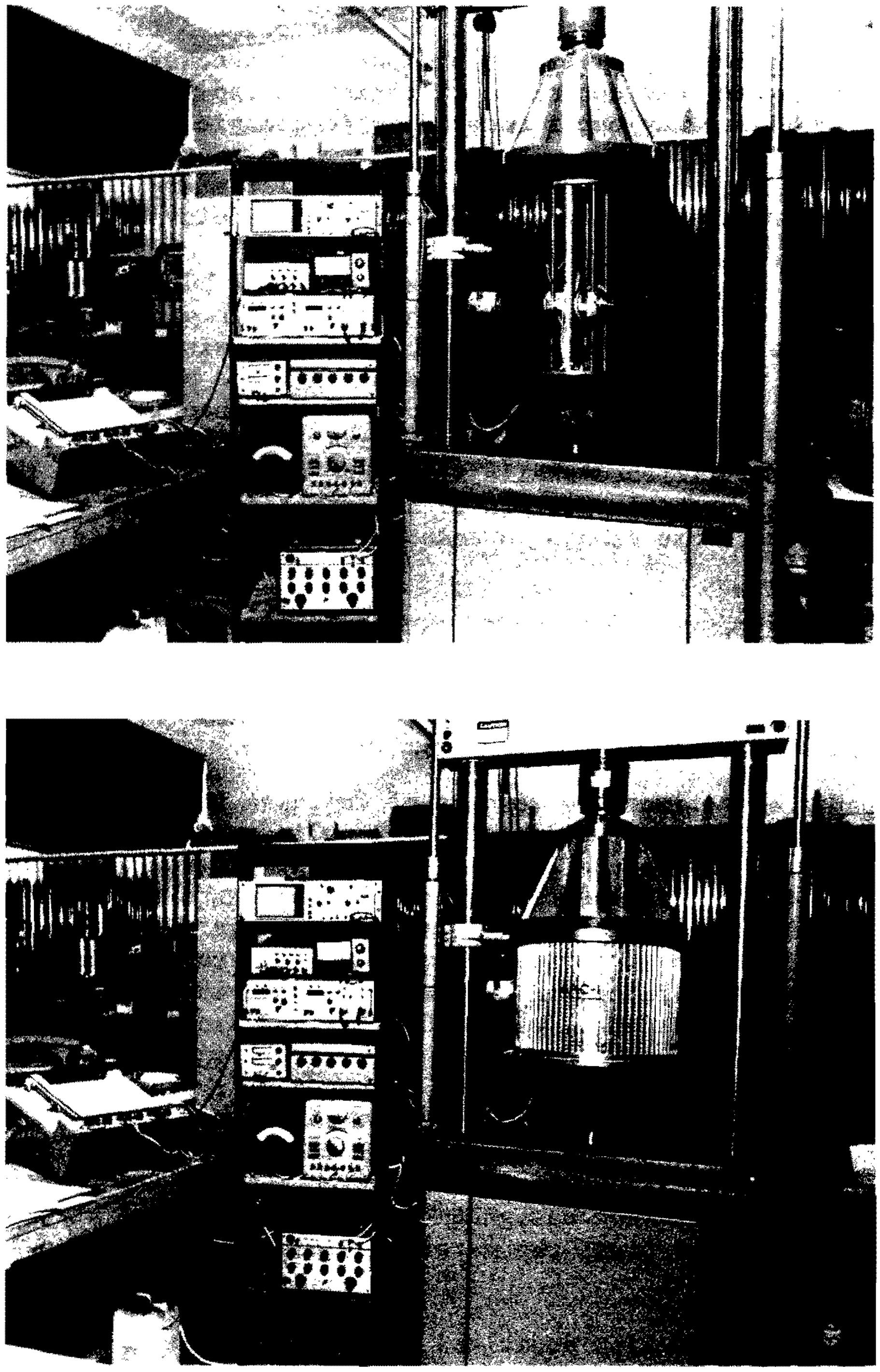


Fig. 13 TEST SET-UP FOR VIBRATION CORRELATION TEST ON SPOT-WELDED SHELL AAC-1, WITH MULTIPURPOSE SCANNING AND MEASUREMENT SYSTEM FOR VIBRATIONS AND IMPERFECTIONS.
a. THE SCANNING AND MEASUREMENT SYSTEM.
b. TEST SYSTEM IN OPERATION.

DISCUSSION

S. Sridharan (Washington University): Your analyses have been elastic only. However, when load eccentricity is introduced to an axisymmetric shell, early collapse may occur under elastic-plastic conditions.

Singer: Here, it has been assumed that all failures are elastic. However, I agree that there are situations when inelastic instability is a problem.

Model tests are very useful in determining the buckling of real structures. The imperfections are of the same order of importance, although it is recognized that the residual stress distributions are not the same. It is my contention that more use will be made of closely spaced stiffened shells with (D/t) ratios greater than 1000. Shells are only sensitive to imperfections that coincide with their buckling shape and closely spaced stiffened shells will have a buckling shape that is different from the probable pattern of imperfections. This will involve placing stiffeners close enough to ensure general instability.

STABILITY UNDER HYDROSTATIC PRESSURE AND AXIAL TENSION

R. K. Kinra, Shell Oil Company

Introduction

This paper summarizes the results of collapse tests on forty-two stiffened and unstiffened fabricated steel cylinders under combined axial tension and external pressure loading. The test program was initiated because the existing data base consisted only of tests on small diameter manufactured tubes with a maximum diameter/thickness (D/t) ratio of 27. The objective of this test series was to develop a comprehensive data base for fabricated cylinders and to determine the adequacy of current API RP 2A[1] design rules for tension and collapse loading.

The test specimen parameters were selected to represent typical offshore platform member sizes (D/t from 36 to 96), commonly used platform steels with yield strengths of 36 to 50 ksi (A36, A633 and A572) and routine platform fabrication procedures. Eighteen and twenty-four inch diameter test specimens were designed to fail at different stress levels, ranging from 0.3 to 0.9 times the material yield strength, when loaded by radial pressure only.

Theoretical interaction equations for predicting the collapse strength of cylinders under combined axial tension and hoop compression are available for elastic buckling and yield failures. However, no theoretical equations exist for the transition range between these limits. The current API interaction equations are empirical equations based on a conservative interpretation of the available test data on small diameter manufactured tubes. Different empirical equations were developed by Miller[2] based on an analysis of the same test data. These theoretical and empirical interaction equations are described and compared with the test program results in this paper.

Theoretical Interaction Equations

The theoretical elastic buckling equation for a cylindrical shell under combined axial load and hoop compression is given by Timoshenko and Gere[3]. For an axial tension stress, the interaction curve determined from this equation is a straight line:

$$\sigma_\theta = \sigma_{re} + 2\frac{\sigma_x}{\sigma_{he}}(\sigma_{re} - \sigma_{he}) \tag{1}$$

where

σ_x, σ_θ = stresses in the axial and hoop directions

σ_{re}, σ_{he} = theoretical buckling stresses in the hoop direction for a cylinder under radial pressure and hydrostatic pressure, respectively.

Elastic buckling tests by Mungan[4] on plexiglass cylinders showed that tensile loads increased the buckling stresses in the hoop direction as predicted by Eq. 1. The D/t values varied between 146 and 400. The ratios of buckling stresses in the hoop direction to the yield stresses were low.

Equation 1 is valid only when elastic buckling occurs. The cylinder may also fail inelastically or by yielding. Several yield failure theories and discussed by Seely and Smith[5]. For a biaxial stress field, the maximum total energy theory, proposed by Beltrami and Haigh, is given by:

$$\sigma_x^2 + 2\mu\sigma_x\sigma_\theta + \sigma_\theta^2 = F_y^2 \tag{2}$$

where

μ = Poisson's ratio

F_y = yield stress

For cylinders under axial tension and hoop compression, Eq. 2 can be written as follows:

$$\left(\frac{\sigma_x}{F_{yx}}\right)^2 + 2\mu\frac{\sigma_x}{F_{yx}}\left|\frac{\sigma_\theta}{F_{y\theta}}\right| + \left(\frac{\sigma_\theta}{F_{y\theta}}\right)^2 = 1.0 \tag{3}$$

where

$F_{yx}, F_{y\theta}$ = yield stresses in the axial and hoop directions

Equation 3 is equivalent to the Hencky-von Mises maximum energy of distortion theory when $\mu = 0.5$.

Empirical Interaction Equations

Tests have been performed on manufactured steel tubes by Edwards and Miller[6] and Stuiver and Tomalin[7] and on steel casing by Kyogoku, et. al.[8] The D/t ratios varied from 11.2 to 27.0 and the yield stresses from 31.4 ksi to 81.9 ksi for the steel tubing. For the steel casing, the D/t ratios varied from 16.2 to 24.4 and the yield stresses from 89.0 ksi to 124.6 ksi.

For each group of steel cylinders with the same D/t ratio and yield strength, Miller[2] found that the interaction curve described by the test data is nearly a straight line until the line intersects the interaction curve given for yield failure. The test data then follow the yield failure curve given by Eq. 3. The equation for the straight line is given by:

$$\sigma_\theta = F_{rc} \left(1 - 0.25 \frac{\sigma_x}{F_y}\right) \tag{4}$$

where F_{rc} is the lower bound value of the buckling stresses in the hoop direction for the cylinders in each group subjected to radial pressure only. The test values of F_{rc} ranged between $0.48F_y$ and $1.0F_y$. Since the lower values of F_{rc} are elastic, one can assume that Eq. 4 is valid in the elastic as well as the inelastic range.

Comparisons have been made between the interaction curves given by Eqs. 3 and 4 and the available test data. Using a value of $\mu = 0.5$ in Eq. 3 gives a lower bound on the test data of Refs. 7 and 8, whereas a value of $\mu = 0.75$ must be used to give a lower bound on some of the test data of Ref. 6. Although values of Poisson's ratio greater than 0.5 are physically inadmissible, μ is treated here as a correlation parameter.

The interaction equation given by the current API design rules, Eq. 5, is a modification of Eq. 3. The predicted buckling stress in the hoop direction of a cylinder under hydrostatic pressure, F_{hc}, is substituted for $F_{y\theta}$ and μ is assumed equal to 0.3. The equation is conservative with respect to all the available test data.

$$\left(\frac{\sigma_x}{F_{yx}}\right)^2 + 0.6 \frac{\sigma_x}{F_{yx}} \left|\frac{\sigma_\theta}{F_{hc}}\right| + \left(\frac{\sigma_\theta}{F_{hc}}\right)^2 \leq 1.0 \tag{5}$$

The hydrostatic collapse stress F_{hc} was selected for the API rules, rather than the radial collapse stress F_{rc}, because there is little difference in these values for typical platform members. The difference becomes significant for closely spaced rings and may become important for deep water platforms. The collapse stresses F_{rc} and F_{hc} may be determined from:

$$F_{ic} = \eta\alpha\sigma_{ie} \tag{6}$$

where

η = plasticity reduction factor which accounts for nonlinearities in material properties and residual stresses

α = capacity reduction factor which accounts for the effects of initial imperfections, boundary conditions and geometric nonlinearities

σ_{ie} = theoretical elastic buckling stress of a cylinder without imperfections and with simple support boundary conditions

Values for η and α are taken from Ref. 9 and values for σ_{ie} are taken from Ref. 10.

Description of Test Models

A total of 42 test cylinders were fabricated from two different yield strength materials. Twenty-seven cylinders were made from A36 material which has a minimum specified yield stress of 36 ksi and fifteen cylinders were made from A633 GR C and A572 GR 50 materials which have minimum specified yield stresses of 50 ksi. A minimum of two tensile coupon tests per plate were performed in accordance with ASTM A370 to determine the material yield strength and stress-strain properties. The test coupons were taken from the plates prior to rolling and in directions corresponding to the axial and hoop directions of the test cylinders. The mechanical properties of the test coupons and the material stress-strain curves are given in Refs. 11 and 12.

The geometries of the test models are given in Fig. 1. There were a total of 9 sets of models with 3 to 6 models with the same geometry and material in each set. Groups 1 to 5 were made from A36 material and groups 6 to 9 were made from A633 and A572 materials. The geometries of groups 1 to 4 were designed to fail at stress levels of $0.3F_y$, $0.5F_y$, $0.7F_y$, and $0.9F_y$, respectively, when loaded by radial pressure only with F_y = 36 ksi. Group 5 was designed to fail at the same stress level as Group 4. This provided data for comparison between stiffened and unstiffened cylinders. The geometries of groups 6 to 9 are the same as groups 1, 2, 3 and 5, respectively. This provides data for direct correlation of the influence of yield stress on buckling strength.

The ring sizes for the 1980 tests were based on the requirements of the API rules[1]. These rules are based upon the requirement that the elastic general instability collapse pressure be 20% greater than the elastic local buckling pressure. Although all test cylinders were designed to fail by local buckling (buckling of the shell between rings or end bulkheads), failures were observed in the butt welds of several ring stiffeners and in the attachment welds of the ring and shell on two specimens.

A metallurgical examination of the failed welds indicated that a large degree of nonfusion and lack of penetration were present. The butt splices which remained intact during the tests also showed significant weld defects. All defective welds were made using the CO_2 shielded "short arc" welding process. This process was not used for the 1981 test specimens.

The 1981 test program was initiated before the metallurgical report on the 1980 tests became available. Although it was suspected that the ring failures during the 1980 tests were due to defective welds, the ring depths of the 1981 test specimens were increased, as shown in Fig. 1, in order to eliminate the possiblity of ring failures.

Test Equipment and Procedure

All testing was performed by Southwest Research Institute with the exception of the materials testing and metallurgical examinations which were carried out by Chicago Bridge and Iron Company.

Out-of-roundness measurements were taken at the middle of each test bay and at one or more rings for the ring stiffened cylinders and at three cross sections for the unstiffened cylinders. Calipers were used to determine the maximum and minimum diameters and a three point gage was used to measure the deviation from a true circular arc over a distance of one-half wave length. Measurements were made at 24 equally spaced locations around the circumference at each cross section. A tabulation of all measurements is given in Refs. 11 and 12.

Residual strain measurements were made on two specimens during the 1980 program. Readings were taken at four times during the fabrication process. Measurements were made using a Wheatstone bridge extensometer calibrated over a 100mm reference distance. During the 1980 program, a single model was instrumented with strain gages and tested to yield under tension load only. During the 1981 program, one test specimen from each group of specimens was instrumented with strain gages. The strain measurements are summarized in Refs. 11 and 12.

An axial load frame with the capability of applying 1570 kips tension was designed and fabricated for the test program. The frame accommodates 144 in. long models which are attached by full penetration welds to load bearing plates at both ends. The test procedure for each specimen was as follows. The frame and test specimen were placed inside a 90 in. I.D., 4000 psig pressure vessel. The test cylinder was filled with water and vented to the atmosphere through the pressure vessel closure. The axial tension load was applied using four hydraulic jacks and the external pressure was applied by pressurizing the tank. The jack loads were adjusted to compensate for the tank pressure acting on the ends of the test cylinders, and the loading was incremented in a step-wise manner.

In the 1980 test program, the axial load was applied first and held constant during application of the external pressure. There was some concern, however, that the loading sequence might affect the measured collapse resistance of the cylinders. To check this hypothesis, the 1981 models were tested using two loading sequences. Specimens from groups 7-9 were tested using the same procedure as 1980. Specimens from groups 1-6, however, were tested by first applying external pressure equal to 80% of the predicted collapse pressure, then applying the prescribed axial load and holding it constant while the external pressure was increased in increments until the cylinder collapsed.

The collapse of the test cylinders was determined by monitoring the efflux of water from the interior of the cylinders. The test procedure called for holding a particular loading increment constant until the efflux of water subsided, indicating that the cylinder was in equilibrium with the applied loading. When the efflux of water increased significantly and did not subside, collapse of the cylinder was indicated, and the test was terminated.

Discussion of Test Results

The test results are compared with the interaction curves given by Equations 3, 4, and 5 in Figs. 2 and 3. From prior tests on small diameter tubing, it was found that a value of $\mu = 0.75$ was needed in Eq. 3 to obtain a lower bound on available test data. Therefore, values of $\mu = 0.75$ and $\mu = 0.5$, which corresponds to the Hencky-von Mises failure theory, were used when making comparisons of Eq. 3 with the test results. The 1980 tests are shown by circles and the 1981 tests by squares.

The 1981 test results for groups 1-6 are significantly higher than the corresponding 1980 test results. The sequence of loading was different in 1981 as explained in the test procedure. Also, the ring sizes were increased and all welds were of good quality. The sequence of loading appears to be the most signicicant of the changes made. The same general increase in failure stress is noted for the 1981 tests in groups 3 and 4. For these groups, the rings remained round in the 1980 as well as the 1981 tests. Therefore, the size of rings and quality of welding do not explain the difference in trend between the 1980 and 1981 test results.

The models of group 1 were designed to fail at a stress ratio of $\sigma_\theta/F_{y\theta} = 0.9$ when $\sigma_x = 0$ and $F_{y\theta} = 36$ ksi. Model 1C was tested to yield under axial tension only. A hydrostatic pressure load was then applied to the model and the test point is designated 1C*. All test points for group 1 are found to follow the Hencky-von Mises theory closely even though the predicted stress ratio for $F_{rc}/F_{y\theta}$ is below 0.9. The butt welds in the rings of models 1A, 1C and 1D failed, probably after local buckling. Some ovalizing of the rings was observed on models 1E and 1F.

The models of group 2 were designed to fail at a stress ratio of $\sigma_\theta/F_{y\theta} = 0.7$ when $\sigma_x = 0$ and $F_{y\theta} = 36$ ksi. An error was made in calculating the required jack pressure for tests 2A, 2B and 2C and the resulting axial loads were somewhat lower than specified. Therefore, the desired combinations of loading at failure were not quite achieved. The loading condition for model 2A approached hydrostatic conditions. The shell separated from ring 2 on model 2B due to poor welding. The effect of this is not apparent in the plot of the test results. Some ovalizing of the center ring was observed on model 2D. The 1980 test results closely follow the predicted values of Eq. 4 and Eq. 3 with $\mu = 0.5$. Test 2E which was performed in 1981 is 1.2 times the value predicted by Eq. 4 and 1.34 times the API predicted value.

The models of group 3 were designed to fail at a stress ratio of $\sigma_\theta/F_{y\theta} = 0.5$ when $\sigma_x = 0$ and $F_{y\theta} = 36$ ksi. The required jack pressures were slightly in error for tests 3A, 3B and 3C resulting in lower than specified axial loads. Models 3A and 3D failed at σ_θ values less than predicted. Models 3E and 3F were tested in 1981 to give additional data points in the region of 3D. Both of these models failed above the predicted values. All rings remained circular. The validity of model test 3A will be investigated under a separate test program for the buckling capacity of cylinders under combined axial compression and hoop compression.

The models of group 4 were designed to fail at a stress ratio of $\sigma_\theta/F_{y\theta} = 0.3$ when $\sigma_x = 0$ and $F_{y\theta} = 36$ ksi. The jack pressure was in error for test 4A, resulting in approximately hydrostatic loading conditions at failure instead of the desired radial loading condition. Model 4D failed at about 70% of the value predicted by Eq. 4. Models 4E and 4F were tested in 1981. The failure loads for these models were found to be significantly higher than for 4C and 4D. All rings remained round.

The models of group 5 were designed to fail at the same stress ratios as the group 4 models. The models of group 5 were unstiffened whereas the models of groups 1-4 were ring stiffened. All group 5 models failed at values of σ_θ higher than predicted and no additional tests were preformed in 1981. The ratio of the test stress and the predicted failure stress given by Eq. 4 is 1.04 for model 5D compared with about 1.3 for the other models.

The models of group 6 were identical to those of group 1 except that they were fabricated from 50 ksi yield stress material. Models 6A, 6B and 6C were tested in 1980. Only model 6C failed above the predicted failure stress. The results of these tests are questionable because the butt welds in the rings of models 6A and 6C failed and the shell pulled away from one ring of model 6B due to poor welding. Models 6D and 6E were tested in 1981. The failure stresses of both models exceeded the predicted values of Eqs. 3 and 4.

The models of groups 7, 8 and 9 were identical to those of groups 2, 3 and 5 except that 50 ksi yield stress material was used. The failure stress ratios of all models follow Eq. 4 closely.

Conclusions

The predicted buckling stress values determined from the current API rules for tension and collapse loading are found to be conservative for all tests, including those test values determined from models with defective welding of the rings, with the exception of model 6A. In many cases the rules are found to be overly conservative, with ratios of test stress to predicted stress ranging from 0.93 to 2.0.

Equation 3 with $\mu = 0.5$ and Equ. 4 of the present paper are found to be in much closer agreement with the test results. The test stress is nearly equal to or higher than the predicted stress for all models except 3D, 4D, 6B and 4F. The ratio of test stress to predicted stress, $\sigma_{\theta T}/\sigma_{\theta P}$, for all models except these, ranges from 0.97 to 1.39. A probable reason for model 6B being low is that the shell pulled away from one of the rings during test. No explanation has been determined for the low values for models 3D, 4D and 4F where the ratios of $\sigma_{\theta T}/\sigma_{\theta P}$ are 0.723, 0.702 and 0.893, respectively. The axial stress exceeds $0.7F_{yx}$ for all of these models . This does not lead to a possible explanation for the problem, since the axial stress for models 2D, 3F, 5D, and 9C also exceeds $0.7F_{yx}$ and these models failed at ratios of $\sigma_{\theta T}/\sigma_{\theta P}$ ranging from 0.98 to 1.10.

Further study is required to fully define the failure mechanism in the region where the straight line defined by Eq. 4 approaches the Hencky-von Mises curve given by Eq. 3 with $\mu = 0.5$. Substitution of a higher value for $\mu (\mu = 0.75)$ into Eq. 3 was found to give a lower bound on test data for small diameter tubes. The resulting interaction curves are also shown in Figures 2 and 3. This approach does not account for any of the low values of the present tests and, therefore, does not provide a suitable alternative. Additional analytical and experimental studies are recommended. Also, further studies should be made of the effect of the sequence of load application since higher values of $\sigma_{\theta T}/\sigma_{\theta P}$ were found when the external pressure was applied before the axial tension load.

Acknowledgements

Two series of tests are reported in this paper. The first series of twenty-four tests, carried out in 1980, was sponsored by Amoco, Brown & Root, Chevron, CBI, Cities Service, Exxon, Gulf, McDermott, Mobil, Shell and Union Oil. The second series of eighteen tests, carried out in 1981, was sponsored by the American Petroleum Institute.

References

1. "Recommended Practice for Planning, Designing, and Construction Fixed Offshore Platforms," API RP 2A, Twelfth Edition, January 1981.

2. Miller, C. D., "Buckling of Cylindrical Shells Under Combined Axial Tension and Hoop Compression," Chicago Bridge and Iron Company, Research Report, June 1980.

3. Timoshenko, S. P., and Gere, J. M., Theory of Elastic Stability, Second Edition, McGraw-Hill, New York, 1961, pp. 495-499.

4. Mungan, I., "Buckling Stress States of Cylindrical Shells," Journal of the Structural Division, ASCE, Vol. 100, No. ST11, Nov. 1974, pp. 2289-2306.

5. Seely, F. B., and Smith, J. O., Advanced Mechanics of Materials, John Wiley & Sons, New York, Second Edition, 1955, pp. 76-91.

6. Edwards, S. H., and Miller, C. P., "Discussion of the Effect of Combined Longitudinal Loading and External Pressure on the Strength of Oil Well Casing," Drilling and Production Practice, Proc. of API, 1939, pp. 483-502.

7. Stuiver, W., and Tomalin, P. F., "The Failure of Tubes Under Combined External Pressure and Axial Loads," SESA Proceedings, Vol. 16, No. 2, pp. 39-48.

8. Kyogoku, T. et. al., "Experimental Study of the Effect of Axial Tension Load on the Collapse Strength of Oil Well Casing," Offshore Technology Conference, Paper No. OTC 4108, 1981.

9. Miller, C. D., and Kinra, R. K., "External Pressure Tests of Ring Stiffened Fabricated Steel Cylinders," Offshore Technology Conference, Paper No. OTC 4107, 1981.

10. "Metal Containment Shell Buckling Design Methods," ASME Boiler and Pressure Vessel Code, Case N-284, Supplement 2 to Nuclear Code Case Book (1980).

11. Marlow, R. S., "Collapse Performance of Fabricated Cylinders Under Combined Axial Tension and External Pressure," SWRI Project No. 03-6139, Jan. 1981.

12. Marlow, R. S., "Collapse Performance of Fabricated Cylinders Under Combined Axial Tension and External Pressure," SWRI Project No. 03-6809, April 1982.

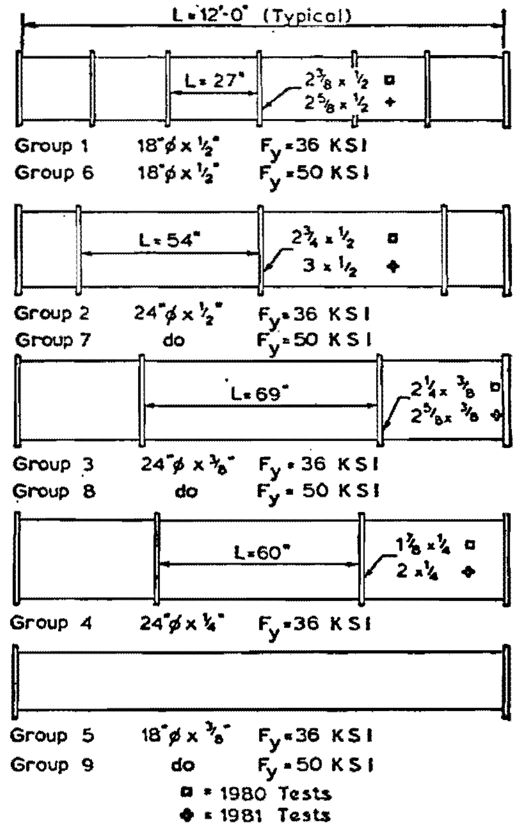


Fig 1 Geometry Of Test Specimens (Nominal Dimensions)

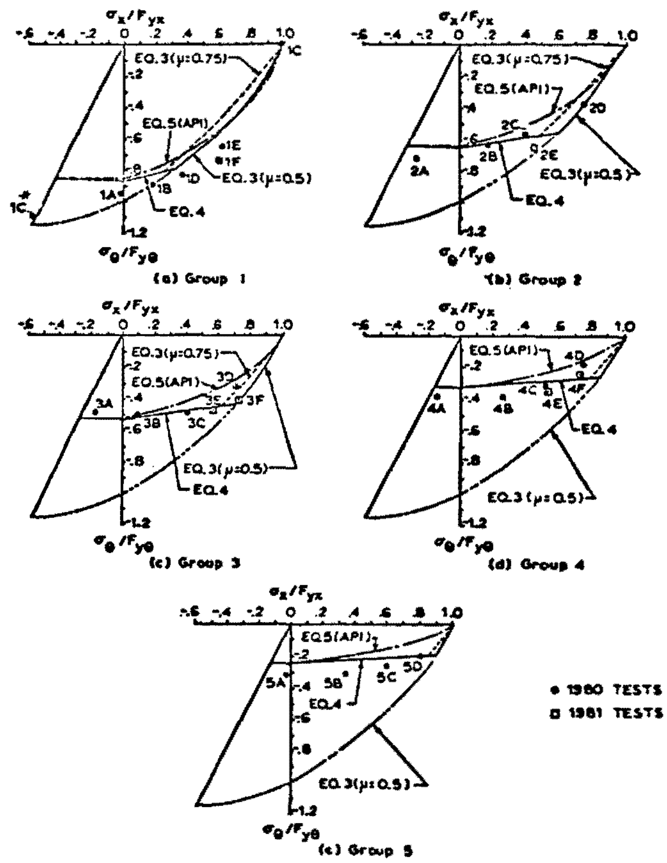


Fig. 2 Comparison Of Test Data From Fabricated Cylinders With Interaction Curves (F_y=36 K.S.I.)

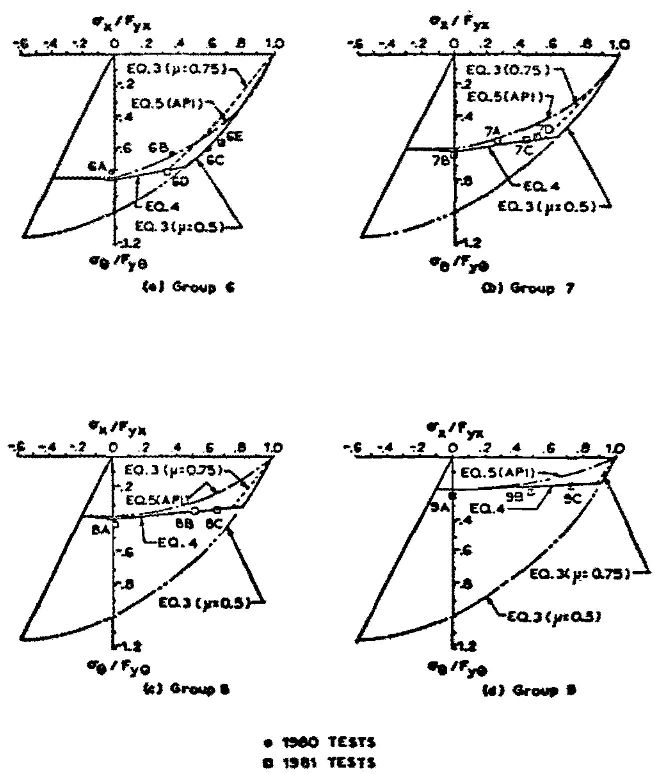


Fig. 3 Comparison Of Test Data From Fabricated Cylinders With Interaction Curves (F_y=50 K.S.I.)

DISCUSSION

P. Marshall (Shell Oil Co.): In 1974 at the annual CRC meeting, I presented a method of dealing with pressure and axial tension by considering a reduced compressive hoop stress. This may help in the troublesome discontinuous region of the curve in Miller's proposals.

P. Frieze (University of Glascow): Why were external (as opposed to internal) stiffener rings used on the test specimens?

Kinra: Because it is too difficult to weld rings inside 18 inch diameter specimens.

D. Faulker (Conoco): I am impressed with the design application you present. If a continuous curve rather than the discontinuous straight line-Von Mises curve had been used, the problem of fitting the design curve to the data may have been reduced.

C. Miller (Chicago Bridge & Iron Co.): The capacity reduction and plasticity knockdown factors used to construct this design curve were drawn from API rules.

S. L. Fu (Gulf Oil): What kind of imperfection levels did you find?

Kinra: Out-of-roundness was 1.5 to 2 times the API allowable. This was adjusted for in our comparisons by using the API equations for members having greater than permissible imperfections.

P A N E L D I S C U S S I O N

STABILITY OF OFFSHORE STRUCTURES

MODERATOR: R. M. Meith, Chevron USA, Inc.

PANELISTS: P. W. Marshall, Shell Oil Company
C. D. Miller, Chicago Bridge and Iron Company
J. R. Lloyd, Exxon Production Research Company

RECORDERS: P. C. Birkemoe, University of Toronto
T. A. Bubenik, Exxon Production Research Company

P. W. MARSHALL - An Overview of Recent Work on Cyclic Inelastic Behavior and System Reliability

There are three legitimate methods to cut the costs of an offshore platform:

(1) Use an inherently efficient concept
(2) Sharpen pencil -- eliminate fat
(3) Optimize cost-risk tradeoffs

The Cerveze platform, which we just saw in the movie, epitomizes methods (1) and (2). I will start from the reliability viewpoint, item (3), and present information on the inelastic behavior of tubular members and structures, which is essential for a realistic assessment of strength and risk. It is also clearly useful in the context of item (2), and may even influence the choice of configuration, item (1).

THE RELIABILITY VIEWPOINT

Our goal here is to minimize the total cost of the structure, which consists of:

initial cost, plus
deferred maintenance and downtime costs, plus
risk cost.

The risk cost is the cost of failure times the probability of failure. To include it in the calculation requires that we estimate the probability of failure. Unfortunately, this is not a precise calculation. The required data base is often incomplete, and the answer we get depends on the assumptions made. Realism requires that we make use of our best understanding of the physical system, including ultimate strength in the case of structures.

As can be seen in FIGURE 1, the optimum point is not sharply defined; thus our calculation of the probability of failure need not be absolutely precise in order to serve its purpose. Furthermore, the reliability viewpoint provides a useful rationale, in that it forces us to examine the bias and uncertainty at each step of the way.

I must add that there are social constraints present which make it unpalatable to make trade-offs between dollars and human safety or pollution risks. Here we make comparisons with other risks which are being accepted by society. The safety index is a useful measure of risk for this purpose, without the legal, social, and psychological impact of probabilities of failure.

We can define the safety index, beta, as the expected value of the margin between real load and real resistance, expressed in units of the standard deviation of total uncertainty. For onshore public structures, beta ranges from 2.5 to 4.0, and failures are so rare that their statistics are not well defined. For new offshore platforms designed for the 100-year storm per API RPZA, the beta ranges from 2 to 3 in terms of the lifetime risk of overload failure; the corresponding average annual loss rate is on the order of 0.1%. This is low enough that overload is not the dominant risk; blowouts, fires, and collisions contribute 0.1 to 0.5%. This may be compared to the 0.4 to 1.3% annual loss rate for mobile rigs. There seems to be a developing public opinion that the latter figures are rather too high for manned platforms; we should be careful that fixed platforms are not tarred and feathered with the same brush.

In FIGURE 2, both load and resistance are defined in terms of their probabilitiy distributions, and formulas are given for the safety index, beta, in lognormal format. The notional probability of failure is then obtained from the standard normal function.

FIGURE 3 shows an example of the kind of data we should be looking at from the probabilistic viewpoint. This is a histogram of the results of 340 tubular joints tested to failure, versus the existing design criteria. The median failure load is biased on the safe side of the nominal ultimate strength, and there is lots of scatter. Clearly, a deterministic interpretation of the nominal ultimate line does not tell the whole story.

WHY INELASTIC ANALYSIS?

Consider the offshore platform shown in FIGURE 4, which is about to take a wave larger than the design condition. Green water will impact on the deck instead of the more transparent supporting structure, resulting in forces well in excess of what the structure was designed for.

Several important elements of the structure are also illustrated:

We have already discussed tubular joints. Their failure is more a problem of strength than one of stability.

Tubular struts provide lateral bracing for the structure. These are designed as beam-columns, with a primary axial load, P, which must be resisted in the presence of bending due to lateral loads, Q, representing locally applied wave forces, bouyancy, and self weight.

At the base of the structure, lateral loads are resisted by soil pressures, transmitted by the tubular piling. These laterally loaded piles may be visualized as portal beam-columns, subject to bending due to the primary shear loads, Q, in the presence of axial loads, P. These have a two-hinge failure mechanism as shown.

Please keep these definitions of strut and portal in mind, as we will be using them later to examine the inelastic behavior of tubular elements.

FIGURE 5 shows this same problem in probabilistic format. The horizontal scale is units of horizontal force, and represents the combined effect of wind, wave, and current acting in their various patterns and directions.

The design load is that due to the 100-year storm. When this is compared to the probability distribution of lifetime extreme storm loads, we find that in a 20-year lifetime there is roughly a 20% chance of exceeding the design load (the precise figure is 0.18127). This random chance of occurrence is represented by the dashed probability density bell curve.

Systematic uncertainties are also present in the applied loads. Differences between hindcast and observed waves sometimes approach legendary proportions. Even given the wave height, the comparison between measured and calculated wave forces may be described as a shotgun plot. Including these uncertainties spreads out the underlying distribution and raises the once-in-a-lifetime extreme tail, as shown by the solid curve. The amount of bias and uncertainty to be introduced at this stage depends on the details of how the nominal design forces are calculated, and on the data base assumed to apply.

On the resistance side of the picture, we see that structures designed to 80% of yield (AISC plus 1/3) have a nominal resistance of 1.25 times the design load, at first yield. If we consider the ultimate strength of struts as beam-columns, an additional factor of 1.22 is gained. In a highly redundant structural system, with parallel members able to take additional load after the first one yields, developing a structural failure mechanism requires additional loads, corresponding to a system reserve strength factor of 1.17. Combining all these factors, which come from recent typical examples discussed later in the paper, results in a nominal failure load of 1.8 times the design storm load. If we now consider the bias and uncertainty in yield strength, tubular joints, beam-column behavior, etc., we get the dashed "a priori" distribution, which indicates a median strength a bit over twice the design load.

This "a priori" resistance can be combined with the final (solid) loads distribution to obtain the safety index, beta, and the notional probability of failure. Unfortunately, when this is done for older platforms which have survived storms well in excess of what they were designed for, the indicated probabilities of failure are unrealistically high. However, this survival experience can be utilized to perform a Bayesian updating, which results in the solid strength distribution. The two solid distributions of load and strength can now be used as a more realistic basis for the probalistic calculations needed for optimizing the cost-risk trade-offs. Again it is necessary to add the cautionary note that the additional resistance indicated by this calibration is not universally applicable; we could just as well have used the Bayesian calibration to adjust the loads distribution downward.

There are a surprising number of references in the literature which present so-called risk analyses using the dashed loads distribution and the deterministic nominal resistance. These are not complete analyses. They prefer to look only at the probabilities of random events, for which there is rigorous mathematical theory -- while turning a blind eye to uncertainties and bias in the physical system, which are admittedly messier to handle. The lifetime probability of failure is NOT the 20% probability of exceeding the design storm, but generally a much lower number due to the reserve strength and safe-side bias present. We sometimes have difficulty with environmental scientists who only look at their part of the problem when attempting to set design criteria. On the other hand, we structural engineers must be careful to avoid becoming too wrapped up with just the resistance side of the picture.

With minor exceptions, when the lateral loads from wind, wave and current exceed the ultimate load capacity of the structure, then collapse ensues because the applied forces keep on coming. For earthquakes, however, it is quite a different story. Earthquake FORCES are not present in nature, but are induced by the structural response to ground MOTIONS. In earthquakes, structures can exceed the elastic limit energy by factors of four or more without collapse. Yielding may actually be helpful in limiting the applied forces, and ductility becomes as important as strength in measuring the performance capability.

Consider a sudden ground displacement of several feet -- capital delta in FIGURE 6. The heavy deck mass essentially gets left behind, leading to the structural distortion shown. There is also some yielding and damage indicated: buckled compression braces, a fractured tension brace, and portal mechanisms in the superstructure legs and in the piling. Yet there is no collapse if the structure remains stable in the damaged condition.

Elastic dynamic analyses may be performed for "strength level" earthquake motions having 0.25g peak ground acceleration and 7 to 13 in/sec peak velocity. Yet structures may be called upon to survive motions like San Fernando's 46 in/sec, or 50 to 60 in/sec in an Alaskan earthquake. In these earthquakes, a large velocity pulse, corresponding to a sudden ground displacement, is responsible for much of the damage. Clearly, we are depending on the inelastic reserve in order to survive these events.

Earthquakes may also be associated with ground distortions (lower case delta in the figure) of several feet. These can produce scary elastic stresses in piling, oilwell casings, and pipelines. Yet, if the ductility is there, the displacements can safely be accomodated in the plastic range.

FIGURE 7 shows a platform designed for mudslides. There are large areas of very soft, recent sediments around the Mississippi delta. In hurricanes, the bottom pressure from large waves can plastically deform the sea floor sediments, inducing back and forth motions in the soil column. Net downslope movements of tens or hundreds of feet can accumulate in slide areas measuring several miles across, with depth of sliding reaching 50 to 200 feet. Generally, platforms must be designed to resist the full force of the flowing soil mass, unless the movements are only a few feet and can be accomodated by inelastic deformation. The figure illustrates a mudslide platform with large piling or caissons, 8 to 20 feet in diameter which function as a two-hinge portal mechanism. Smaller piling and conductors may survive by forming a three-hinge mechanism, but would not contribute to the gross lateral resistance of the structure in this mode. In view of the uncertainty in applied soil forces, inelastic behavior and reserve strength are of concern, particularly for the large D/t caissons under cyclic loading.

FIGURE 8 illustrates a new type of compliant platform, in which large deflection behavior, including the P-delta effect, is important. The structure experiences a quasi static tilt due to wind, current, and average wave forces, as well as a smaller oscillation due to cyclic wave forces. Tubular members are used for oilwell conductors, and sometimes for vertical piles to support the tower. These are designed to remain elastic for expected conditions, limiting the tilt to 3-5 degrees. Because current, long term drift oscillations, and very low frequency wind spectra are not as well understood as wave forces on conventional structures, there is some risk of excess tilt occurring. However, this is not catastrophic if the piles and conductors can form plastic hinges without local buckling or fatigue fractures.

It is feasible to design platforms to resist low velocity collisions from small vessels, as occasionally result from maneuvering errors of supply boats. See FIGURE 9. Here we want to be sure the structure does not collapse, even though it may sustain damage. Plastic bending, buckling, and crushing of tubular members is of interest here, as well as the residual strength of the structure with damaged or missing members.

I hope the foregoing examples have served to answer the question of "Why inelastic analysis?" It offers exciting possibilities in several important applications. A realistic understanding of the ultimate strength of structures is a crucial part of assessing their reliability.

INELASTIC BEHAVIOR OF TUBULAR SECTIONS

Let us first consider the behavior in bending. For very stocky sections, we do not have to worry about local buckling. The moment-curvature (M-phi) behavior is shown in FIGURE 10. It is fairly linear up to the yield moment (FyxS). A modest amount of plastic curvature brings us to the fully plastic moment (FyxZ). With strain hardening, ultimate tensile failure is reached at a moment (FuxZ) of about twice the yield moment, and curvatures beyond the range of most practical applications.

The behavior of compact sections is illustrated in FIGURE 11. These can reach the fully plastic moment -- and, beyond this, possess sufficient rotation capacity to redistribute moments and form a plastic mechanism. The mode of section failure is plastic collapse, not classical buckling. The tension-compression couple combined with large curvature act to cause the flattening shown.

The upper D/t limit for this type of behavior depends on the kind of loading. It is about 50 for combinations of tension, bending, and hydrostatic pressure, as encountered in deep water pipelaying operations. It is lower for combinations of compression, bending, and shear -- e.g. beam-columns as used in structures. Hydrostatic pressure severely reduces the bending performance. Under certain conditions, the collapse can propagate for beyond the region of severe loading which initiated it -- the so called propagating buckle.

Bending behavior of semi-compact tubular sections is shown in FIGURE 12. These can still develop the full plastic moment. However, only limited curvature and rotation capacity is exhibited, before local buckling leads to a fairly rapid degradation of capacity to about half the peak. The applicability of plastic design requires a detailed analysis which considers this degradation. The buckle can be outwards, as shown, or inwards. Filling the member with grout is not particularly effective in suppressing the outward buckle.

For members which fail in the plastic buckling range, FIGURE 13, the bending strength is somewhere between yield and fully plastic, with essentially negligible plastic rotation capacity. The upper D/t limit for this class is 190 per the API design equations, and about 100 at the experimentally observed onset of local buckling.

For members which fail in the elastic buckling range, FIGURE 14, the capacity is less than yield, and very sensitive to imperfections. There is a very sudden, catastrophic drop at the onset of local buckling, to perhaps 1/5 of the peak capacity. The classical diamond pattern of buckling may be observed.

FIGURE 14 shows the strength interaction between axial load P and bending moment M for tubular sections. The fully plastic interaction is given by equivalent cosine and arc sine expressions. The latter is comparable in form to the AISC utilization ratio, which corresponds to first yield interaction, except that there is about 30% additional capacity for combinations dominated by bending.

For detailed analysis of the inelastic behavior of beam-columns, the plastic deformation of tubular sections may be described by the moment-thrust-curvature relations (M-P-phi curves) of FIGURE 15. Note that residual stresses cause early departure from linear behavior, consistent with the observed buckling strength of centrally loaded columns.

A more general representation of the inelastic behavior requires four parameters: moment, thrust, curvature, and axial deformation. Behavior is described in terms of an interaction surface and a flow rule. The added dimemsion is important in understanding the behavior of struts, particularly the phenomenon of column growth which occurs during cyclic buckling and straightening.

INELASTIC BEHAVIOR OF TUBULAR MEMBERS

The foregoing M-P-phi curves have been used with a 20-segment inelastic beam-column model to study the ultimate strength behavior of struts. Recall that these are members carrying primarily axial load, in the presence of lateral loads due to wave force, self weight, bouyancy, local acceleration, etc. Typical results are shown as the solid lines in FIGURE 16. These are closely followed by the arc sine equation shown in the figure, and indicate ultimate capacities well in excess of API and AISC first yield criteria. For typical struts with mostly axial load, and L/D in the range of 20 to 50, the capacity is 20-25% greater. Where there is more bending, the difference is even more dramatic.

Prof. Sherman has conducted a series of over 100 tests of strut and portal type beam-columns. His results are compared with the arc sine ultimate strength equation in FIGURE 17. The proposed equation is generally conservative, except for the "D" series and the dark points. The "D" series has tubes with D/t of about 80 which failed by sudden local buckling, before achieving a plastic mechanism. The dark points are as-received manufactured tubes with yield strength in excess of 50 ksi, a rounded stress strain curve, and a low UTS/yield ratio -- atypical of larger fabricated tubes used in offshore platforms.

FIGURE 18 shows the same results in probalistic format. 95% of the struts failed at 1.2 to 2.0 times the API design load. The bias and scatter of Sherman's test results relative to the arc sine ultimate strength formula is similar to that of the old CRC formula relative to its data base (the dotted line). Including the material variability slightly increases the safe-side bias and the scatter.

Now let us look beyond the peak ultimate strength of struts, to their axial load-deformation behavior. In FIGURE 19, load and deformation are normalized on their values at peak capacity, assuming elastic behavior up to that point. Curves 1, 2, 3, 4, 5 denote increasing lateral load, Q, of 0, 10, 20, 40, 80% of yield, respectively.

The B series represents the base case, with kL/r of about 30 and D/t of about 48. These exhibit a moderately short yield plateau before column and local buckling combine to degrade the load capacity.

The A series has a more compact D/t of about 36. It shows a very long plateau for the case of zero axial load, in which the strut shortens plastically. However, even a modest amount of lateral load and bending causes the behavior to revert to the base case.

The C series has a longer kL/r of about 75. These show essentially no yield plateau before buckling degrades the load capacity. However, some useful ductility is created by the slight rounding of the load-deformation curve which occurs on the way up to peak load. An exception is test #5, which exhibits the very ductile behavior of what is primarily a beam mechanism.

Finally, the D series, which has a D/t of about 80, show essentially catastrophic failures. These results were excluded from the probabilistic analysis of the preceeding figure.

FIGURE 20 repeats the picture of cyclic inelastic behavior of struts which was presented at the 1976 CRC Meeting. Although this is now considered a Model "A" antique, the physical processes at work remain the same. In sequence, we have (a) elastic loading of a straight member, (b) decreasing axial capacity as the member assumes a post buckling shape, (c) elastic unloading of the bent member, (d) inelastic straightening, and (e) tensile stretching. Hysteresis loop (f) (g) (h) (i) repeats the process in a position displaced by tensile stretching. Loop (j) (k) (l) shows the decreased load capacity of a bent member. Not shown is the further loss in capacity which comes from local buckling of the section.

This is a one degree of freedom strut algorithm useful for expediting numerical analysis of large structural systems. It has been called a phenomenological model, in that it attempts to describe directly the element behavior as we know it from experiments and from more complex analytical models. FIGURE 21 shows some of the recent experimental results which have led to major revisions to the algorithm.

Firstly, the shape of the hysteresis loop is seen to depend on kL/r, and to a lesser extent on D/t. There is a significant ductile plateau, over which peak load is maintained, for struts with low kL/r and low D/t, as well as "fat" hysteresis loops. The sharply peaked, pinched loops of the original model are observed only for struts with high kL/r or high D/t.

Secondly, the virgin compressive peak load does not repeat on subsequent cycles. The rate of degradation in peak load is faster at larger imposed excursions (i.e. larger ductility ratios).

Finally, there is the phenomenon of column growth. The hysteresis loops do not return to yield tension without additional extension.

FIGURE 22 shows the Model "B" strut algorithm which includes the plateau and cyclic degradation of peak load. The tabulated values give the length of the plateau (or ductility prior to degradation), which controls the shape of the hysteresis loop along with Fn, Cl, and C2. Tabulated values of R2 describe the degradation of peak capacity as a function of imposed ductility.

More complicated Maison and Zayas phenomenological models, with up to 14 empirical parameters, have also been developed. These include the column growth phenomenon.

FIGURE 23 shows the cyclic behavior of portal beam-columns. The plots show shear load versus imposed lateral displacement, in the presence of axial load, with the results scaled up to prototype magnitudes. The results shown are for Sherman's "B" series with D/t of 48 and kL/r of about 30.

In test PBl, with zero axial load, local buckles form and straighten out on each cycle, leading to fatigue cracks and failure after 60 cycles. Failure is defined here as the capacity being reduced to 10% of that on the original cycle.

In test PB3, the lateral displacements and plastic bending are imposed in the presence of an axial load of 32% of yield. With this compressive bias, the buckles never straighten out, but grow progressively on each cycle, leading to failure after 8 cycles.

In test PB4, the axial load of 48% of yield dominates over the imposed bending. Failure occurs after only twc cycles, with the rather sudden formation of a buckle which completely encircles the tube.

For most of the tests, however, the failure is gradual rather than instantaneous. Deterioration is observed to be a function of accumulated plastic rotation of energy. There also seems to be a threshold plastic strain below which the cycles are not damaging.

FIGURE 24 shows a modified NBEM portal beam-column element which models this behavior with an exponential decay function. Here we see the model being used to hindcast two of the tests. Test PB3 at an imposed ductility of 4 failed in 3 cycles, while test PB3 at a ductility of 8 failed in 1-1/4 cycles. This phenomenological model mimics the experiment reasonably well.

OVERLOAD BEHAVIOR OF TUBULAR STRUCTURES

The massive computer program INTRA was developed as the "ultimate weapon" for nonlinear static and dynamic analysis of tubular structures. We have been discussing the elements STRT-B to model braces and NBEM to model platform legs and piling. As can be seen in FIGURE 25, the program has lots of other goodies, much of which are specialized to treat pile-soil interaction during earthquakes. As it presently stands, this elaborate theoretical tool is still in need of calibration against benchmark experimental data.

FIGURE 26 shows a model structure tested at Berkeley. It represents a Southern California design at 1/6 scale. The lower part of the figure shows the lateral displacements which were imposed on the structure, for which its inelastic cyclic response was measured. This model has bracing D/t ratios of about 48; a second model was also tested with D/t ratios of about 36.

FIGURE 27 gives a general picture of the plastic mechasisms which formed on the structure during the test. Combined column and local buckling occurred first in the upper X-braces; these were eventually severed due to low-cycle fatigue in the folds of the local buckles.

Experimental hysteresis loops for the test structure are shown in the top of FIGURE 28 as total frame load versus deflection at the top of the structure (point of load application). These show the deteriorating strength and stiffness which follow local buckling and eventual failure of the bracing members. However, portal action of the legs continues to maintain stability of the structure, and gives it considerable residual strength.

The lower part of the figure is a plot of frame load at peak frame displacement, with both experimental results and those from the INTRA hindcast. The test results are lower in the early cycles due to the rounded load-displacement behavior observed while loading, whereas INTRA assumes linear behavior up to element capacity. However, INTRA does correctly reproduce the deterioration strength and stiffness in the later cycles, where the peak loads fall off despite ever increasing excursions. These comparisons provide both reassurance that INTRA gives reasonable results, and cautions that it is not exact, even with the updated strut algorithm.

FIGURE 29 schematically shows the testing arrangements for another valuable set of benchmark data, in which the structure is loaded under actual dynamic conditions on a shaking table. This is a smaller version of the same frame tested earlier under imposed displacements. The following scaled motions were applied:

Strength Level -- 0.28g prototype
Ductility Level -- 0.56g prototype
Maximum Incredible -- 1.23g prototype

The latter produced damage similar to that observed in the earlier model. It was followed by two Strength Level aftershocks to study progressive deterioration. The data from this experiment have yet to be hindcast analytically, although there is an API project in the works to do this.

I will now briefly dicuss results from a couple of practical applications of INTRA. Shown in FIGURE 30 is the initial design for an oil production platform in Southern California, after a push-over analysis which was performed for the API ductility check. The failure mechanism shown identified an unexpected weak link in the structure. The desired plastic mechanism was for the two compression braces and the two tension braces at each level in the structure to working at capacity. However, failure of horizontal brace #30 permitted a compression-only failure to jump from level to level, even though this member was not highly stressed in the elastic analysis. Beefing up this horizontal led to the desired mechanism, with more energy being absorbed prior to any loss of capacity.

FIGURE 31 shows the results of an inelastic time history dynamic analysis of the same structure. The input ground motion is the Pacoima accelogram amplified by deep layers of unconsolidated sediments at the site. Excursions of 40 inches at the mudline, and 70 inches at deck level are seen. Clearly, this is not your run-of-the-mill Strength Level jiggle; this is the BIG ONE.

A plot of yielding and structural damage after this event is shown in FIGURE 32. We see that all piles have yielded at the mudline. A couple of braces just barely yielded, one in compression and one in tension. One brace has gone well past buckling, and used up 26% of its available energy to complete severance; it would be in need of repair. However, the structure as a whole was nowhere near collapse.

FIGURE 33 shows the results of a wave overload analysis of an old Gulf of Mexico structure, subjected to today's higher design wave. Wave forces were applied as a static load set, and increased proportionately until failure occurred. The failure mechanism for broadside waves involves one compression diagonal in each of the four trusses. Inelastic load redistribution from compression members to tension members does not occur, due to the omission of horizontal bracing between the top two bays. Nevertheless, the load at failure was 1.4 times the load at first yield according to conventional elastic design criteria.

We also looked at the structure's performance in the combined wind, wave, and current forces hindcast for the site during Hurricane Frederick. Using the original Model "A" strut algorithm, failure occurred at a load factor of 0.66 with a ratcheting type failure being triggered as the first strut passed its sharply peaked ultimate load. This type of behavior is indicated by the dashed line, and required great ingenuity on the part of the analyst in order to maintain numerical stability. Using the modified Model "B" strut algorithm, failure occurred at a load factor as 0.73. This 11% improvement is the result of the buckling plateau allowing the most highly stressed brace to maintain its peak load while the others catch up.

Incidentally, the structure is still there, and recent underwater inspection has shown no damage due to overload. There must be some hidden conservatism remaining in our methods of analysis. Since we seem to have wrought all we can out resistance side of the problem, perhaps it is time to re-examine the loading side. The recent trend towards higher design waves, large currents, and increased drag coefficients, when taken all together, somehow seems to fly in the face of experience. Survival of the older platforms continues to provide a valuable basis for calibration, whether this is carried out using Bayesian statistics or on a less rigorous common sense basis.

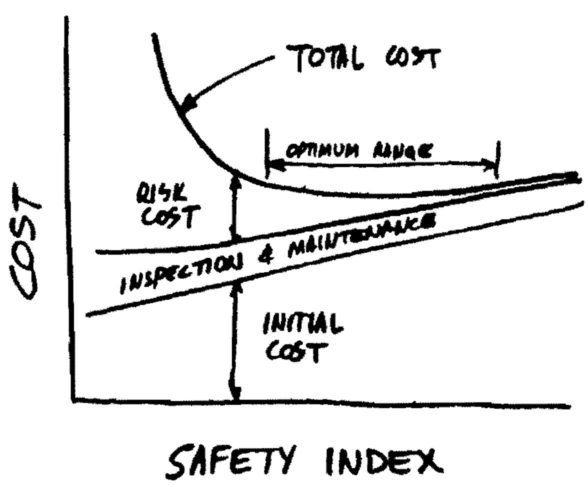
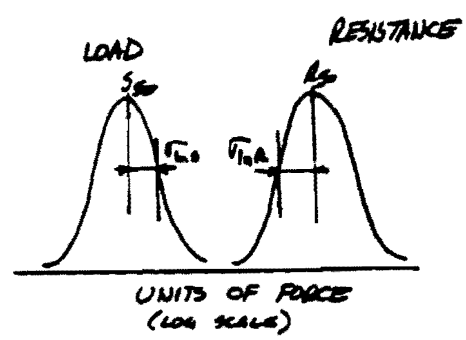


FIGURE 1



$$\beta = \frac{\ln\left(\frac{R_{50}}{S_{50}}\right)}{\sqrt{(\sigma_{\ln R})^2 + (\sigma_{\ln S})^2}}$$

$$pr\{F\} = \Phi(\beta)$$

FIGURE 2

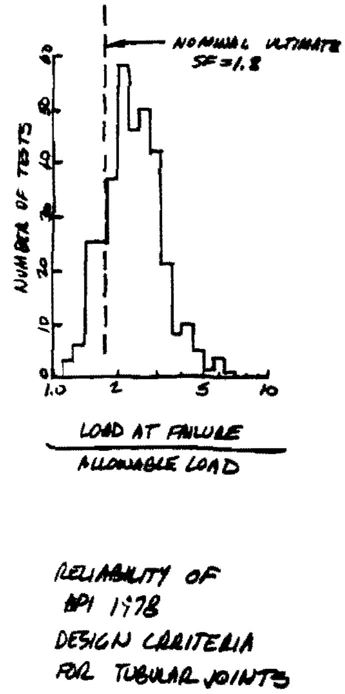


FIGURE 3

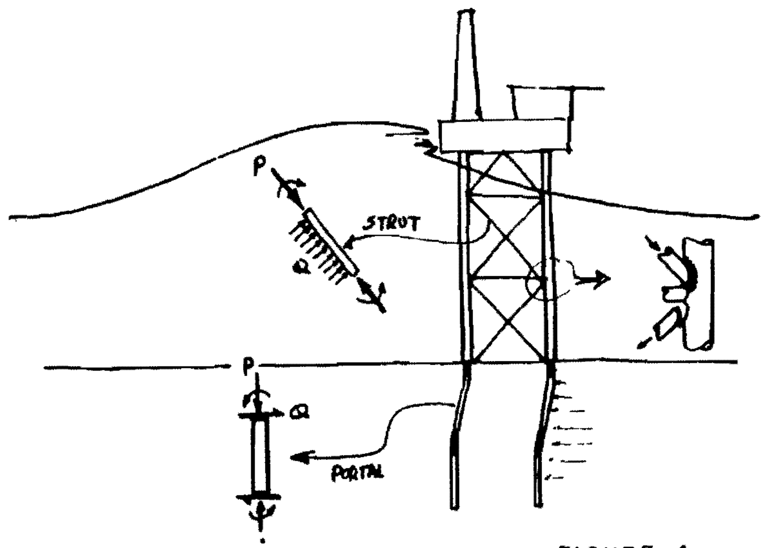


FIGURE 4

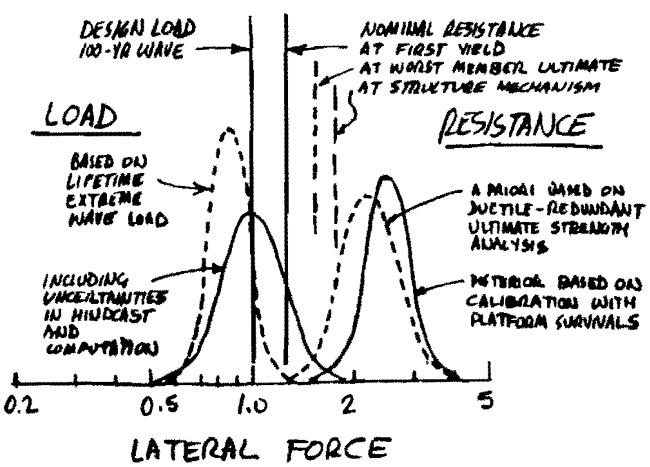


FIGURE 5

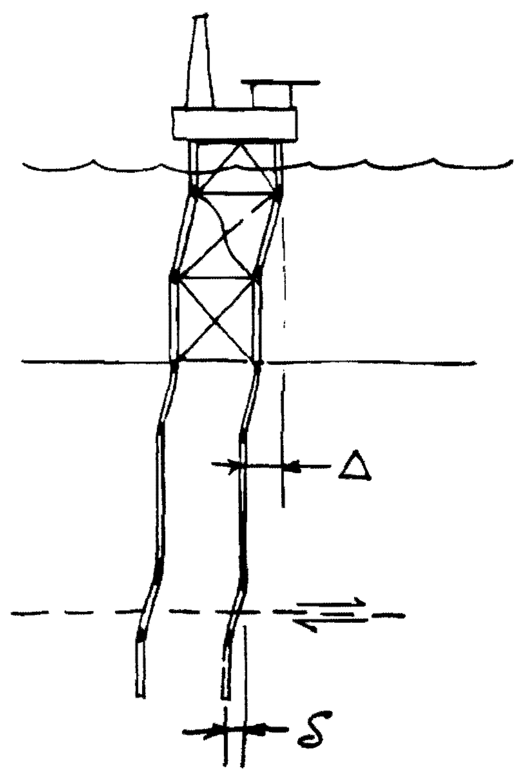


FIGURE 6

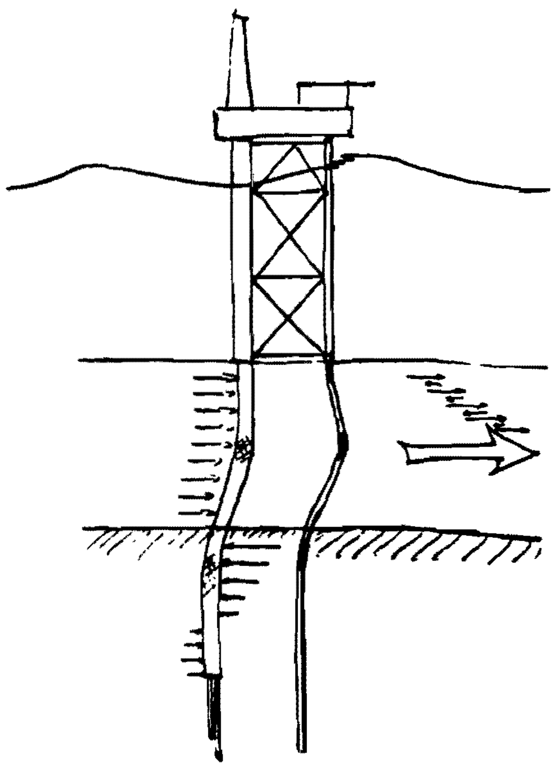


FIGURE 7

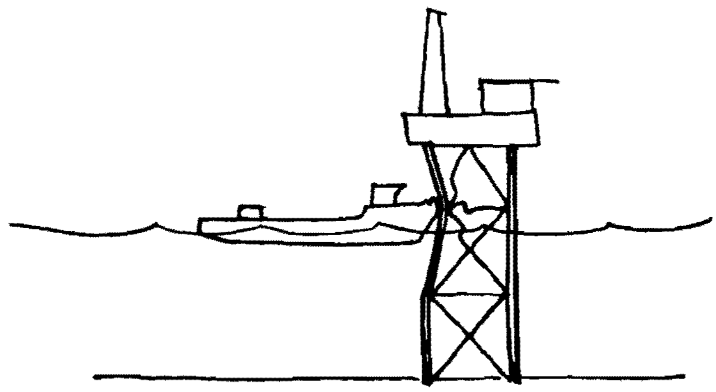


FIGURE 9

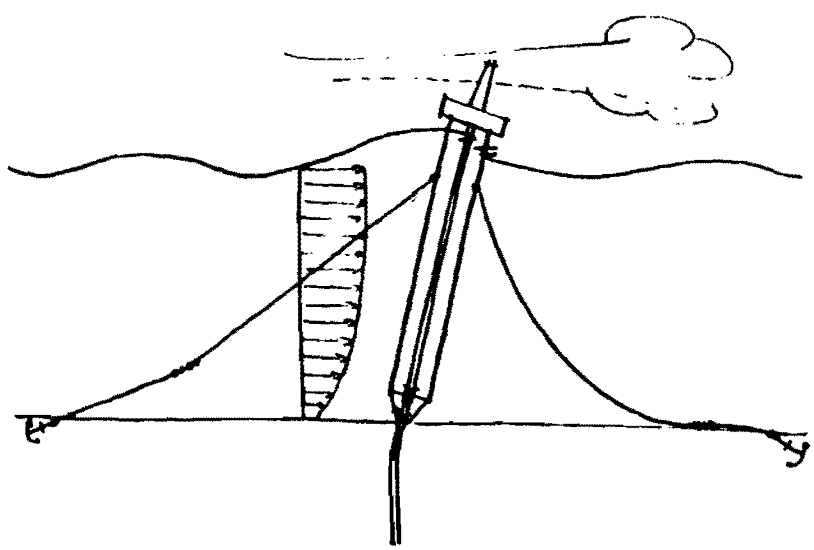


FIGURE 8

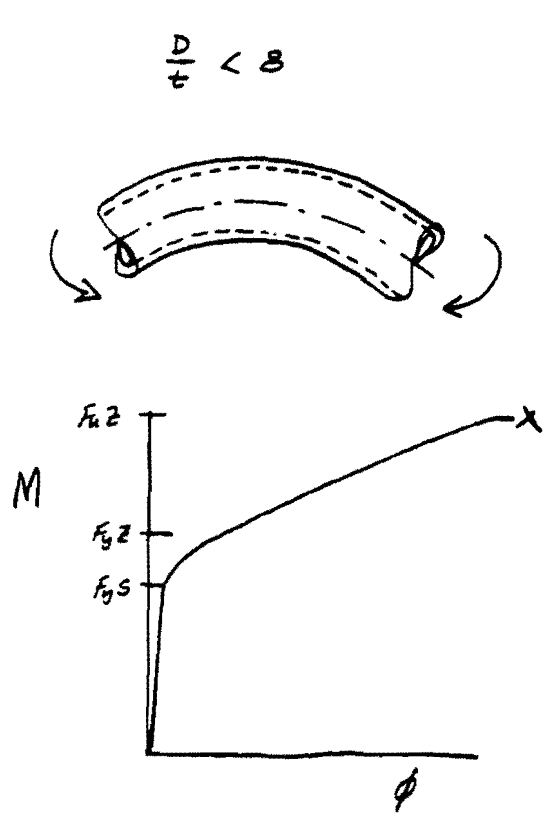


FIGURE 10

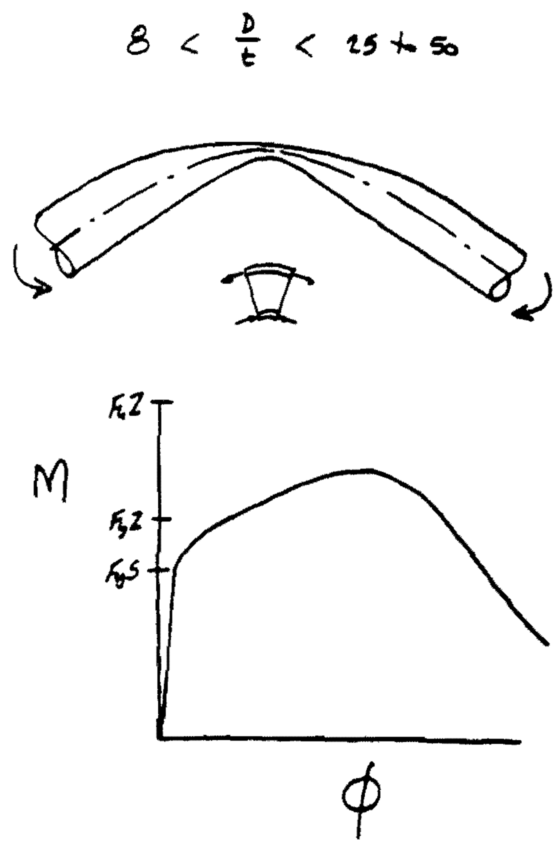


FIGURE 11

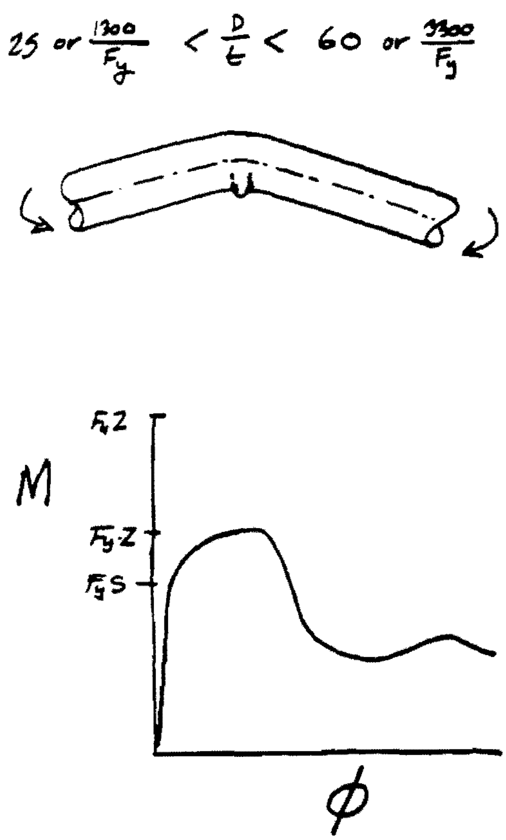


FIGURE 12

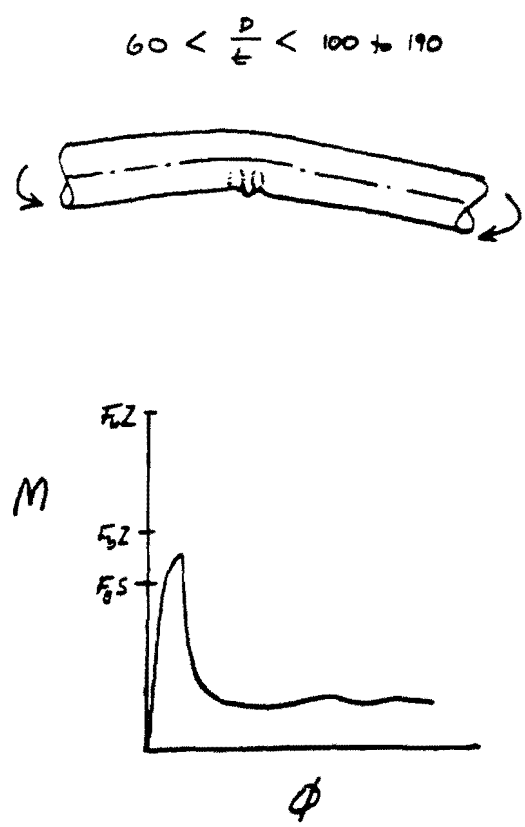


FIGURE 13

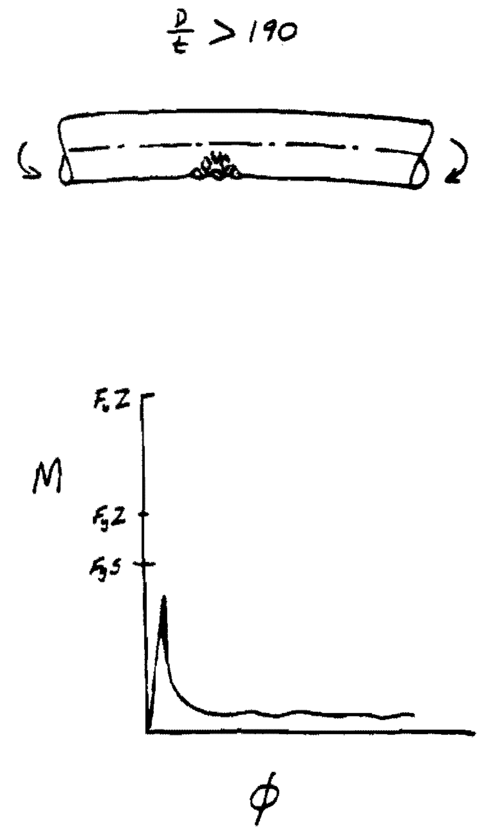


FIGURE 14A

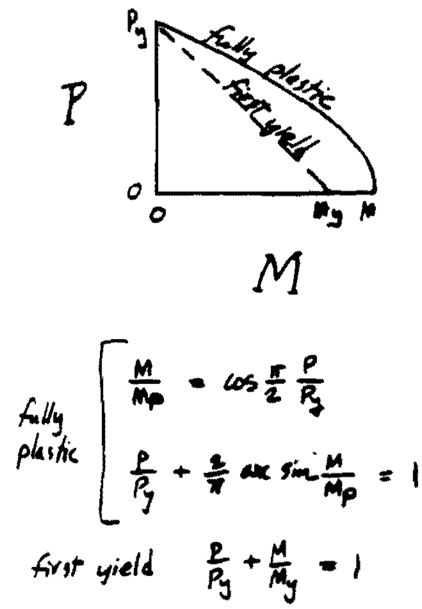


FIGURE 14B

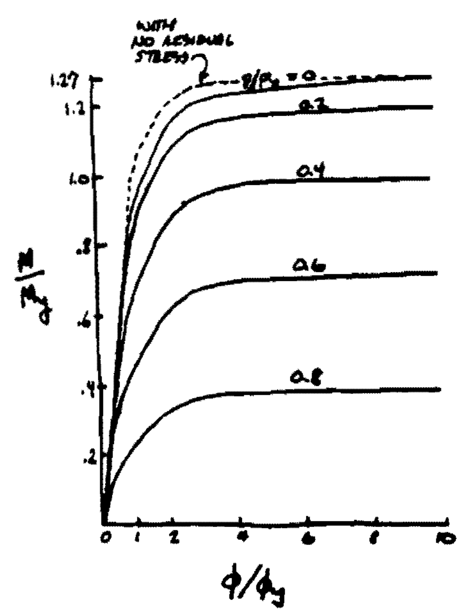


FIGURE 15

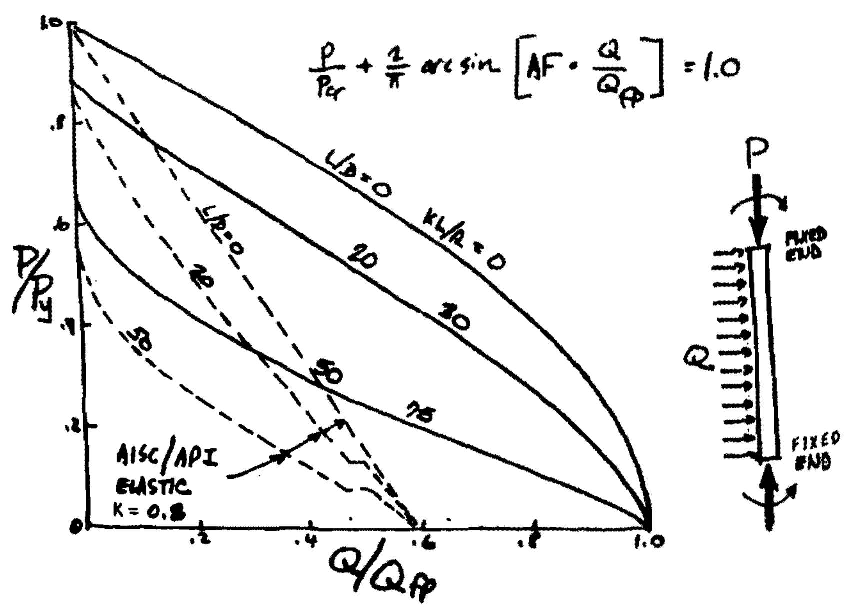
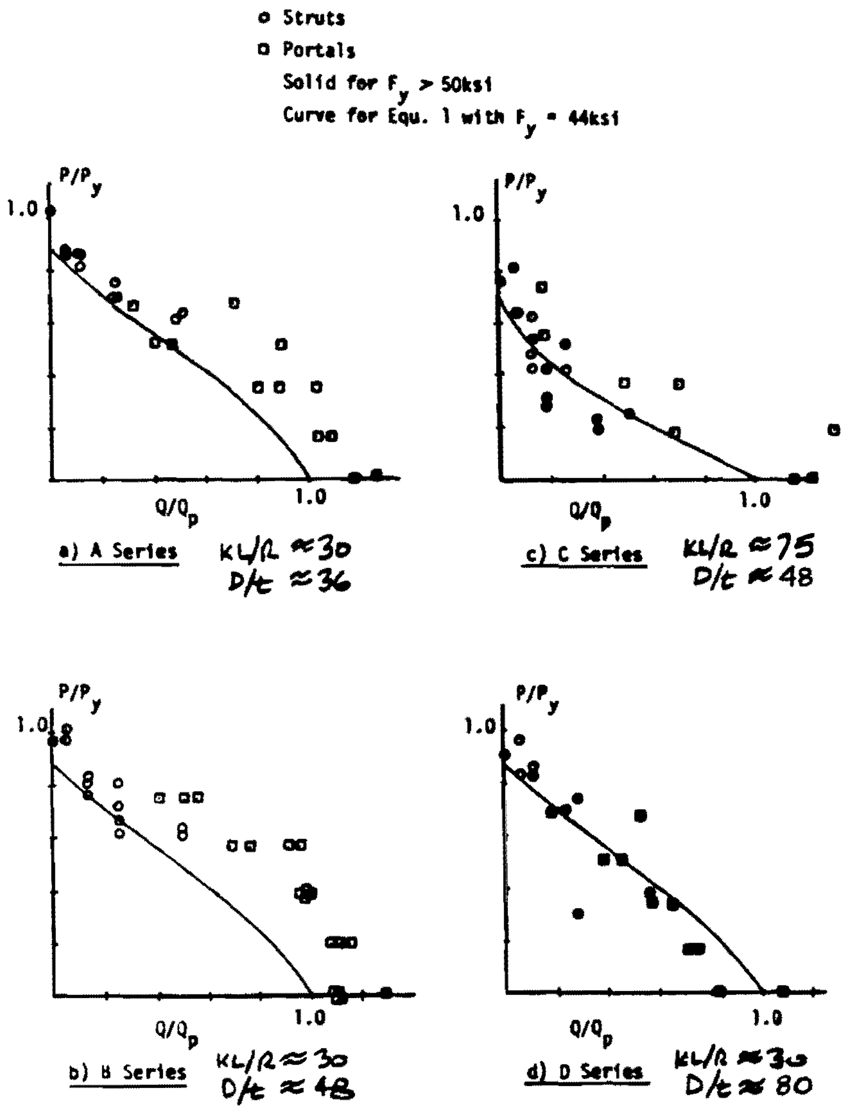


FIGURE 16



STRENGTH DATA ON INTERACTION PLOTS

FIGURE 17

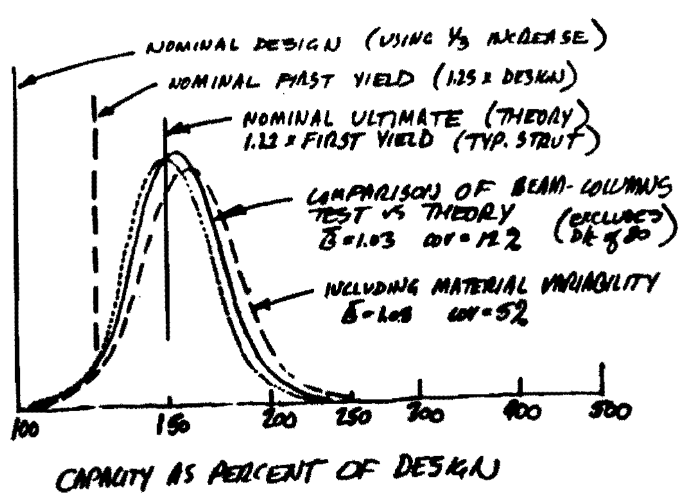


FIGURE 18

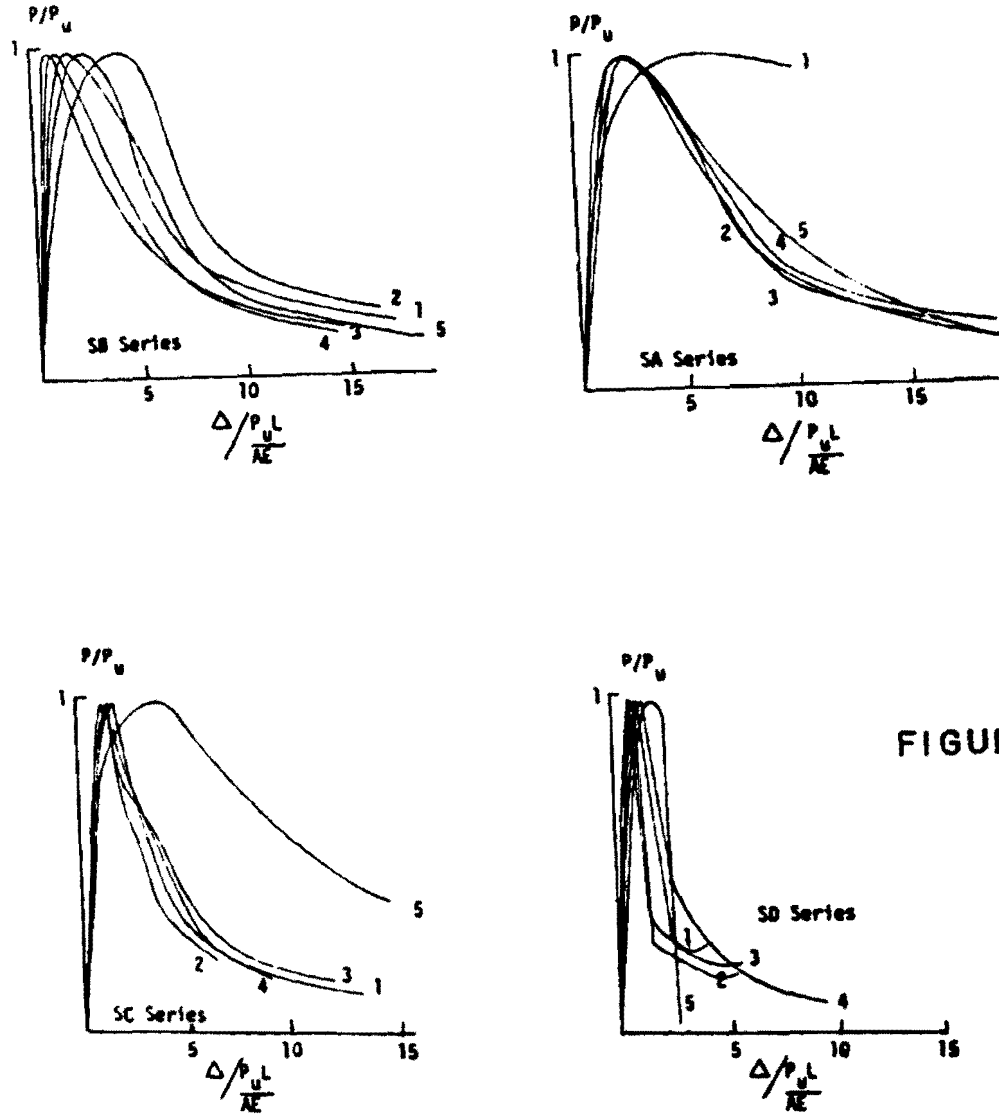


FIGURE 19

NORMALIZED AXIAL LOAD-DEFLECTION CURVES FOR STRUT TESTS

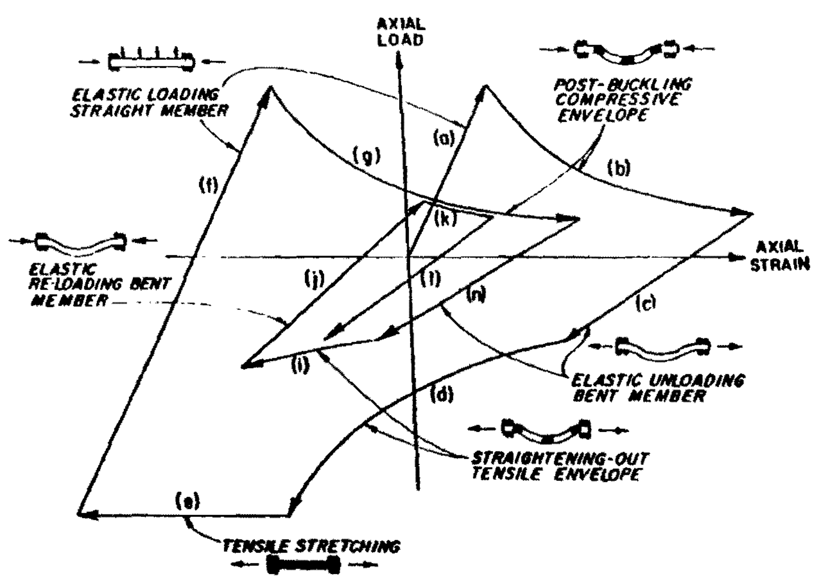


FIGURE 20

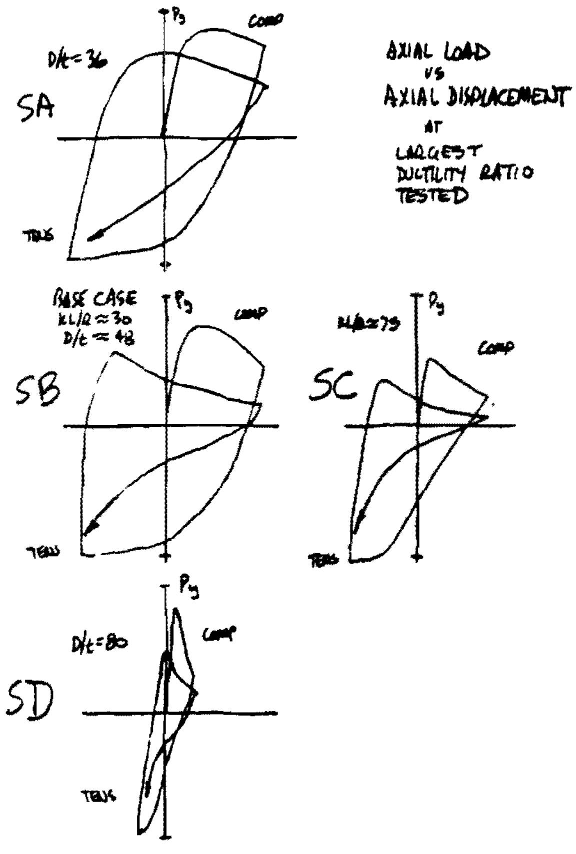
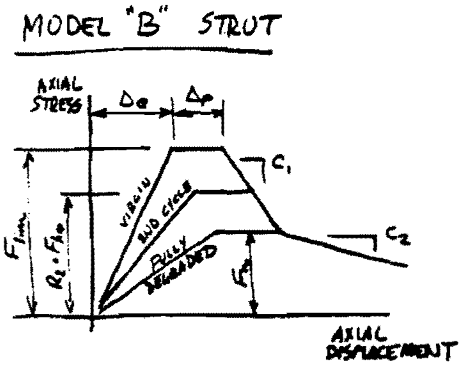


FIGURE 21



SERIES	SA	SB	SC	SD
APPROX KL/R	30	30	75	30
APPROX D/t	36	48	48	80
PLATEAU Δp/Δe	1.6	1.1	0.5	0.4
DEGRADATION RATIO R_2				
FOR $\mu = 2$		.97	.83	.44
FOR $\mu = 4$	.86	.81	.69	
FOR $\mu = 8$	.75	.74	.68	

FIGURE 22

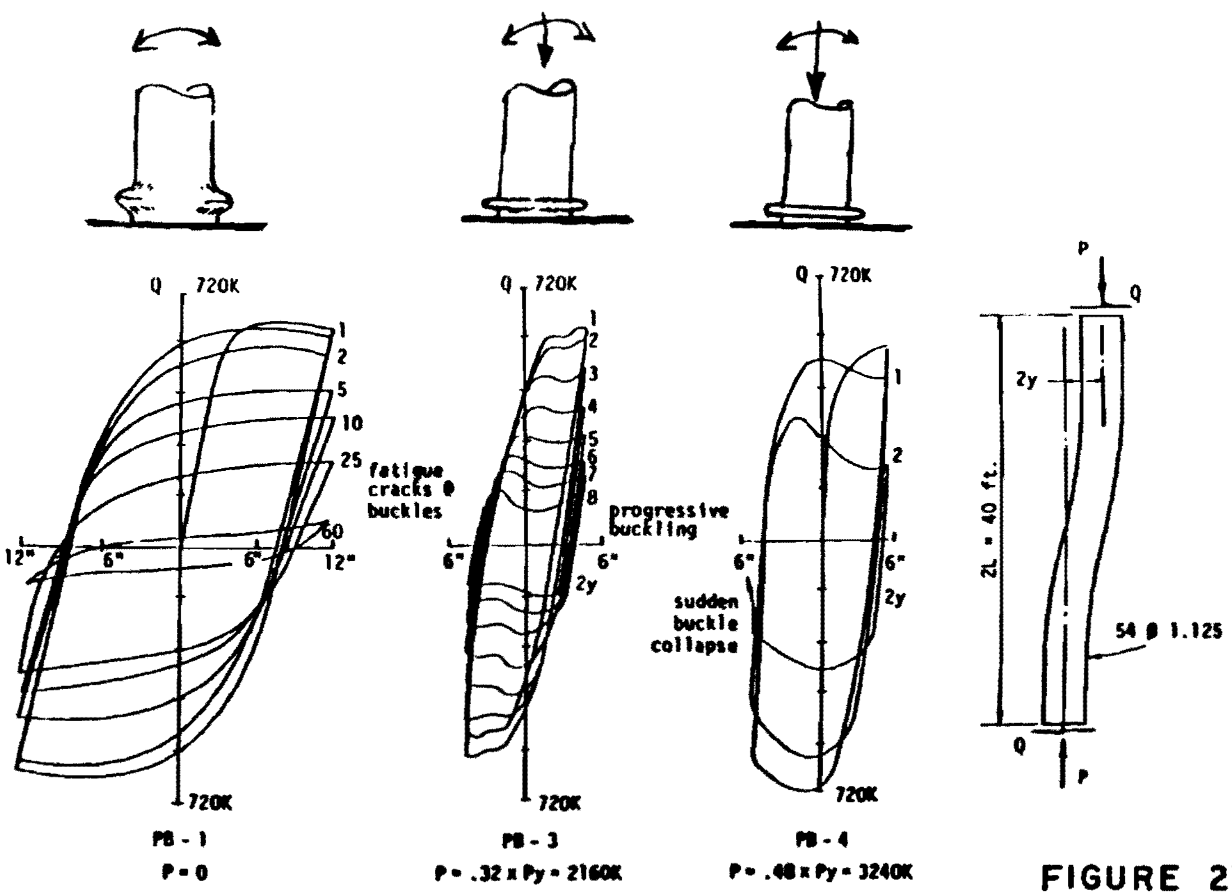


FIGURE 23

CYCLIC SHEAR FOR PROTOTYPE PORTAL BEAM COLUMN

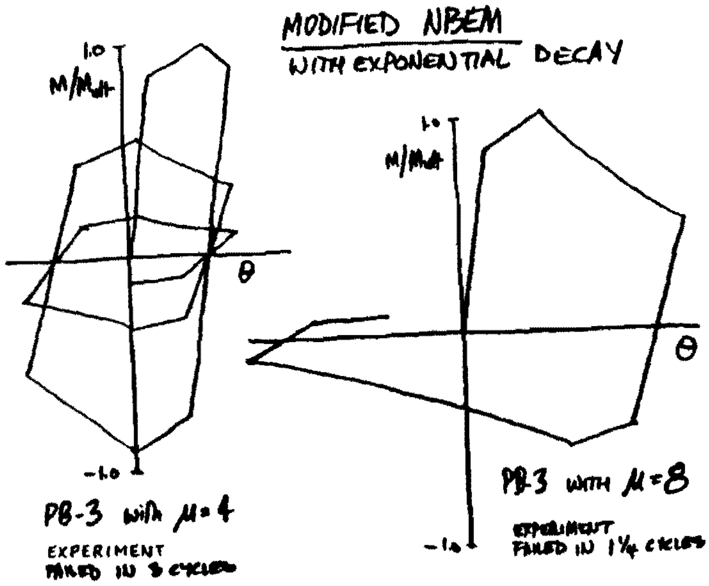
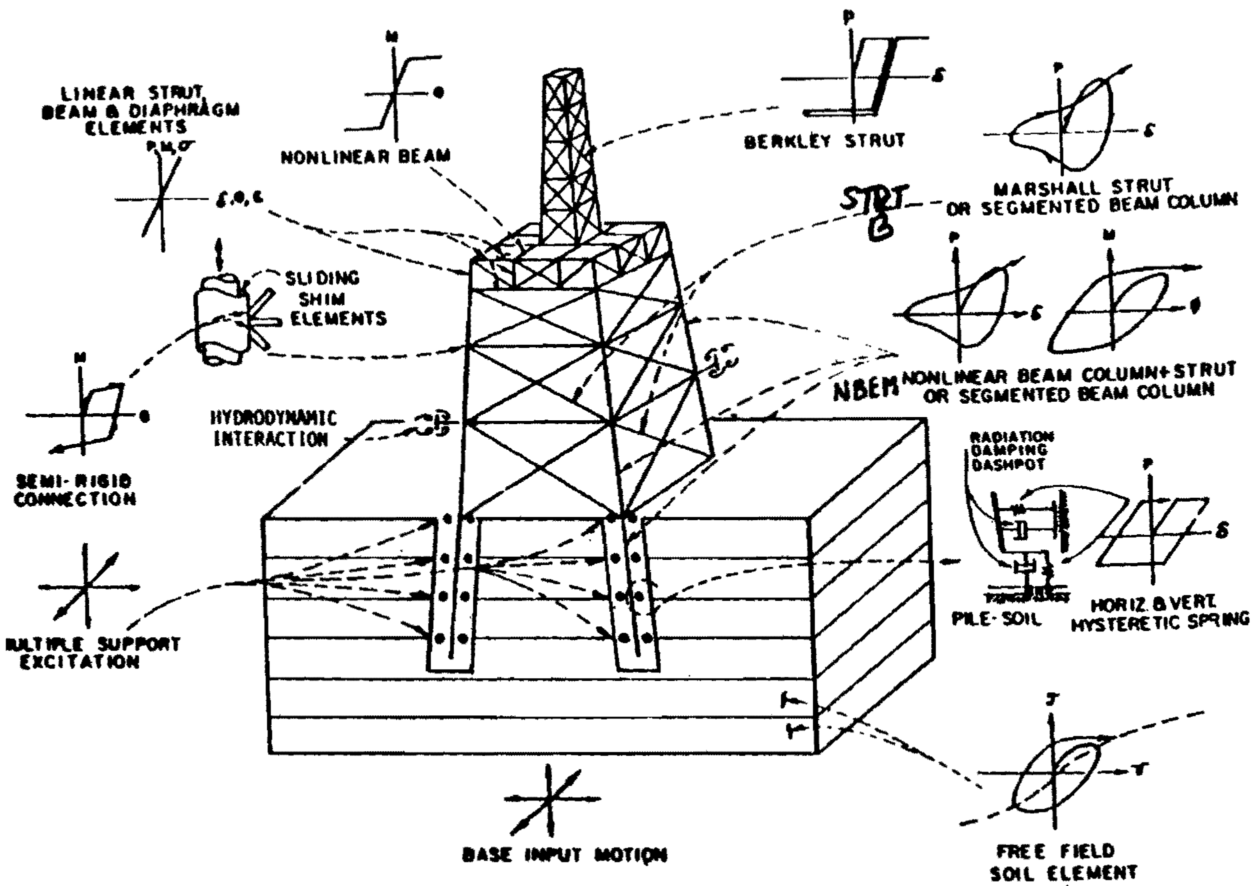


FIGURE 24



FINITE ELEMENT MODEL OF SOIL-PILE-STRUCTURE SYSTEM

FIGURE 25

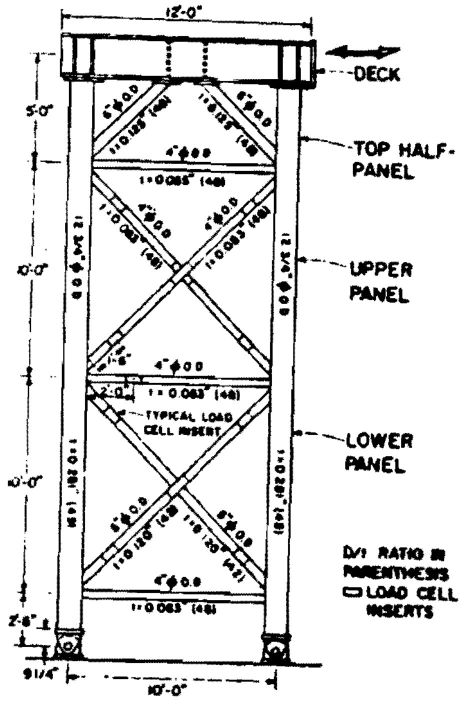
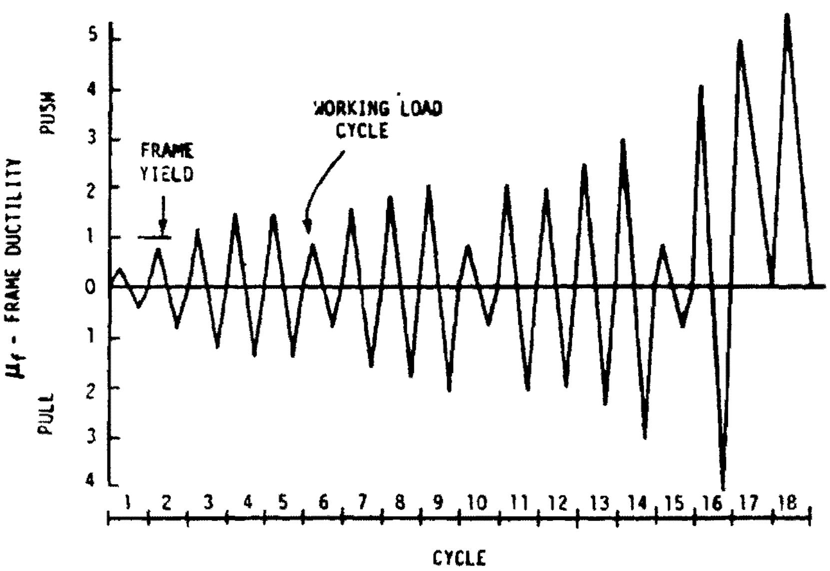


FIGURE 26



Prescribed Deck Displacement History

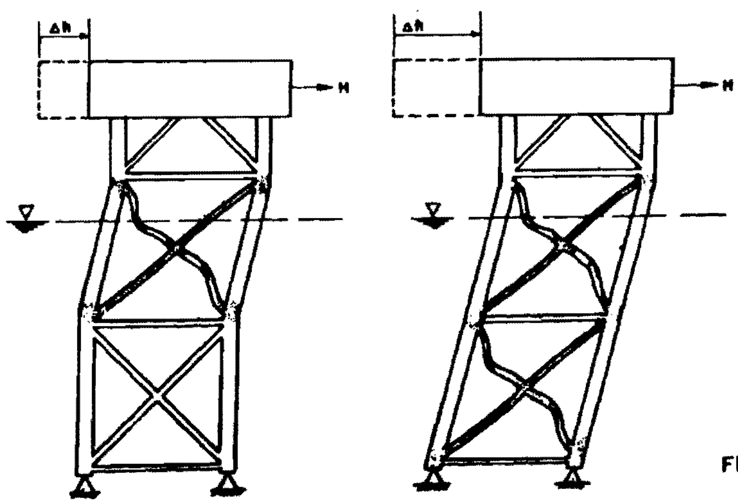
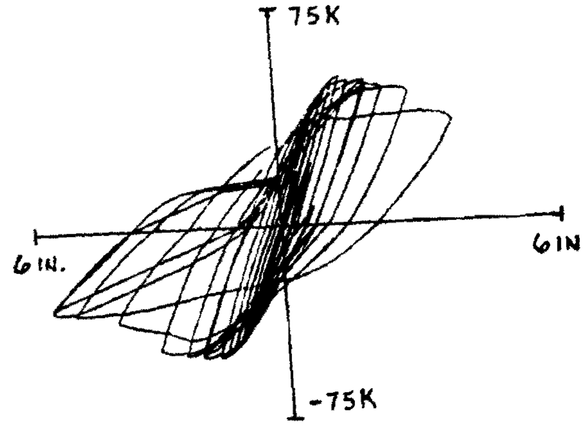
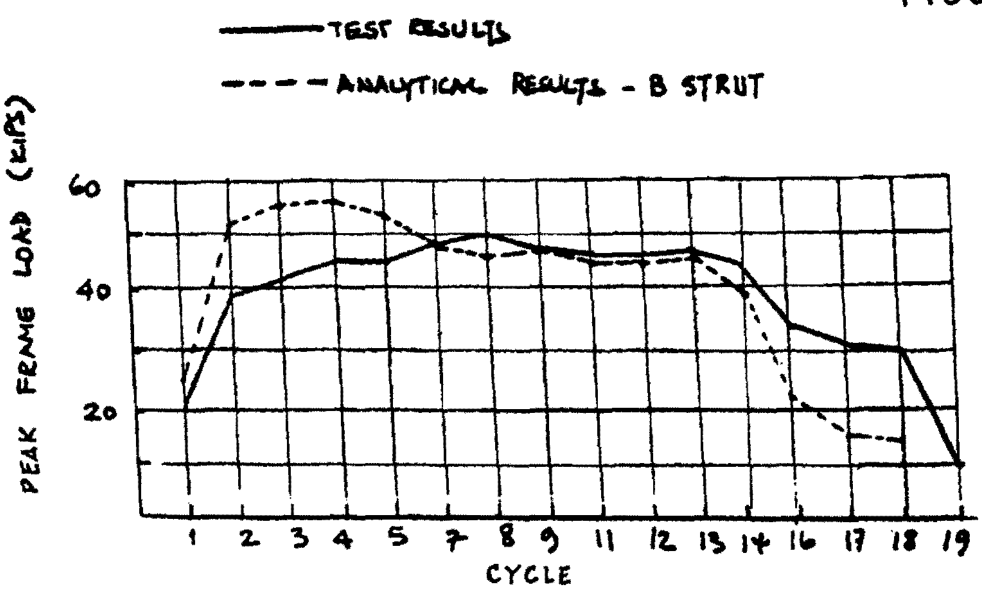


FIGURE 27



EXPERIMENTAL HYSTERESIS LOOPS
DECK LOAD VS DECK DISPLACEMENT

FIGURE 28



FRAME I.
Peak Loads on Push Half Cycles

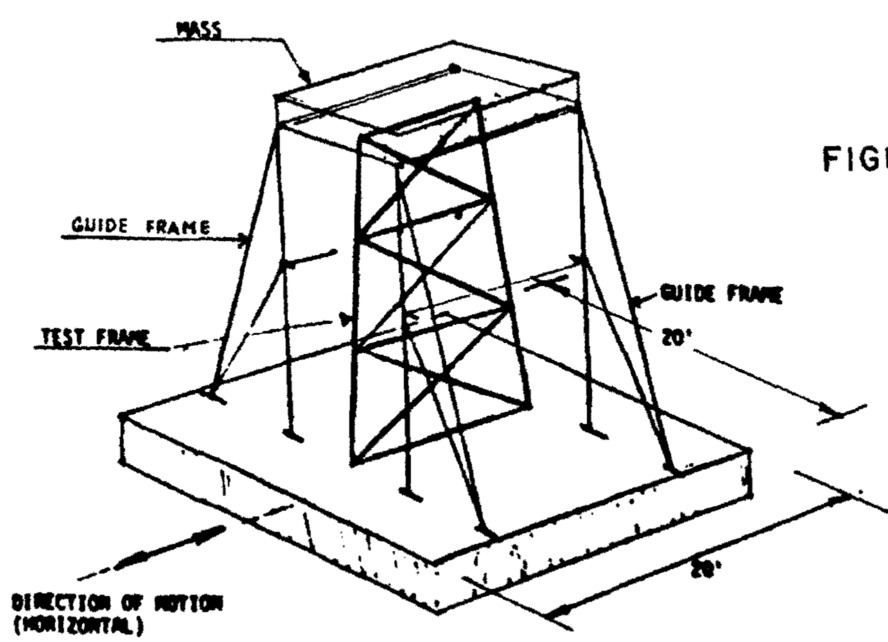


FIGURE 29

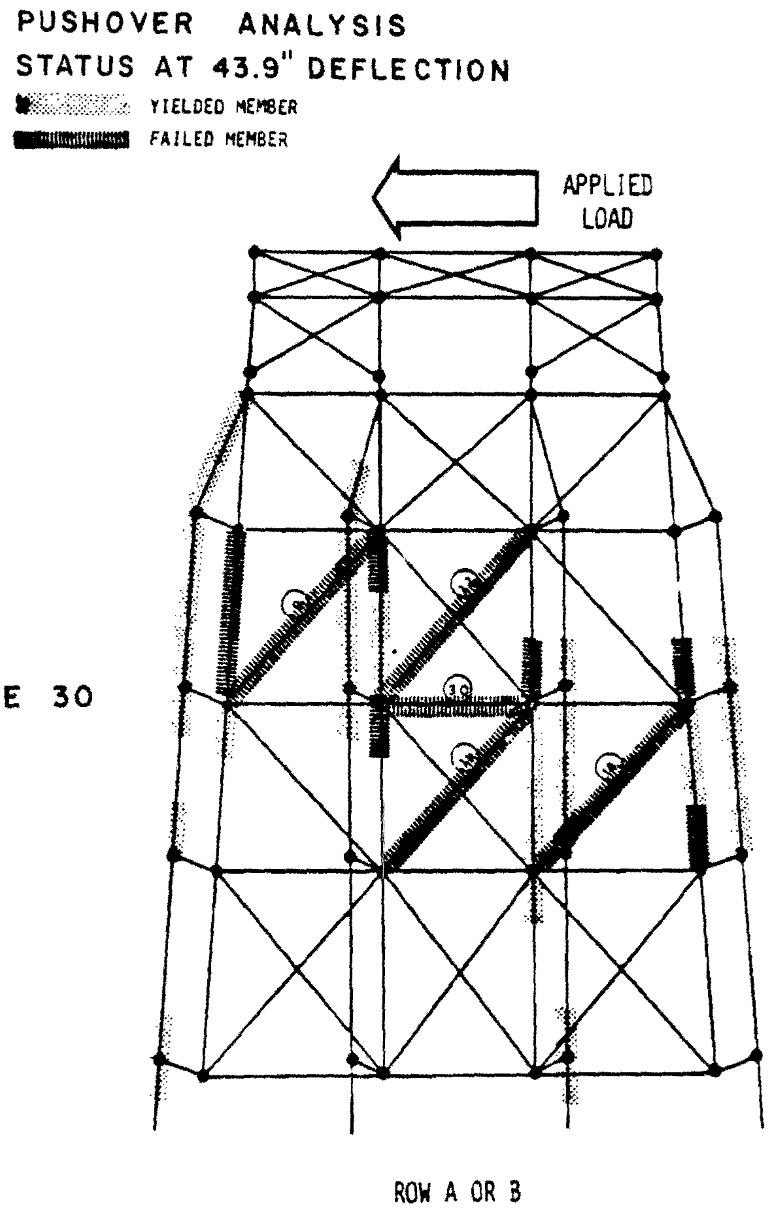
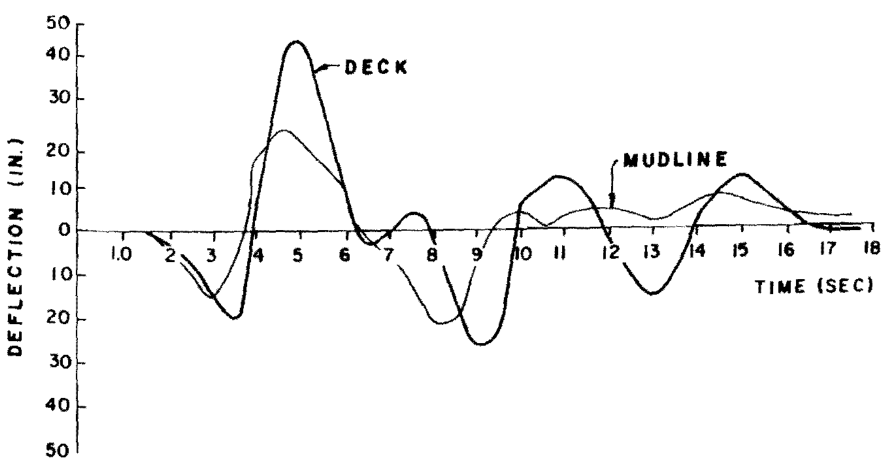


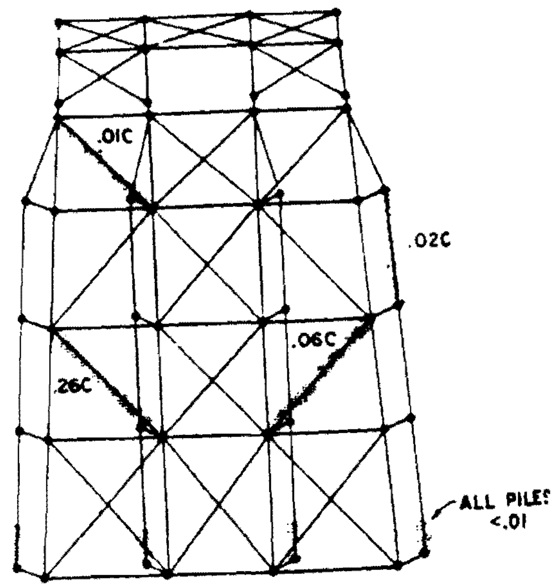
FIGURE 30



12 PILE PRODUCTION PLATFORM
MOTIONS FOR PACOIMA DCHARM END-ON

FIGURE 31

FIGURE 32



AREAS OF YIELDING & SPECIFIC ENERGY RATIOS AT END OF MOTION (1.00=FAILURE)

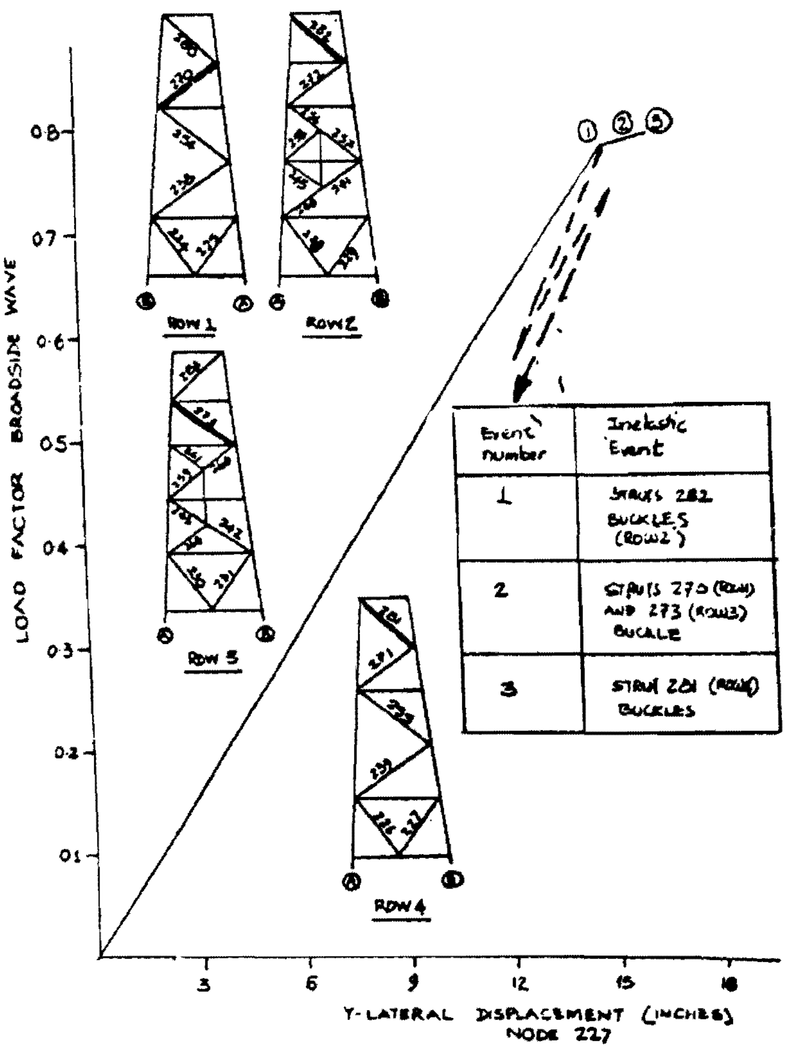


FIGURE 33

RESPONSE RESULTS, BROADSIDE WAVE

C. D. MILLER - Stability Considerations in the Design of Circular Tubes as Members of Offshore Structures

INTRODUCTION

The stability of circular tubes that are typical of members of offshore structures continue to pose a challenge to the engineers responsible for writing design rules. The theoretical buckling strength based upon linear elastic bifurcation analysis is well known for stiffened as well as unstiffened cylinders and simple formulas have been determined for many geometries and types of load. Initial geometric imperfections and residual stresses which result from the fabrication process, however, reduce the buckling strength of fabricated tubular members. The amount of reduction is dependent upon the geometry of the member, type of loading (axial, bending, etc.), size of imperfections and material properties. Knockdown factors based upon available test data and judgement of those drafting the rules, are contained in several different sets of rules (1-7) related to design of fabricated steel shells. The values of the knockdown factors vary significantly for many geometries and loadings.

The two most commonly used rules for design of offshore platforms are those of the American Petroleum Institute (1) and Det Norske Veritas (5). The API rules are limited to unstiffened or ring stiffened cylinders and cones with D/t values less than or equal to 300. The members of many of the North Sea platforms and new concepts for deep water exceed this limit. The effect of rings on buckling is considered only for external pressure loading. The effect of shear stress is not considered in combination with other stress components. The effects of column type buckling and buckling of stiffening elements are determined from the AISC Manual of Steel Construction and ASTM Standards are used for material specifications. Interaction equations are given for axial tension stresses as well as axial compression stresses in combination with hoop compression stresses.

The DNV rules are much more comprehensive than the API rules. Buckling criteria for bars, frames and plates as well as cylinders is included. The DNV rules consider stringer stiffened and ring and stringer stiffened cylinders as well as unstiffened and ring stiffened cylinders. Stiffened flat plate equations are used for stringer stiffened cylinders, however. Knockdown factors are given for D/t ratios up to 1600. Interaction equations are given for combinations of axial compression, hoop compression and in plane shear stresses.

The design criteria given in the API and DNV rules will be discussed and comparisons will be made. Some discussion and comparisons will also be made with an ASME Code Case (7) which contains rules for design of containment vessels. These rules are equally applicable to offshore structures and include design criteria for unstiffened, ring and/or stringer stiffened cylinders and doubly curved shells under any load combination. Modified orthotropic shell equations are used for ring and stringer stiffened cylinders. The rules are limited to D/t values of 2000.

FABRICATION TOLERANCES

The design rules given by API, DNV and ASME are based upon the requirement that the final structure meet specified tolerances. There are some similarities as well as significant differences in these requirements.

Overall Out-of-Roundness

The API and ASME rules require that the difference between the major and minor outside diameters not exceed 1% of the specified nominal outside diameter. The API rules have additional limits. The difference in diameters shall not exceed 1/4 in. (6.35 mm) for cylinders up to 48 in. (1.22 m) diameter and 1/2 in. (12.7 mm) for cylinders greater than 48 in. (1.22 m) diameter. The DNV rules require that the actual radius not exceed 0.5% of the specified radius.

Local Deviation from a True Circle

The DNV and ASME rules limit the maximum deviation from a true circular arc over a distance corresponding to one-half of the theoretical buckle wave length to a value between 0.3t and 1.0t. The API rules have no limits in addition to those given above. A possible justification for not having a local deviation requirement in the API rules is that the D/t values are limited to 300 and the stiffener spacings are usually not close enough to increase the buckling mode above $n = 2$. Also, only one longitudinal seam is normally required so that an initial form corresponding to $n = 3$ or higher is unlikely. Even so, it is the opinion of the author that the API rules should contain requirements for local deviation.

Straightness

The API rules specify that the straightness deviation not exceed 1/8 in. (3.17 mm) in any 10 ft. (3.05 m) for lengths over 10 ft. (3.05 m). The maximum deviation of the entire length must not exceed either 3/8 in. (9.52 mm) in any 40 ft. (12.2 m) or L/960. The DNV rules limit the maximum deviation over the entire length to L/667. In addition the local deviation over a distance $4\sqrt{Rt}$ must not exceed $\sqrt{Rt}/30$.

GENERAL DESIGN CRITERIA FOR BUCKLING OF UNSTIFFENED SHELLS

The API and ASME rules for designing tubular members are based upon an allowable stress design method where the design buckling stress is divided by a safety factor to obtain the allowable stress. The DNV rules are based upon a load and resistance factor design (LRFD) method where factors are applied to the loads as well as the buckling resistance. An API committee is presently drafting an alternate set of rules based upon the LRFD method.

The stability requirement of the API rules for states of stress which can be defined by one single reference stress is given by:

$$\Sigma f_i \leq \frac{F_i}{SF_i} \qquad (1)$$

where

f_i = stress due to a particular type load

SF_i = safety factor for load i

F_i = design buckling stress for load i

The design buckling stress, F_i, includes the interaction between column type buckling and shell buckling. The elastic shell buckling stress, F_{ie}, the inelastic shell buckling stress, F_{ic} and the design buckling stress are given by the following formulas:

$$F_{ie} = \alpha_i \sigma_{ie} \qquad (2)$$

$$F_{ic} = \eta F_{ie} \qquad (3)$$

$$F_i = \overline{K} F_{ic} \qquad (4)$$

where

σ_{ie} = elastic bifurcation buckling stress of perfect cylinder

The terms $\overline{K}$, α_i and η are factors (frequently known as knockdown factors) that account for parameters which modify the theoretical elastic buckling stress. $\overline{K}$ is a slenderness factor which accounts for the effect of overall column buckling, α_i is a capacity reduction factor which accounts for the effects of initial imperfections and boundary conditions and η is a plasticity reduction factor which accounts for nonlinearity in material properties and residual stresses.

The slenderness factor which is given by Eq. 5 is determined by substituting the shell buckling stress F_{xc} for F_y in the appropriate AISC design formula. $\overline{K}$ applies only to axial compression load and is equal to unity for all other loads. The design buckling stress for bending loads is limited to $0.66F_{xc}$.

$$\left.\begin{array}{ll} \overline{K} = 1 - 0.25\lambda^2 & \lambda \leq \sqrt{2} \\ \overline{K} = \lambda^{-2} & \lambda > \sqrt{2} \end{array}\right\} \qquad (5)$$

where

$$\lambda = \frac{KL}{\pi r}\sqrt{\frac{F_{xc}}{E}}$$

KL = effective length
r = radius of gyration
E = modulus of elasticity of steel

The safety factor for axial compression is determined from the same AISC equation as $\overline{K}$ and is given by:

$$\left.\begin{aligned} SF_x &= 1.667 + 0.265\lambda - 0.0442\lambda^3 && \lambda \leq \sqrt{2} \\ SF_x &= 1.92 && \overline{\lambda} > \sqrt{2} \end{aligned}\right\} \qquad (6)$$

The safety factors for all loads are summarized in the following table.

LOADING CONDITION	AXIAL TENSION	AXIAL COMPRESSION	HOOP COMPRESSION
ORDINARY	1.67	1.67 - 1.92	2.0
EXTREME	1.25	1.25 - 1.44	1.5

The stability requirement of the DNV rules for states of stress which can be defined by one single reference stress is given by:

$$\Sigma f_i \, \gamma_{fi} \leq R_{di} = \frac{R_{ki}}{\gamma_m} \, \frac{\psi}{\kappa} \qquad (7)$$

where

f_i = stress due to a particular load type
R_{di} = design buckling resistance
γ_{fi} = load factor depending on load type
R_{Ki} = characteristic resistance, equivalent to F_{ic} in Eq. 3.
γ_m = coefficient which covers material uncertainties (1.15 for elastic design, 1.3 for plastic design).
ψ = factor to reflect post buckling behavior (1.0 when redistribution is possible, 0.9 when not possible).
κ = coefficient to account for the increased sensitivity of more slender structures

The values of κ are given by:

$$\kappa = 1.0 \qquad \bar{\lambda} < 0.5$$
$$\kappa = 0.7 + 0.6\bar{\lambda} \qquad 0.5 \leq \bar{\lambda} < 1.0 \qquad (8)$$
$$\kappa = 1.3 \qquad \bar{\lambda} > 1.0$$

where

$$\lambda = \sqrt{F_y/\alpha_i\sigma_{ie}}$$

The characteristic buckling resistance, R_K, is given by:

$$R_K = \phi F_y$$

$$\phi = \frac{1}{\sqrt{1 + \bar{\lambda}^4}}$$

The load factors γ_{fi} are summarized in the table below.

LOADING CONDITION	LOAD CATAGORIES P	L	D	E
ORDINARY	1.3	1.3	1.0	0.7
EXTREME	1.0	1.0	1.0	1.3

P = permanent loads

L = live loads

D = deformation loads

E = environmental loads

Beam-column formulas are given to determine column type buckling. The interaction between shell buckling and column type buckling is accounted for by using a modified yield stress, F_{ym}, which is given below:

$$\frac{F_{ym}}{F_y} = \min \begin{cases} 1.0 \\ 1.07 - 0.8 \dfrac{F_y}{E} \dfrac{D}{t} \end{cases} \qquad (9)$$

REDUCTION FACTORS TO ACCOUNT FOR IMPERFECTIONS

Reduction factors are given by the DNV and ASME rules to account for the difference between test and theory for elastic buckling. The API rules give simple equations which are equivalent to the theoretical buckling stresses reduced by reduction factors. In general, the reduction factors given in the ASME rules and those used to determine the API equations are in close agreement.

Several test programs have been sponsored by the offshore industry in the United States since 1977 to determine the buckling strength of stiffened and ring stiffened cylindrical shells that are representative of offshore platform construction. The test cylinders were fabricated from steel plates with minimum yield strength of 36 ksi (248N/mm^2) and 50 ksi (344N/mm^2).

Axial compression tests (8, 9, 10) have been conducted on cylinders with D/t ratios of 44 to 248. One set of tests (9) investigated the effect of overall column buckling. The capacity reduction factors which account for effects of imperfections must be determined from tests which fail elastically. Since all tests failed inelastically, only the product of α and η could be determined. The number of tests that have been conducted on fabricated steel cylinders with D/t ratios over 300 is very limited and information on initial shape is even more limited.

External pressure tests (11) have been conducted on fabricated cylinders with D/t ratios of 30 to 120. The value of $\alpha = 0.8$ was found to give a lower bound on test data for cylinders which met the prescribed fabrication tolerances. A value of $\alpha = 0.8$ is used in the API and ASME rules. The values of the knockdown factors given in the DNV rules are dependent upon the parameter $Z[Z = L^2 (1 - \nu^2)/Rt]$ and vary from 0.45 to 0.75 for hydrostatic pressure and 0.6 to 1.0 for radial pressure. A summary of most of the tests that have been performed on steel cylinders that are representative of offshore structures is contained in Paper 16 of Ref. 12. This reference also contains other theoretical and experimental studies of buckling of shells.

Ring Stiffened Cylinders

Equations are given in the API, ASME and DNV rules for sizing rings so that local buckling precedes general instability. The assumption is made in the API and DNV rules that the buckling mode corresponds to $n = 2$ for cylinders subjected to external pressure. The API rules consider only hydrostatic pressure loading.

Ring and Stringer Stiffened Cylinders

The API rules do not include ring and stringer cylinders. The ASME rules are based upon orthotropic shell theory with the rigidity factors modified to treat the stiffened shell as a gridwork when the stringer spacing is not close enough to make the shell fully effective. Factors have been determined from tests that were available in 1978.

The DNV rules for design of stringer stiffened cylinders are the same as for stiffened plane panels. The rules for sizing rings are the same as those for ring stiffened cylinders except the effective thickness (t + area of stringer/stringer spacing) is substituted for the shell thickness in the equation for cylinders subjected to axial compression or bending loads.

PANAL DISCUSSION (Miller)

Design studies indicate that ring and stringer stiffened cylinders are more efficient than ring stiffened cylinders for cylinders with large D/t ratios and with axial compression loads that are a significant part of the total load. Until quite recently experimental data was available from only two cylinder tests (13) that were representative of offshore structures. In 1977 the Department of Energy, Creat Britain, funded an experimental program for testing stiffened cylinders under axial compression. DNV has initiated a test program for buckling of stringer stiffened cylinders under combinations of axial compression and hoop compression. These test programs will be discussed by others at this conference.

Conoco Inc. and American Bureau of Shipping (ABS) have initiated a test program on fabricated ring stiffened and ring and stringer stiffened steel cylinders subjected to combinations of axial compression and hoop compression. The cylinders will be fabricated from steel sheet with yield stress of 50 ksi ($344 N/mm^2$). The D/t ratios range from 300 to 1000. There will be 14 sets of cylinders with 4 or 5 cylinders in each set. The cylinders of each set will be tested under four different loading conditions: axial load alone, radial pressure alone, hydrostatic pressure and axial compression combined with hoop compression. Initial shape measurements of all models will be taken. The tests will be conducted at the University of Glasgow, Scotland under the direction of Professor Paul Frieze and at Chicago Bridge and Iron Company under the direction of the author. These tests will be used by ABS to draft design buckling rules for tension leg platforms.

Interaction Equations

Interaction equations are given in the API, ASME and DNV rules for cylinders subjected to combined loads. The DNV rules give the following equation for buckling of unstiffened cylinders or buckling between rings of ring stiffened cylinders subjected to combined axial compression, bending, hoop compression, torsion and shear:

$$\left(\frac{\sigma_a}{R_{da}} + \frac{\sigma_b}{R_{db}}\right)^2 + \left(\frac{\sigma_\theta}{R_{d\theta}} + \frac{\sigma_{\phi\theta}}{R_{d\phi\theta}}\right)^2 \leq \left(\frac{1}{\gamma_m}\right)^2 \qquad (10)$$

where σ_i is the stress due to applied loads and R_{di} is the design buckling resistance defined in Eq. 7. The subscript a corresponds to the axial load stress, b corresponds to the axial stress resulting from a moment load, θ corresponds to hoop stress and $\phi\theta$ corresponds to the in-plane shear stress resulting from transverse shear and torsion.

The following interaction equation is given by the DNV rules for stiffened cylinders under the same load combinations as for unstiffened cylinders:

$$I_{ef} \geq \sigma_N I_{efN} + \alpha_\rho I_{efP} + \alpha_T I_{efT} \tag{11}$$

$$\alpha_N = \left(\frac{\sigma_x}{R_{dx}} + \frac{\sigma_b}{R_{db}}\right) \gamma_m$$

$$\alpha_P = \frac{\sigma_\theta}{R_{d\theta}} + \gamma_m$$

$$\alpha_T = \frac{\sigma_{\phi\theta}}{R_{d\phi\theta}} \cdot \gamma_m$$

where I_{efi} is the effective moment of inertia of a ring when the cylinder is subjected to load i.

The API rules provide the following interaction equations for cylinders subjected to axial compression and hoop compression:

$$\frac{\sigma_x - 0.5F_{ha}}{F_{aa} - 0.5F_{ha}} + \left(\frac{\sigma_\theta}{F_{ha}}\right)^2 \leq 1.0 \tag{12}$$

$$\frac{\sigma_x}{F_{xc}} SF_x \leq 1.0 \tag{13}$$

$$\frac{\sigma_h}{F_{hc}} SF_h \leq 1.0 \tag{14}$$

where $F_{aa} = \frac{F_{xe}}{SF_x}$, $F_{ha} = \frac{F_{he}}{SF_h}$ and $\sigma_x = \sigma_a + \sigma_b$

Equation 13 describes a parabolic shaped curve between the loading cases of hydrostatic pressure and axial load alone. This curve was determined from a study of elastic buckling failures and the buckling stress values given in Eq. 13 are for elastic buckling. Equations 14 and 15 apply to inelastic buckling and they are based upon the assumption that little or no interaction exists between the axial and hoop stresses. There are no inelastic buckling tests of fabricated cylinders subjected to axial compression and hoop compression. For this reason, API has approved funds for a test program which will start in 1982 and continue in 1983.

Equation 11 makes no distinction between elastic and inelastic behavior, nor does it recognize the special loading case of hydrostatic pressure. Because of this, a designer will obtain different design requirements if he analyzed a cylinder with $\sigma_\theta = 0.5\sigma_a$ using equation 11 and then with the special rules given for hydrostatic pressure. This is illustrated in Figure 1.

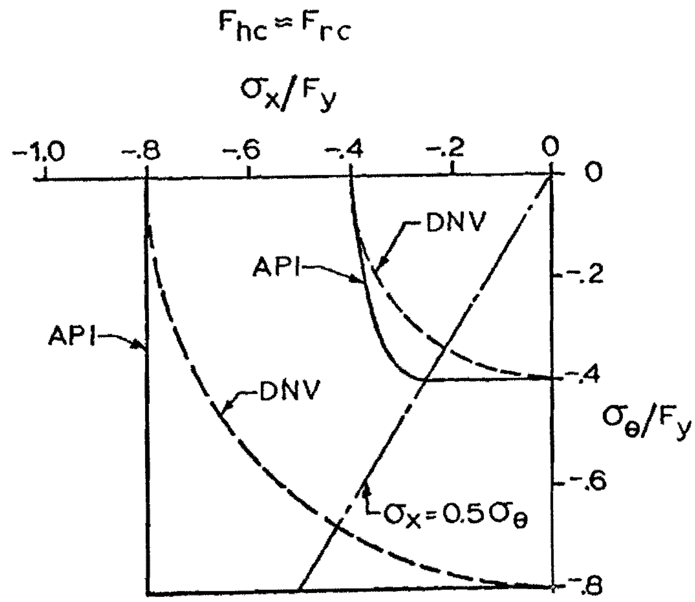


Fig. 1 Interaction Curves For Combined Axial Compression And Hoop Compression

Additionally, the API rules give the following interaction equation for cylinders subjected to axial tension and hoop compression:

$$\left(\frac{\sigma_x}{F_y}\,SF_x\right)^2 + \left(\frac{\sigma_\theta}{F_{hc}}\,SF_h\right)^2 + 0.6\left(\frac{\sigma_x}{F_y}\,SF_x\right)\left|\frac{\sigma_\theta}{F_{hc}}\,SF_h\right| \le 1.0 \qquad (15)$$

A series of tests on cylinders under combined axial tension and hoop compression was recently completed. The results of these tests are reported in Ref. 13 and a presentation was made at this conference.

CONCLUSIONS

It is apparent that there are significant differences in the buckling design rules given by API (1) and DNV (5) for offshore structures. The drafters of both sets of rules recognize the need for more test data from models which are representative of the fabricated members of offshore platforms. The initial shape of the test models and material properties are required for determination of knockdown factors to account for differences between test results and theory.

The test results should be used to develop reasonably simple mathematical models for computer determination of knockdown factors corresponding to fabrication tolerances and type of material specified. This will permit the design of more complex members with varying plate thicknesses, stiffener spacings and stiffener sizes. The mathematical models presently available with sufficient accuracy to account for initial geometry and fabrication processes are too time consuming and costly to be of use either to the designer or drafter of design rules. This is especially true for inelastic buckling.

The wide differences in the present design rules for buckling of fabricated shells result from this lack of correlation between test results and predicted stresses based upon theoretical methods and also from lack of adequate test data. Other papers presented at this conference will show that progress is being made in both areas.

REFERENCES

1. American Petroleum Institute, "Recommended Practice for Planning, Designing and Constructing fixed Offshore Platforms," API RP 2A, Eleventh Edition, January 1980.

2. American Society of Mechanical Engineers Boiler and Pressure Vessel Code, Section VIII, Divisions 1 and 2 and Section III, Division 1, 1980 Edition.

3. British Standards Institute, Specification for Unfired Fusion Welded Pressure Vessels, BS5500, 1976.

4. DAST Rule 013, "Beulsicherheitsnachweise fur Schalen," May 1980, (German Standard).

5. Det Norske Veritas, Rules for the Design Construction and Inspection of Offshore Structures, Appendix C, 1977.

6. European Convention for Construction Steelwork, Proposed Rules R4.6 "Buckling of Shells."

7. American Society of Mechanical Engineers, Boiler and Pressure Vessel Code Case N-284, "Metal Containment Shell Buckling Design Methods," Supplement #2 to Nuclear Code Case Book, 1980.

8. Marzullo, M.A., and Ostapenko, A., "Tests on Two High-Strength Short Tubular Columns," Fritz Engineering Lab. Report No. 406.10, May 1977.

9. Chen, W.F. and Ross, D.A., "The Strength of Axially Loaded Tubular Columns," Fritz Engineering Lab. Report No. 393.8, Lehigh University, September 1978.

10. Ostapenko, A., and Grimm, D.F., "Local Buckling of Cylindrical Tubular Columns Made of A-36 Steel," Lehigh University, Fritz Engineering Lab. Report No. 450.7, February 1980.

11. Miller, C.D., and Kinra, R.K., "External Pressure Tests of Ring Stiffened Fabricated Steel Cylinders," Offshore Technology Conference, Paper No. OTC 4107, 1981.

12. Buckling of Shells in Offshore Structures, edited by Harding, J.E., Dowling, P.J. and Agelidis, N., Granada Publishing Company, London, Toronto, Sydney, New York, 1982.

13. American Petroleum Institute, "Specification for Fabricated Structural Pipe," API Spec 2B, Third Edition, November 1977.

14. Miller, C.D., Kinra, R.K., and Marlow, R.S., "Tension and Collapse Tests of Fabricated Steel Cylinders," Offshore Technology Conference, Paper No. OTC 4218, 1982.

J. R. Lloyd - Design Strategy for Redundant Space Frames

SUMMARY

Conventional design practice sizes individual members in a structural system according to code requirements. When the system is highly redundant, some members may not be heavily loaded, yet their presence contributes significantly to the system resistance in a damaged condition.

An approach is presented to illustrate how such redundant members can be sized to achieve target residual strength when primary members are deleted from the system. The approach essentially optimizes the distribution of material among the structural members. A three-dimensional, X-braced space frame is used to illustrate the method.

1.0 INTRODUCTION

The objective of structural design codes is to provide safe and reliable structures. All codes, whether they are based on working strength design (WSD) or load and resistance factor design (LRFD), address the design of individual members. For example, to check the structural adequacy of a system of members subjected to prescribed loading, the analyst first establishes the forces in each member (usually with linear analysis) and then verifies that the members and connections are strong enough by code standards to resist these internal forces. If all members and connections in the system satisfy the code and meet other accepted engineering practices, then the system is considered to meet code standards. Each of the members so designed will have a reserve strength against failure.

Implicit in this procedure is the premise that failure of one member to satisfy code requirements constitutes an unacceptable condition or "failure" of the system to meet code. Indeed, if the system of members is structurally determinate, failure of one member will constitute overall system failure, as well as failure to meet a code. Such a situation can arise with diagonally-braced or K-braced trusses (Figure 1a, b).

PANEL DISCUSSION (Lloyd)

For the highly redundant structures that represent a large portion of civil engineering designs, failure of one member usually can be tolerated without system collapse (Figures 1c, d, and e). The main point is that structures that are redundant have a residual strength even if a primary member has failed. Futhermore, different structures designed to the same code do not necessarily have the same reliability or reserve strength above the design load level.

The need for system redundancy is difficult to address in a code format, and much of the success of engineering achievements can be attributed to the conservative and experienced judgement of the designers. If failures were highly predictable, there would be little need for conservatism in any form, but this is not the case. Loads can exceed those predicted, or members can be weaker than expected, i.e. members can fail at loads below as well as above the design member strength. Material defects, corrosion, impact damage, or fatigue can cause member understrength. It is very important, therefore, to provide adequate reserve strength by either (1) maintaining tight quality control, (2) overdesigning members in statically determinate structures and/or (3) providing alternative load paths as in redundant structures. This paper addresses only the latter class of design conservatism.

It is very easy to mathematically quantify the degree of structural redundancy by comparing the number of unkown internal member forces with the system degrees of freedom. This measure, however, can be very deceiving, if the redundancy is created by very weak members, or some areas of the structure are highly redundant but others are not. The best measure of the importance of a single redundant member is to evaluate the residual strength of the system if other primary members have failed. Though the system strength will be reduced when members are removed, a residual strength will exist. This residual strength is a good measure of the redundancy of an individual member as well as the overall redundancy of the system. It serves as a reasonable basis to size the redundant members.

The most difficult problem in designing redundant structures is sizing members which, under applied loading, theoretically may carry no load at all. For example, in the X-braced truss shown in Figure 2a, linear analysis will show that the horizontal braces carry little or no load, and the economy minded engineer may be inclined to delete them from the system. However, the horizontal member plays a crucial role in providing an alternative load path if one or more of the main diagonals should fail (Figure 2b). The question is how to size the horizontal brace to achieve some target level of residual strength after primary member failure, or alternatively, how much larger do the diagonals have to be to sustain the loss of one diagonal?

The objective of this paper is to present a methodology for establishing a design strategy that achieves target system residual strength with a minimum amount of material. An example strategy is developed for a three dimensional X-frame configuration.

2.0 ANALYSIS METHODOLOGY

The following simplifying assumptions have been made in the method presented.

1) The structural systems are triangulated truss type and loads are applied only at nodes, so that primary and secondary moments can be ignored, i.e. pinned frame.

2) Members behave in an elastic perfectly plastic sense in both tension and compression. This assumption is necessary to achieve a simple solution. While the assumption is highly restrictive, the resulting solutions provide suitable bounds for system strength. This point is discussed further in the example problem.

3) The optimization cost function is the total weight of all the members in the system. While this cost function is commonly used in optimization problems, it should be recognized that other factors are important in evaluating actual costs, such as connection details and construction.

4) Each load condition is optimized independently. The maximum member size is retained for each member or member class, considering all load conditions. (It would be preferred to treat all load conditions in a single optimization step, but this would greatly compound the computational difficulty, achieving an accuracy unwarranted by the crudeness of the previous assumptions.)

Structural adequacy is herein based on the maximum or ultimate load resisting capacity of a given structural frame system, in contrast to the allowable strength limits of individual members as in code checking. In redundant structural systems, an infinite number of combinations of member sizes can be selected to provide a given ultimate capacity. If weight penalties are associated with member size, it is possible to find a combination of member sizes for the structural system that will minimize the total structure weight. In Appendix B, a linear programming formulation is presented that develops an upper bound estimate of the minimum structure weight using a load path optimization procedure. This procedure is the same as given La Pay and Goble.* Equilibrium is satisfied in the ultimate limit state, but compatibility is not satisfied. The procedure establishes the member sizes that minimize the structure weight when a strength limiting mechanism is formed.

Once this optimum configuration has been established, the problem is to assess the system redundancy by systematically removing primary structural members. The main result achieved is the required member sizes and system cost to achieve a target ultimate strength.

---

*La Pay, W. S. and Goble, G. G., "Optimum Design of Trusses for Ultimate Loads," Journal of the Structural Division, ASCE, Vol. 97, No. ST1, January 1971, pp. 157-174.

This assessment can be made by undertaking the following steps.

1. Establish the member sizes that minimize the undamaged structure weight for a prescribed set of load conditions, using the procedure in Appendix B.

2. Select a target residual strength fractionally less than the initially prescribed loads.

3. Remove a primary member from the structural system, but retain its weight in the total system weight.

4. Increase the member sizes as necessary to resist all the load conditions, following the same optimization procedure as given in Appendix B. The maximum member sizes are retained among those determined for all load conditions considered.

5. Repeat the cycle from step 3 replacing the deleted member and removing another member again retaining the maximum size for each member.

6. Repeat the cycle from step 2, choosing another target residual capacity and starting with the member sizes of the optimized intact structure found in step 1.

Using this procedure, the requirements for redundant members (such as horizontals in X-braced trusses) can be established for any given target residual strength i.e. the fraction of the original intact ultimate strength. Associated system weights will also be developed.

3.0 EXAMPLE

A three dimensional X-frame is used to illustrate the application of the method. Figure 3 shows the three level X-frame considered. All bracing members are assumed pinned at their ends and have the same yield strength and compression to tension capacity ratio, α. The vertical members are assumed to have equal tension and compression strength. The four support nodes are pinned.

The frame is to be designed to resist a horizontal load applied at the top, and the load can be applied in any horizontal direction. The frame is therefore assumed structurally symmetric about the vertical axis. All members of the same type have the same properties; for example, all diagonal members are the same size regardless of level. Vertical members at a given level are the same size, though their sizes vary with level.

In this example, a unit horizontal load is applied at the top of the frame, divided equally among the top four nodes. Three load directions 0°, 22.5° and 45° are considered as separate load conditions (See Figure 3).

Three distinct frame configurations have been considered (See Figure 4):

A) Base case (with all horizontals and framing members included).

B) Case A without horizontals 9, 10, 11, 12, 27, 28, 29, and 30.

C) Case B without framing members 17, 18, 35, and 36, (no horizontal or framing members).

For Case A, conventional elastic analysis would lead to the conclusion that the horizontal and framing members are entirely redundant and can be eliminated, i.e. Case C. In normal design practice, the sizing of such redundant members (if included at all) is usually based on minimum slenderness ratios, minimum wall thickness, secondary bending stresses, local loading, or construction loading. As will be shown later, sizing of these members should also depend on (1) the level of desired residual strength when individual members have been removed from the system, and (2) the member compression to tension strength ratio, α.

3.1 Compression to Tension Strength Ratio, α

Figure 5 is a schematic illustration of how the simplifing assumption of elastic-perfectly plastic behavior models the likely actual load deformation behavior of a typical brace. While the modeling is not very precise, neither is the knowledge of actual member behavior. It can be considered that $\alpha = 0.6$ is an approximate representation of deformation behavior of an individual member with kl/r of about 80 and 36 ksi yield strength. A value of $\alpha = 0.2$ is a fair representation of member behavior well after peak buckling strength has been mobilized. Naturally, a more realistic solution would have to make use of actual member deformation behavior in a nonlinear analysis procedure. This more exacting approach would make the optimization solution step very difficult indeed. The following results should be evaluated with these modeling limitations in mind.

3.2 Optimization Basis

In this example, optimization was based on the total weight of all the members. Since vertical loads are not applied in the example, the strength of the optimized vertical members can be considered to be the strength increase necessary to resist lateral loads, incrementally greater than needed to resist vertical loads had they been applied. The optimization results are first presented for the intact (undamaged) frame. Then the consequences of diagonal member deletion are presented, using the same optimized intact frame as the initial design.

3.3 Intact Optimization Results

The intact optimization results are given in Table 1 for the three frame configurations (A, B, and C) and three values of the parameter, α. The member strengths in Table 1 are normalized by the applied load. For example, in Case A the diagonal braces would have to have a tension capacity of 35.3 kips if the applied load is 100 kips. The system weight is normalized by the applied load, F, a characteristic dimension of the frame, L, the material yield strength, σ_y, and material density, ρ.

$$w = \frac{\sigma_y}{\rho F L} \sum \rho A_i l_i \tag{1}$$

where the summation includes all vertical, diagonal, horizontal and framing members. A_i and l_i are the cross sectional area and length of each member, respectively.

Table 1 shows that when the compression to tension strength ratio is one ($\alpha = 1.0$), optimal solutions for all three cases are the same; namely, the horizontal and framing members are not needed, and the remaining member types all have the same strength. This ultimate strength result is consistent with that of elastic analysis. Since the stiffnesses of paired compression and tension diagonals are the same in an elastic analysis, they tend to attract equal loading, and the horizontals do not carry any load, regardless of their size.

When loads exceed the elastic limits either through yielding or buckling, the loads will no longer be shared equally between paired diagonals. This effect is demonstrated in Table 1 for compression to tension strength ratios less than one, i.e. $\alpha = 0.6$ and $\alpha = 0.2$. It can also be seen that it requires less system weight to achieve a given ultimate strength by using horizontal and framing members (Cases A and B) than to increase the sizes of diagonals (Case C).

Essentially, the horizontal members must transfer part of the tension diagonal load from the upper bay to the tension member of the lower bay, since the compression diagonal cannot carry the full load of the upper tension diagonal.

Comparing Cases A and B for both $\alpha = 0.6$ and $\alpha = 0.2$ shows that when horizontal braces are deleted, the framing members must transfer the loads that would normally be carried by the horizontals. For example, the load carrying capacity in the direction of the horizontal is exactly the same for the horizontals (Case A) and frames (Case B), i.e. 0.29463 x cos 45° = 0.2083 and 2.35702 cos 45° = 1.66667.

When there are neither horizontals nor framing members (Case C), the load carrying capacity of even the tension diagonals is limited to the compression capacity of the diagonals. This can be verified by checking horizontal equilibrium at the leg nodes. The tension capacity, q_D required of the diagonals in terms of the applied shear for Case C is given by

$$q_D = \sigma_y A_D = \frac{F}{4\alpha \cos 45}$$

$\alpha = 0.6 \qquad q_D/F = 0.58926$

$\alpha = 0.2 \qquad q_D/F = 1.76777$

The framing system for Case C not only weighs more, but it also fails to provide resistance against distortion of the frame in the horizontal plane. For these reasons, Case C is not considered a suitable design alternative.

3.4 Frame Redundancy

System redundancy and corresponding design strategy have been evaluated for frame Cases A and B. Only diagonal brace failure has been considered. The procedure follows that outlined previously; each of the six diagonal braces on one face (members 1, 5, 19, 23, 37, and 41, Figure 3) was individually removed and replaced. Only one member was removed at each resizing step. This process was repeated for 10 increments of load from 0.1 to 1.0 times the original ultimate load. The normalized structure weight dependence on load level is shown in Figure 6.* The normalized member sizes are shown in Figures 7 through 12.*

Figure 6 shows that frame Cases B and C require the least total weight to achieve a target redundancy strength when member compression and tension strengths are equal ($\alpha = 1.0$). Frame Case A generally weighs more. It would seem that Case B and Case C solutions would themselves be candidate optimum solutions for Case A, therefore the Case A frame should not weigh more. There are two reasons why this apparent paradox occurs. First, Cases B and C are not true subsets of Case A, because of the required horizontal at the top level for Cases B and C. In Case A all horizontals were required to be the same size so that Cases B and C are not true candidate optimum solutions. Second, the maximum member sizes for each member class may not occur for the same load condition, this is a shortcoming of not optimizing for all load conditions in a single step.

When compression strength is less than tension strength ($\alpha = 0.6$ and $\alpha = 0.2$); frame Cases A and B have about the same weight, and frame Case C weighs more. The optimum design strategies for Cases A and B achieve essentially the same effectiveness in the use of material for residual frame strengths up to about 60 percent of the undamaged structure ultimate strength. Table 2 summarizes the fraction weight increase of bracing necessary to achieve various fractions of the intact frame strength. For example, to achieve 60 percent of the original ultimate strength, the frame must be strengthened at a weight increase from 0 to 10 percent for both frame Cases A and B, depending on α. Note that the weight increases quite rapidly beyond 60 percent of the original strength.

If 60 percent is chosen as the target residual capacity, then the frame is fully one diagonal member redundant, provided the factor of reserve between the ultimate strength and the design load is 1.67 (1.0/0.6) or greater. Therefore, if the members of the example frame are designed to 60 percent of their ultimate capacity, then the frame

---

*In these "curves," the costs and sizes for each descrete load levels are connected by straight lines to indicate connectivity. Intermediate points do not necessarily constitute optimum solutions.

should be structurally sound with one failed diagonal. This factor of reserve is very consistent with the nominal reserves achieved in AISC design practice for individual members.

3.5 Member Sizing

Figures 7 through 12 give the member sizes for optimum design with the diagonals systematically removed at each level of residual load. Table 3 gives the corresponding member design strategy for a frame residual capacity of 60 percent. For example, with $\alpha = 0.6$ the diagonals should have a tensile strength of 0.44 times the applied load. The horizontals should have 0.47 times the tensile strength of the diagonals, and the framing members should have 0.23 times the tensile strength of the diagonals. Again, the apparent spurious result that Case B framing members are smaller than those for Case A is a consequence of Case B not being a true subset of Case A and the optimization steps being carried out separately for each load condition. Therefore, the design strategies in Table 3 are conservative.

Table 3 points out the futility of attempting to achieve 60 percent residual strength in cases where the compression strength is considerably diminished ($\alpha = 0.2$).

3.6 Mechanics of Load Redistribution

When a diagonal brace fails or is removed, essentially two mechanisms of load redistribution occur simultaneously:

1) The loads formerly carried by the failed diagonal are transferred by means of the horizontals to the diagonal pair of the failed diagonal.

2) The loads are transferred by framing shear and system torsion to the other parallel bent. The shear is transferred by the framing members, and the torsion is developed by diagonals on the remaining three faces of the frame.

These mechanisms cannot be mobilized without adequate horizontal or framing braces. The results of this example simply establish the sizes of these members necessary to achieve the desired redundancy at minimum structure weight.

Figure 13 gives the member loads for one load path optimization solution.

Case A (See Figure 8)

$\alpha = 0.6$

$\Theta = 0°$

$F/F_o = 0.6$

Member 37 removed

It can be verified from equilibrium at Section n-n that 31 percent of the total load is carried by the damaged bent (A-B) and the remaining load is carried by the undamaged bent (C-D).

4.0 CONCLUSIONS BASED ON EXAMPLE FRAME ANALYSIS

Intact Solution

1) When the compression to tension strength ratios for members is one ($\alpha = 1.0$), the optimum solution shows that horiziontal and framing members are unnecessary. This result is consistent with conventional elastic analysis.

2) When the ratio α is less than one, the optimum solution requires horizontals and framing members. Also, the diagonal braces must be stronger than given by the case $\alpha = 1.0$.

3) Framing and horizontal braces are "interchangeable" with no weight penalty.

Redundancy Solution

4) There is little distinction in system weight between frame Cases A and B.

5) System weight increases sharply at residual strength targets greater than 60 percent of the intact, optimum design strength.

6) To achieve approximately 60 percent residual capacity for the example problem, horizontals should have about 50 percent of the strength of the diagonals, and the framing members should have about 25 percent of the diagonal member strength. While these rules must be restricted to the example studied, the extension to other X-braces may be valid and needs to be checked.

APPENDIX A Notation

A_i	cross sectional area of member i
C	optimization cost function
f_i	axial force in member i (+ tension, - compression)
F_{kj}	external force on node k in direction j
F	total force applied to example frame
ℓ_i	length of member i
L	characteristic dimension of example frame
q_i	tensile strength of member i
Q_ℓ^i	member strength for class ℓ
w	dimensionless weight of system
X_i	transformation variable for force in member i

α_i	ratio of compression strength to tension strength for member i
θ	direction of applied load (See Figure 3)
σ_{ikj}	direction cosines of member i at node k in coordinate direction j
σ_y	yield stress of all members
ρ	material density

APPENDIX B

Load Path Optimization Procedure

In this appendix a procedure to develop an upperbound minimum weight solution for redundant structural systems is developed. A linear programming solution is based on satisfying node equilibrium and selecting member sizes to minimize total system weight. The solution is an upperbound estimate of the minimum weight, since compatibility is not guaranteed. Essentially, an optimum load path solution that develops a failure mechanism is generated.

The equations of equilibrium for each node in the structure are given by

$$\sum_{i=1}^{M} \phi_{ikij}\, f_i + F_{kj} = 0 \qquad \begin{array}{l} j = 1,2,3 \\ k = 1,2---N \end{array} \qquad \text{(B1)}$$

where F_{kj} is the external force applied to node k in a direction j

f_i is the axial force in member i (+ tension, - compression)

ϕ_{ikj} are the direction cosines of member i at node k in coordinate direction j

N is the total number of unsupported nodes

M is the total number of members

This is a set of 3N equations in M unknowns. Remembering that the classes of structures addressed are redundant, M exceeds 3N.

Member loads are restrained to be less than member strengths.

$$-\alpha_i q_i \le f_i \le q_i \qquad i = i,2,---M \qquad \text{(B2)}$$

where q_i is the strength of member i in tension ($q_i = \sigma_y A_i > 0$),

α_i is the member compression to tension strength ratio ($\alpha_i > 0$).

It is usual practice to group members into common member classes to account for structure and loading symmetries. This practice also avoids specification of too many mill sizes, thereby avoiding material wastage, etc. Member strengths are therefore defined by

$$q_i = Q^i_{o\ell} + Q^i_\ell \qquad \ell = 1,2,\text{---}S \tag{B3}$$

where $Q^i_{o\ell}$ represents the initial specified strength of member class ℓ (member i being in class ℓ).

where Q^i_ℓ is the increment of member strength for class ℓ to be determined that minimizes the system cost.

S number of member classes.

Substitution of Equation B3 into Equation B2 gives 2M inequalities in S additional unknowns.

The optimization cost function is the system weight

$$C = \frac{\rho}{\sigma_y} \sum_{i=1}^{M} 1 \; q \tag{B4}$$

where 1_i is the length of member i.

Equation (B1) through (B4) can be cast into a linear programming format by making the following transformation:

$$X_i = f_i + \alpha_i q_i \tag{B5}$$

Substituting Equation B5 into Equations (B1) through (B4) establishes the following standard linear programming format:

$$X_i \geq 0 \qquad i = 1,2,\text{---}M$$

$$X_i - (1 + \alpha_i)Q^i_\ell \leq (1 + \alpha_i)Q_{o\ell} \qquad 1 = 1,2,\text{---}M$$

$$\sum_{i=1}^{M} \phi_{ikj}X_i - \sum_{i=1}^{M} \phi_{ikj}\alpha_i Q^i_\ell = -F_{kj} + \sum_{i=1}^{M} \phi_{ikj}\alpha_i Q_{o\ell} \qquad \begin{matrix} j = 1,\ 2,\ 3 \\ k = 1,\ 2,\text{---}N \end{matrix}$$

$$C = \frac{\rho}{\sigma_y} \sum_{i=1}^{M} 1_i Q^i_\ell$$

In this formulation the variables are the X_i and Q^i_ℓ, and the cost function C is minimized. The solution gives a set of f_i that satisfies node equilibrium and member sizes Q^i_ℓ that minimize the cost function.

TABLE 1

Optimum Member Strengths for Example Frame Normalized by Applied Load - Intact Condition

α	Frame Case	Level	(Member Load)/(Applied Load) Vert.	Diagonal	Horizontal	Framing	Weight (w)
1.0	A	3	0.35355	0.35355	0.0	0.0	24.73
		2	1.06066	"	"	"	
		1	1.76777	"	"	"	
	B	3	0.35355	0.35355	0.0	0.0	24.73
		2	1.06066	"	-	"	
		1	1.76777	"	-	"	
	C	3	0.35355	0.35355	0.0	0.0	24.73
		2	1.06066	"	-	-	
		1	1.76777	"	-	-	
0.6	A	3	0.39967	0.44194	0.20833	0.01359	30.53
		2	1.07603	"	"	"	
		1	1.75240	"	"	"	
	B	3	0.39967	0.44194	0.15625	0.29463	31.63
		2	1.22447	"	-	"	
		1	1.75240	"	-	"	
	C	3	0.35355	0.58926	0.0	0.0	32.73
		2	1.06066	"	-	-	
		1	1.76777	"	-	-	
0.2	A	3	0.58926	0.58926	1.666667	0.0	55.56
		2	1.29636	"	"	"	
		1	2.00347	"	"	"	
	B	3	0.58926	0.58926	0.20833	2.35702	
		2	1.42318	"	-	"	
		1	2.00347	"	-	"	
	C	3	0.35355	1.76777	0.0	0.0	72.73
		2	1.06066	"	-	-	
		1	1.76777	"	-	-	

TABLE 2

Fraction increase in structure weight to achieve target residual strength in damaged condition.

α	Truss Case	Fraction of intact strength 0.2	0.4	0.6	0.8	1.0
1.0	A	1.05	1.08	1.10	1.13	1.32
	B	1.02	1.03	1.04	1.10	1.43
0.6	A	1.00	1.00	1.02	1.15	1.40
	B	1.00	1.00	1.00	1.10	1.38
0.2	A	1.00	1.02	1.05	1.35	1.61
	B	1.00	1.00	1.02	1.28	1.58

TABLE 3

Example Frame Design Strategy at 60% Residual Strength

α	Case	q_D/F	q_H/q_D	q_F/q_D
1.0	A	0.35	0.35	0.29
	B	0.35	0.35*	0.17
0.6	A	0.44	0.47	0.23
	B	0.44	0.34*	0.67
0.2	A	0.61	2.75	.42
	B	0.61	0.34*	4.00

F - total load applied to frame

q_D - tensile strength of diagonal members

q_H - tensile strength of horizontal members

q_F - tensile strength of framing members

* - horizontal members at top level only.

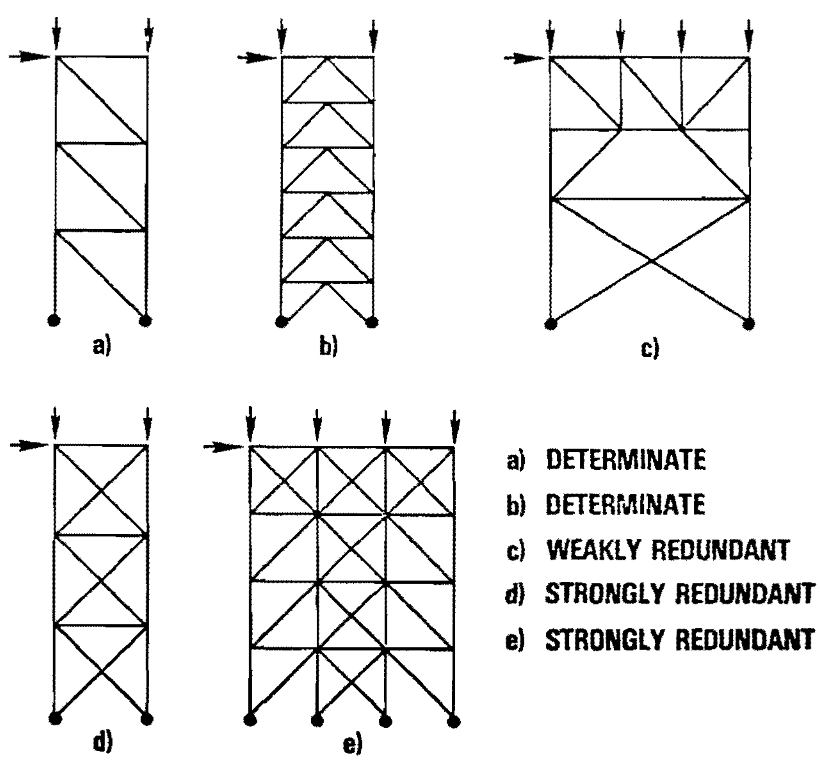


FIG. 1 DETERMINATE AND REDUNDANT TRUSSES

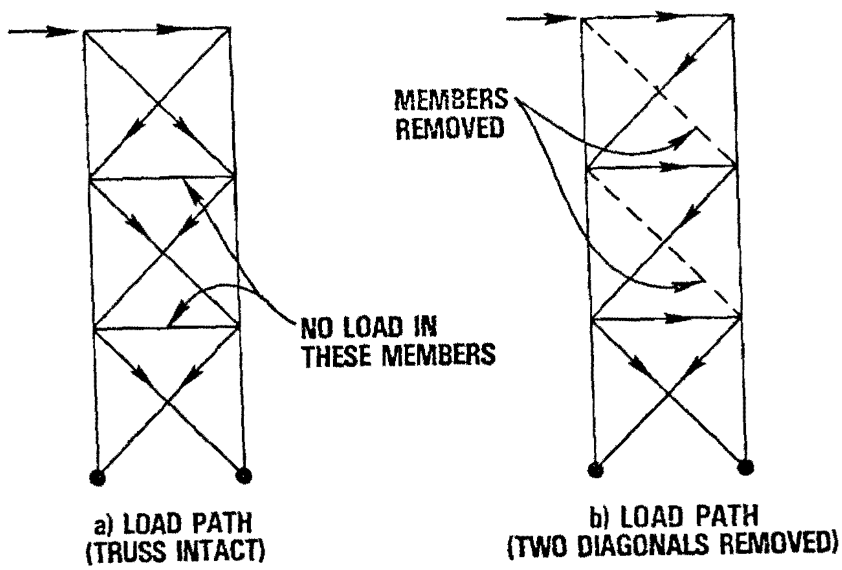


FIG. 2 THE ROLE OF HORIZONTALS IN PROVIDING ALTERNATIVE LOAD PATHS IN A DAMAGED X-BRACED TRUSS

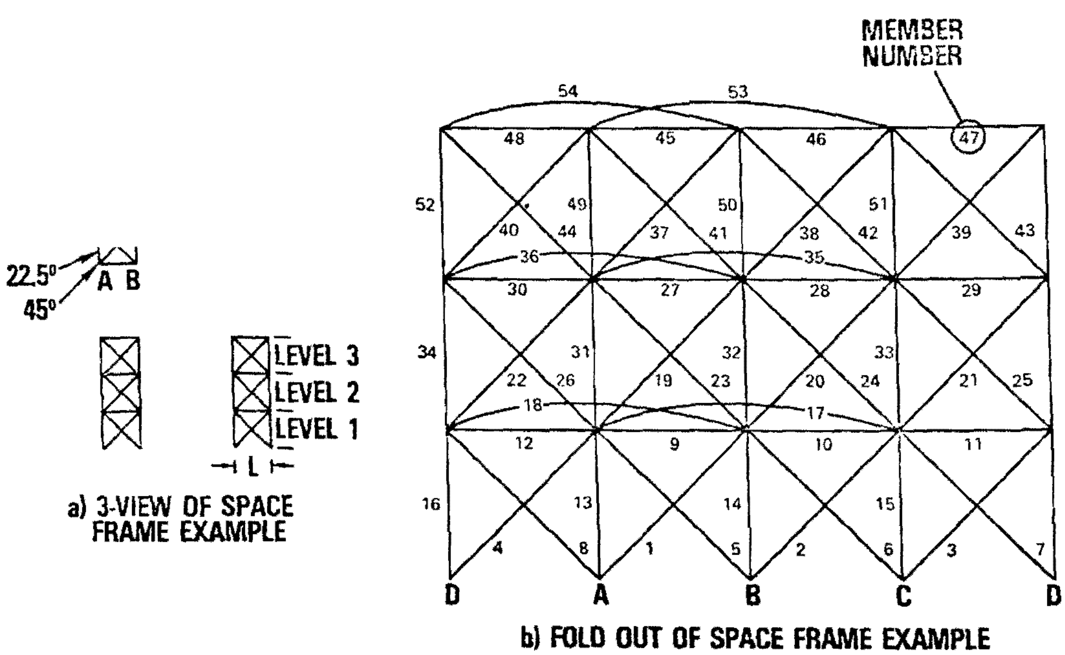


FIG. 3 SPACE FRAME EXAMPLE

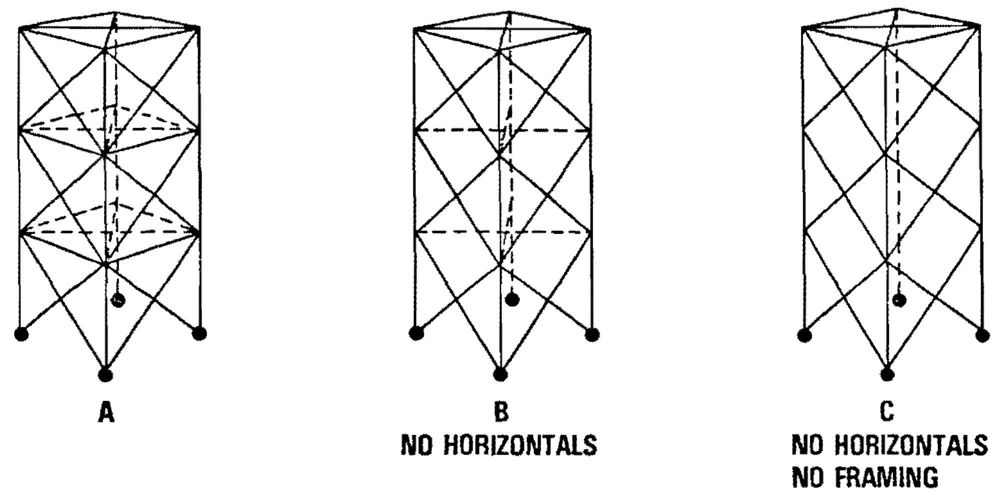


FIG. 4 FRAME VARIATIONS CONSIDERED IN EXAMPLE
(BACK SIDE X-BRACES NOT SHOWN FOR CLARITY)

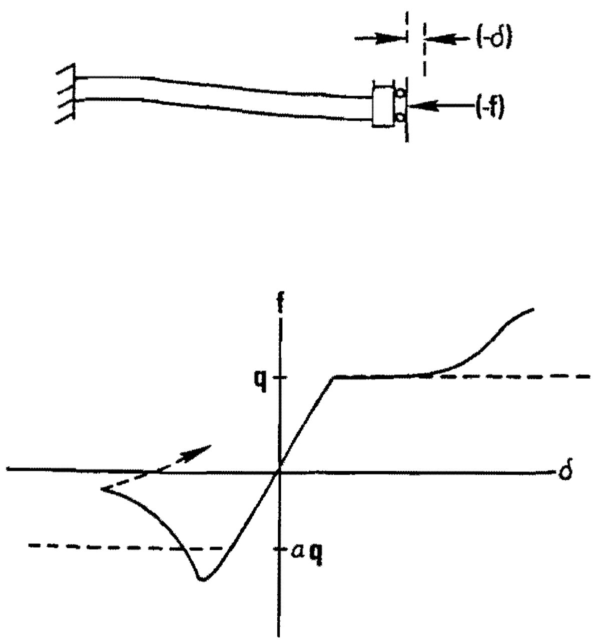


FIG. 5 ELASTIC PERFECTLY PLASTIC REPRESENTATION OF ACTUAL MEMBER BEHAVIOR

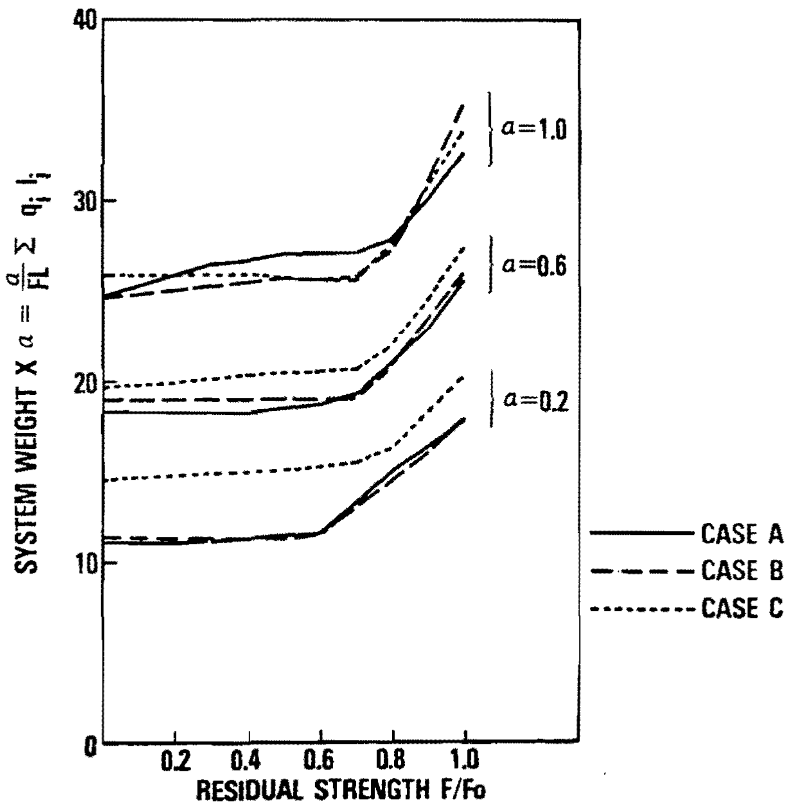


FIG. 6 SYSTEM WEIGHT FOR SPACE FRAME EXAMPLE

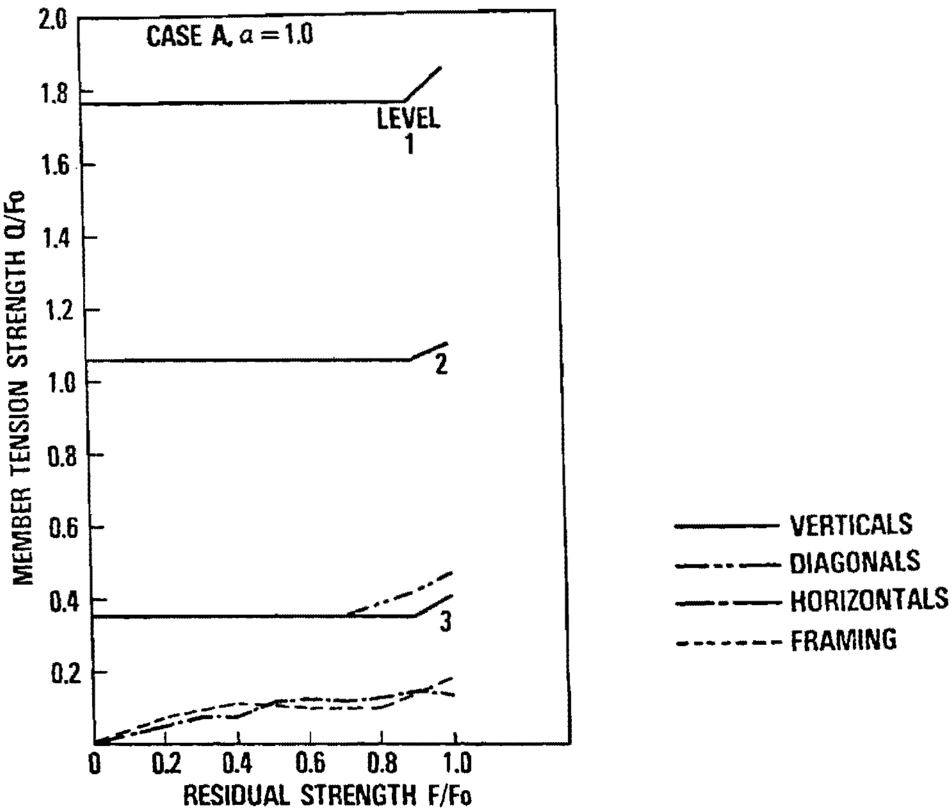


FIG. 7 OPTIMAL SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE A, a=1.0)

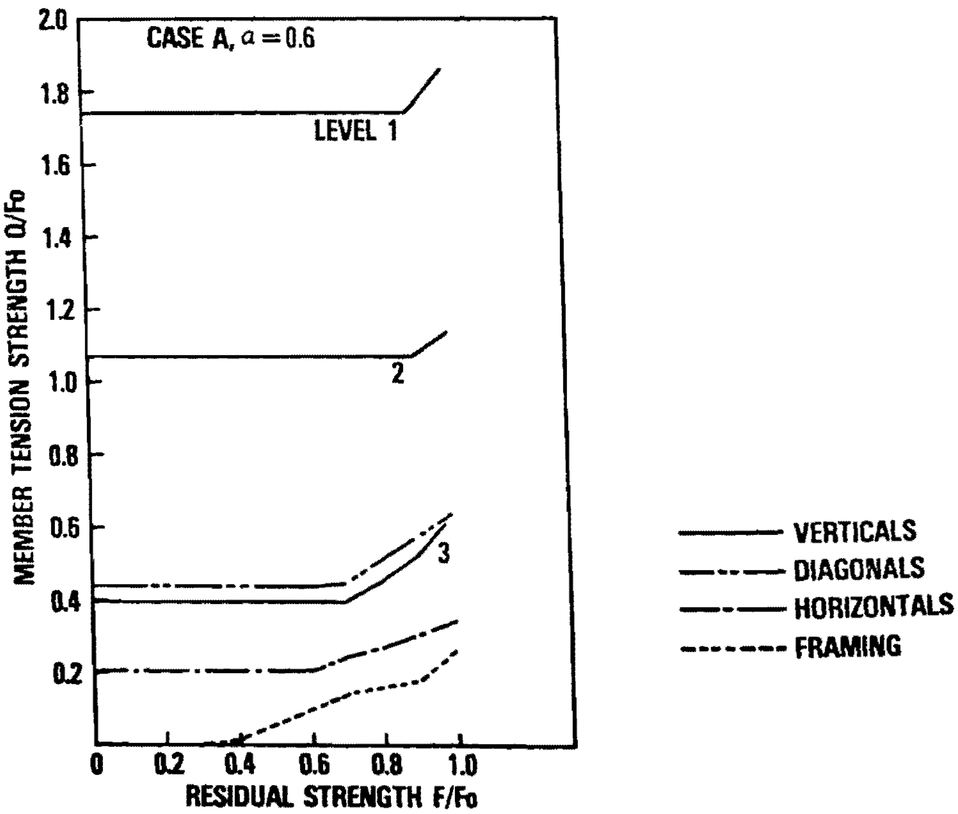


FIG. 8 OPTIMAL SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE A, a=0.6)

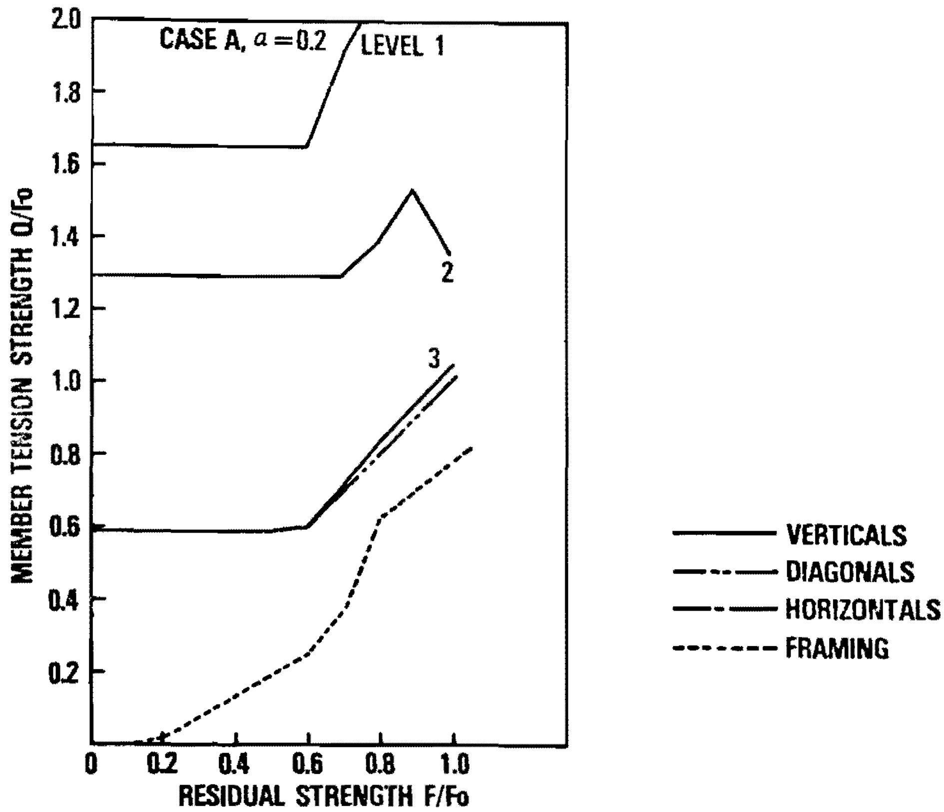


FIG. 9 OPTIMAL MEMBER SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE A, a=0.2)

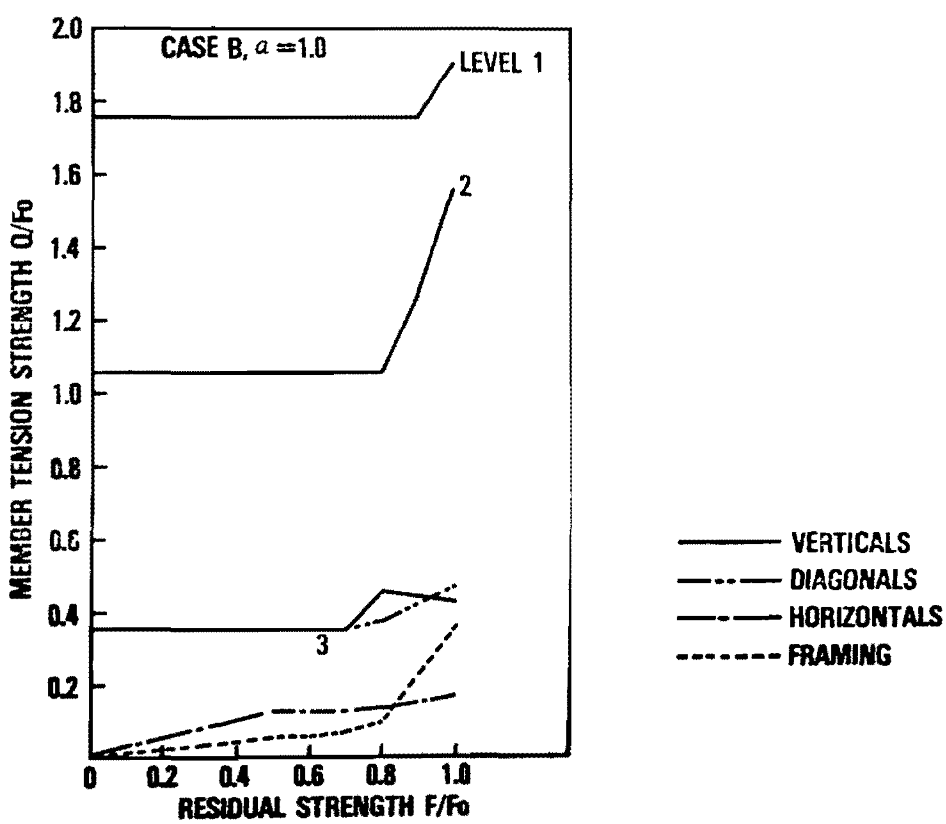


FIG. 10 OPTIMAL MEMBER SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE B, a=1.0)

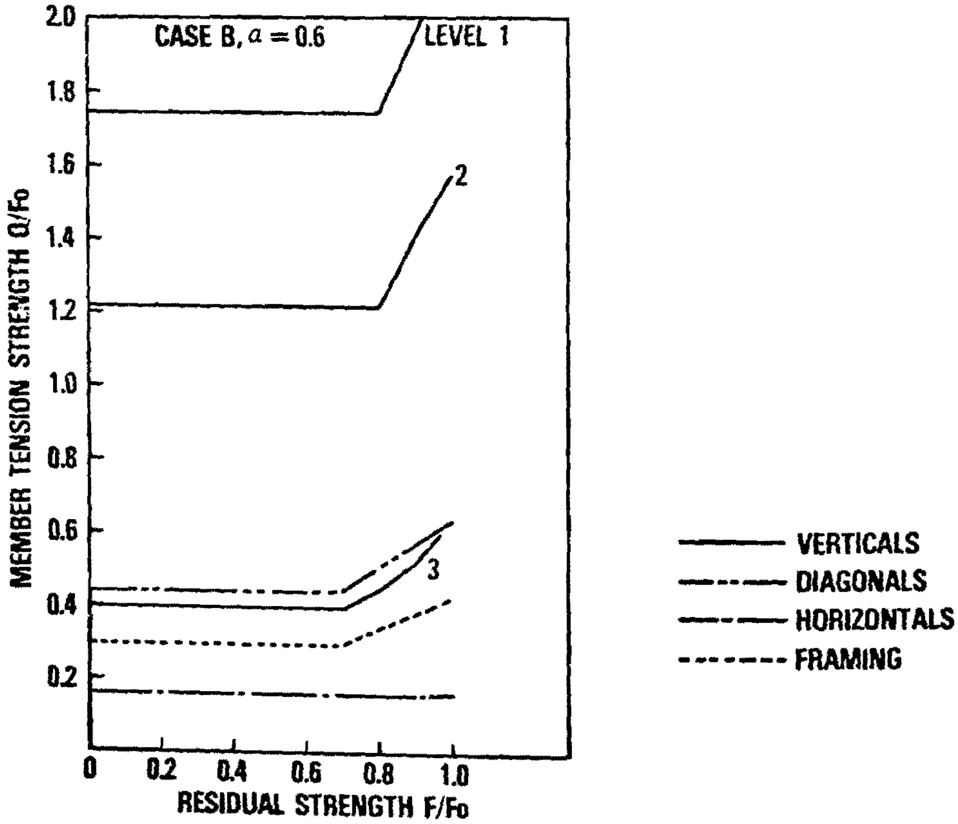


FIG. 11 OPTIMAL MEMBER SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE B, a=0.6)

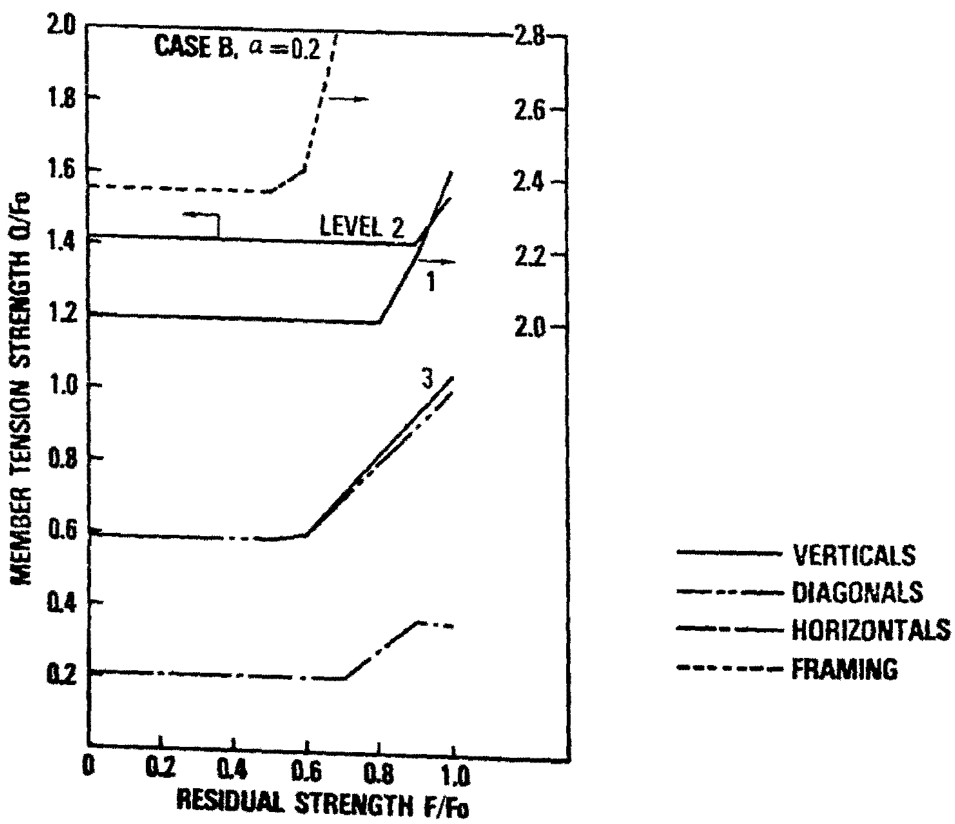


FIG. 12 OPTIMAL MEMBER SIZES FOR SPACE FRAME EXAMPLE -DIAGONAL MEMBER REMOVED (CASE B, a=0.2)

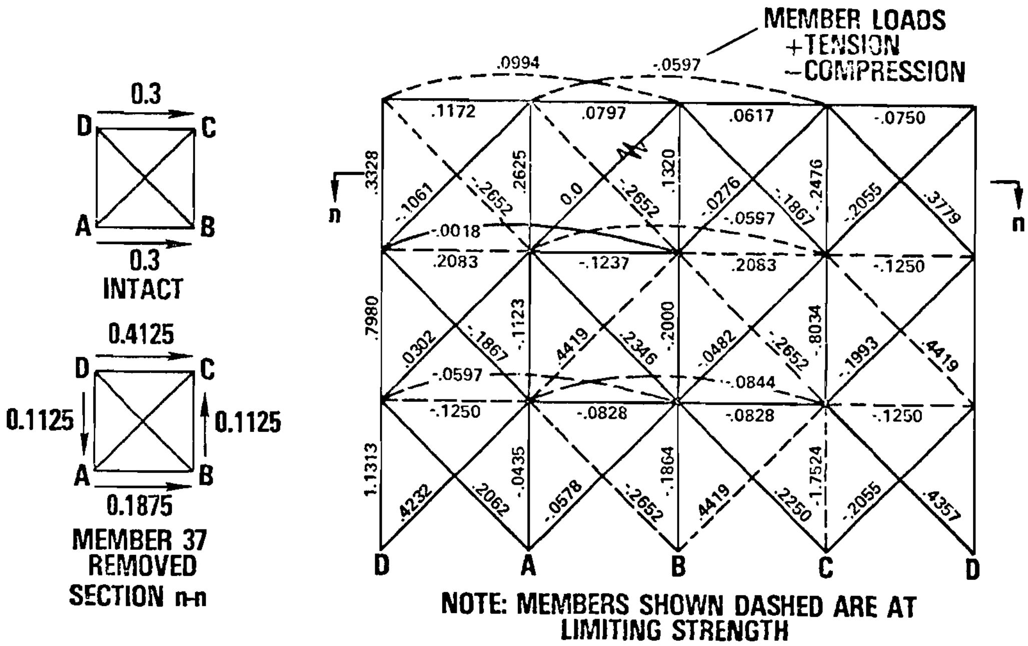


FIG. 13 LOAD REDISTRIBUTION OPTIMIZATION SOLUTION FOR EXAMPLE FRAME (CASE A, $a=0.6$, $\Theta=0°$, $F/F_o=0.6$)

DISCUSSION

Jack Sybert (Chevron): I do not agree with Jim Lloyd's concept of redundancy. I think that a code writer tells an engineer what the strength of a member is and lets the engineer include the strength in engineering fashion. If the engineer designs a redundant structure with the same amount of steel as a non-redundant structure, the redundant structure will have less reserve. Redundancy adds weight and cost.

Lloyd: I think the code writer must be aware of how a code is interpreted. Most code writers write codes to be used in a particular way. If the codes are interpreted in different ways, the reserve will be different. So, in a way, I agree with you.

J. W. Hotchies (Algoma Steel): What additional requirements would you propose for very cold environments?

Marshall: In water, the temperatures are not very different from the Gulf to the Hibernia Fields, so no special requirements are needed. For above water, use steels with high toughness. Above water temperatures require property toughness specifications.

S. X. Gunzelman (Brown & Root): Would you comment on redundancy of large-diameter type structures?

Marshall: Stiffening will provide a form of redundancy.

Lloyd: For platforms with very large-diameter members, my approach doesn't apply. The loss of a diagonal can be accounted for by high bending loads carried by the large-diameter members.

G. Foss (Det Norske Veritas): DNV has had much of the same discussion on redundancy. I think it is impossible to require redundancy in general. One feasible alternative is to make requirements in terms of loadings, such as accidental loadings. In that way, you consider loadings that might actually occur.

Meith: I might comment that Jim Lloyd's API committee is working to define and quantify redundancy.

J. Singer (Israel Institute of Technology): I have several comments. First, there is some old data on cones. Second, corrugations may be important in tubulars because corrugated shells have desirable buckling characteristics. I will be happy to provide information on boundary conditions for anyone who is interested. Third, imperfections in large specimens should be collected internationally in a common data base. Here, we will be able to correlate large and small scale specimens.

Miller: We are currently testing a number of ring and stringer stiffened cylinders in a joint-industry project, which was originally started by Conoco and the American Bureau of Shipping. We are measuring deflections initially and during testing. We are also testing two ring-stiffened cylinders in order to duplicate tests that were performed at the David Taylor Model Basin. I fell that the David Taylor tests were not representative because the welds that attached the ring stiffeners were three times the thickness of the stiffeners. We are also becoming more successful in fabricating cylinders out of much thinner materials.

D. Faulkner (University of Glasgow): I have two comments. First, I think it is important to separate the effects of residual stresses and initial imperfections in our analyses. These effects should be included in Professor Singer's data base. Also, the academic community is becoming more interested in inelastic analyses. I also have a question. How do you (Pete Marshall) evaluate the coefficient of variation for loads?

Marshall: We use distribution results of hindcasts for typical structures in the Gulf of Mexico, and we include uncertainties in waves, drag forces, and wave forces.

J. L. Durkee (Consulting Structural Engineer): Pete, please define hindcasts.

Marshall: Hindcasts are series that recreate wave heights and currents from winds in historical records.

D. Lai (Amoco Production Company): I would like to share with the audience work we have performed on critical loads of a typical member of an offshore structure. We calculated the effective length factor of the member to be less than that recommended by API. Also, we calculated the resultant stiffness of the structure at the ends of the member to be greater than that given in the DNV Rules. Since we have sophistocated numerical techniques available, API and DNV should consider updating their rules.

Marshall: Was your analysis based on an x-braced or single braced member?

D. Lai: The analysis was based on a single braced member.

C. Marsh (Concordia University): Did you include the effect of member relaxation in your analysis?

D. Lai: No, but we hope to in this comming year's work.

Lloyd: David, I agree that we have enhanced our numerical ability. Before, we used simple methods with lost of conservatism. Now as we become more sophisticated, we must make sure we include all effects in our analyses. If we miss something, like the effect of joint flexibility in David's analysis, we are in danger of making a mistake. I think we should use our tools, but we must take adequate care.

G. Foss: I would like to add that DNV's Rules will also come up with K lower than API's Recommendations. A lot of things enter a design check. If averages vary by twenty percent, there may be no signficant difference.

T A S K   R E P O R T E R S

TASK REPORTER 13 - LOCAL INELASTIC BUCKLING

L. W. Lu - Lehigh University

Local and Overall Mode

G. Askar, Lehigh University

Structural elements using thin plates have found increasing applications due to their economic advantages. In the design of a compression member of this kind, one of the basic problems in stability is the interaction of local and overall stability modes. This study presents a theoretical analysis for the general stability behavior of a thin walled box-column under axial loads (Fig. 1).

For such elements instability occurs either with the buckling of its individual elements or with the overall buckling of column itself. If the dimensions of the cross section are of the same order as the length of the column, the mutual reinforcing in the joining edges of the walls against deformations and rotations is negligible and these elements act like simply supported plates. This mode of buckling in the extreme is known as local buckling mode. On the other hand, in a case where the cross section dimensions are small relative to the column length, the walls deform collectively and an Euler type of column buckling occurs. In reality however, the behavior of an intermediate length column is governed by the interaction of both extremes as illustrated in Fig. 2. In this study an analysis of the elastic stability of box-type columns by folded plate theory based on exact and approximate solutions is given. The exact analysis consists of the exact solution of the differential equations. The approximate analysis, however, replaces the differential equations by a variational expression. Linear and quadratic polynomials multiplied by a trigonometric function in the axial direction were used for in-plane and out-of-plane deformations.

The analysis applies in particular to the intermediate range as well as the extreme cases of large and small cross sections. In the formulation, each panel is subjected simultaneously to plate and membrane actions. The simultaneous use of both in-plane and out-of-plane actions provides a general treatment of the three dimensional problem. The critical buckling curves are obtained by solving the eigen value problem for the overall system. The material is assumed to remain elastic throughout for all values of the parameters and plastic effects are not considered. Clearly for certain values of parameters, especially for local buckling it is expected that for many materials the elastic buckling range will be exceeded.

The results of the study are presented in the Figures 3 through 6 in terms of nondimensionalized critical load and wave length for various values of thickness, cross section dimensions and deformation mode order m. Figures show that the wave length λ is the important parameter. Indeed as λ becomes small, all the stability curves corresponding the various m's coincide. In the range of small λ's, the cross section b and c are comparable to the wave length and the reinforcement of each of the walls by the others at least. Hence the stability strength of the column is determined by the behavior of the individual walls. On the other hand for large values of λ, i.e. when the wave length of the buckling mode is much larger than the cross section dimensions, the stability is determined by the collected behaviour of walls. In this limit, as the wall dimensions are small, the walls reinforce each other and they deform as a whole, and the hypotheses of the one dimensional beam theory are valid. Indeed it is observed that the curves according to the Euler and the exact analysis coincide. The region of maximum value of stability parameters are those of interaction buckling range. It is also known as simultaneous design region. However, this region is extremely sensitive to imperfections.

The effect of wall thickness/length ratio h/a is seen on the interaction region. This region moves in the direction of increasing wave length with decreasing h/a ratio. This means that the wave length is not the sole criterion for determining the type of stability as overall. For thinner walls, the column will fail by local buckling even for quite slender cross sections.

The effect of cross section dimensions (rectangular or square) is readily understood by noting that the local stability is determined by the weakest wall behavior. In this regard, the stability parameters in the local stability region are smallest for the square section.

In addition to the exact solution the results according to the approximate solution based on a variational principle is presented in Fig. 7. It is seen that the approximate solution yields extremely accurate results.

The stability analysis for box-colúmns with stiffened walls is also studied here as they are widely used in thin-walled structures (Fig. 8). Stiffeners are assumed to be equally spaced. Hence, a typical panel of the column is represented by a plate having the properties of an orthogonal anisotropic material. The fundamental equations of the problem are obtained by using orthotropic plate theory and orthotropic field equations in elasticity. Clearly the theory gives best results when the spacings of stiffeners are small. This theory applies also to materials such as composites, plywood and natural wood which are similarly representatives of orthotropic materials.

Stability curves for two different type of materials, stiffened plate and natural wood are obtained by using orthotropic analysis (Fig. 3-11). Each curve is compared with the one obtained from the previous analysis which considered only isotropic material. In Figs. 11 and 12 the elasticity modulus and the plate rigidity in the principal directions are increased in the same order as it is usually the case in naturally fiberous materials. This means that the Young modulus in the plane stress does not differ from that of bending. In the presence of longitudinal fibers the column strength in the overall buckling range is increased. No effect is seen on the local buckling strength. Transverse fibers, on the contrary, effects the local buckling strength considerably.

Fig. 11 is an example for longitudinally stiffened plates and represents the case where Young modulus on the plane stress differs significantly from that of bending. For the case shown, when the ratio of the elasticity module is increased in the order of 2, then the ratio of bending rigidity is obtained in the order of 60. Plate and stiffeners have the same isotropic material properties. The effect of longitudinal stiffeners is seen on both local and overall buckling strength of the column. However, considerable increase is seen only in the overall buckling strength.

References

1. Goldberg, J. E. and Leve, H. L. 1957
   THEORY OF PRISMATIC FOLDED PLATE STRUCTURES,
   IABSE, Publication 1957, Vol. 17

2. Graves-Smith, T. R.
   THE ULTIMATE STRENGTH OF LOCALLY BUCKLED COLUMNS OF ARBITRARY LENGTH, Thin Walled Steel Structures,
   p. 35, Crosby Lockwood, London, 1969

3. Maquoi, R. and Massonet, C.
   INTERACTION BETWEEN LOCAL PLATE BUCKLING AND OVERALL BUCKLING IN THIN-WALLED COMPRESSION MEMBERS.
   Theories and experiments, Buckling of Structures,
   Synposium, Cambridge/USA, 1974

4. Lechnitzby, S. G.
   THEORY OF ELASTICITY OF AN ANISOTROPIC ELASTIC BODY
   Holden-day, Inc., 1963

5. Askar, G.
   THE STABILITY ANALYSIS OF THIN-WALLED BOX COULMN BY FOLDED PLATE THEORY, Thesis, Istanbul Technical University, 1977

6. Natan, B. R., ed.
   COMPOSITE MATERIALS <u>3</u>: ENGINEERING APPLICATION OF COMPOSITES, 1974

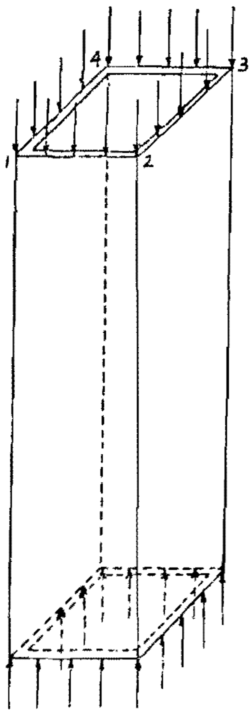


FIG. 1

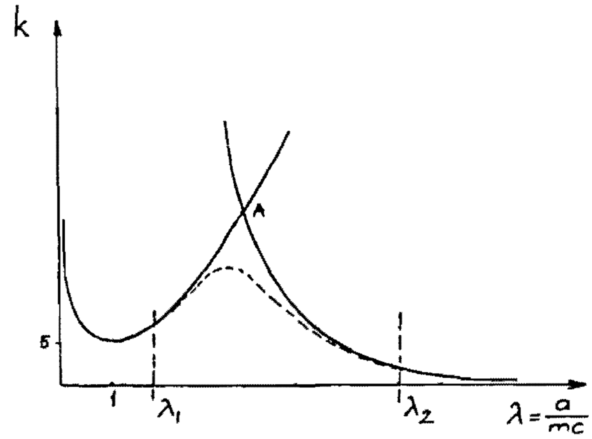


FIG. 2

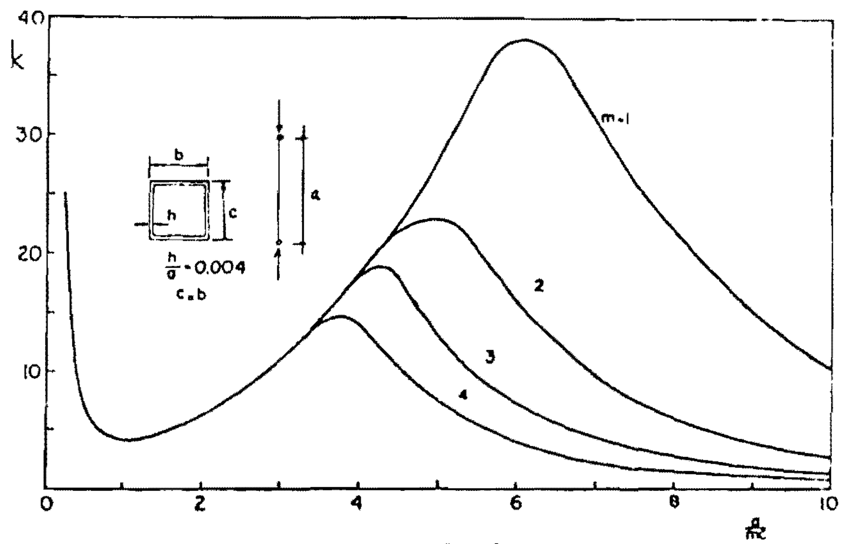


FIG. 3

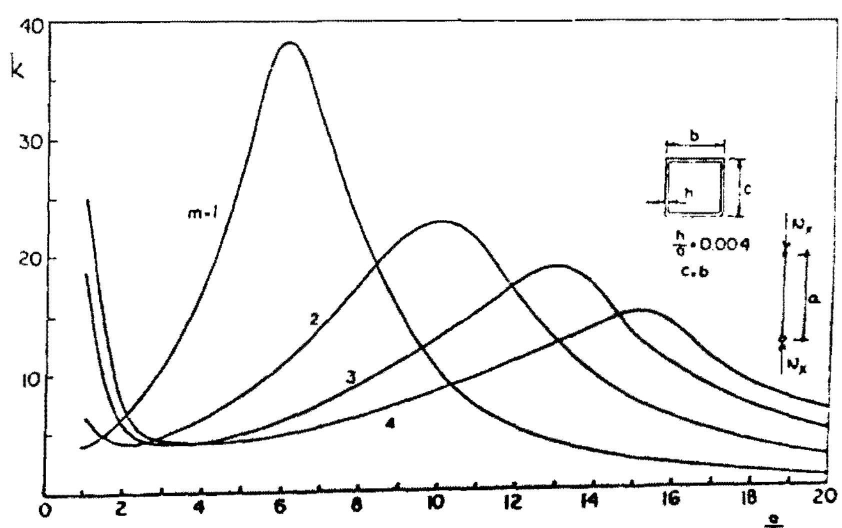


FIG. 4

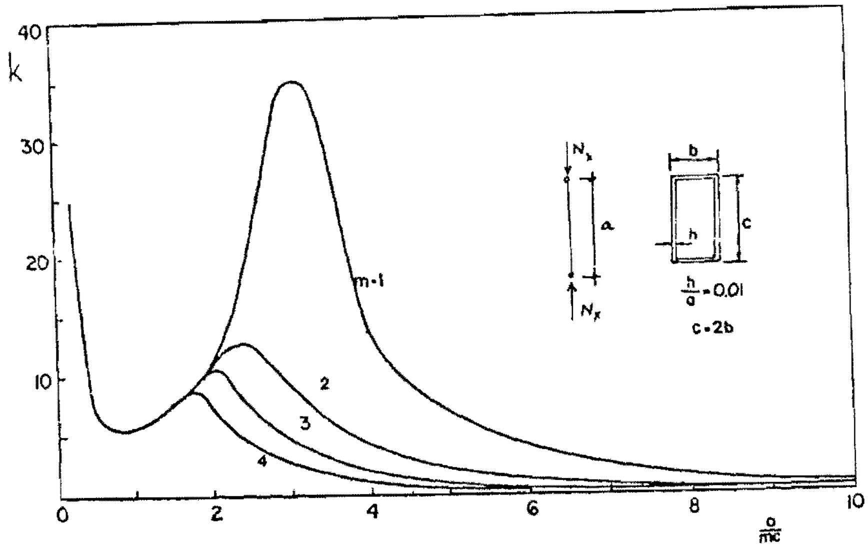


FIG. 5

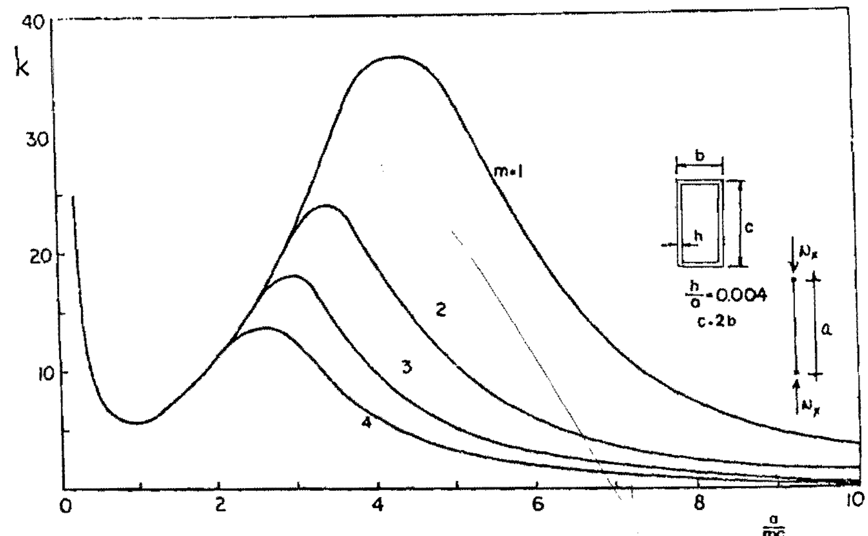


FIG. 6

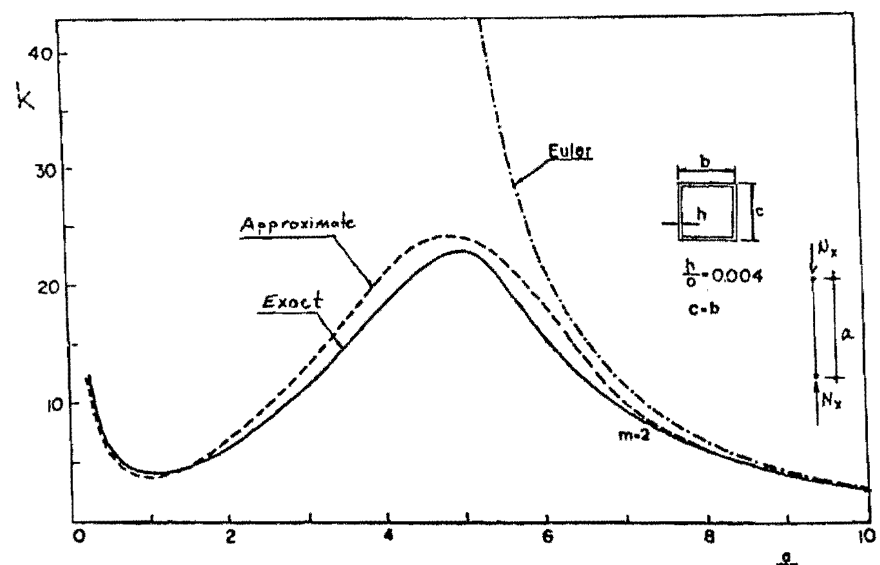
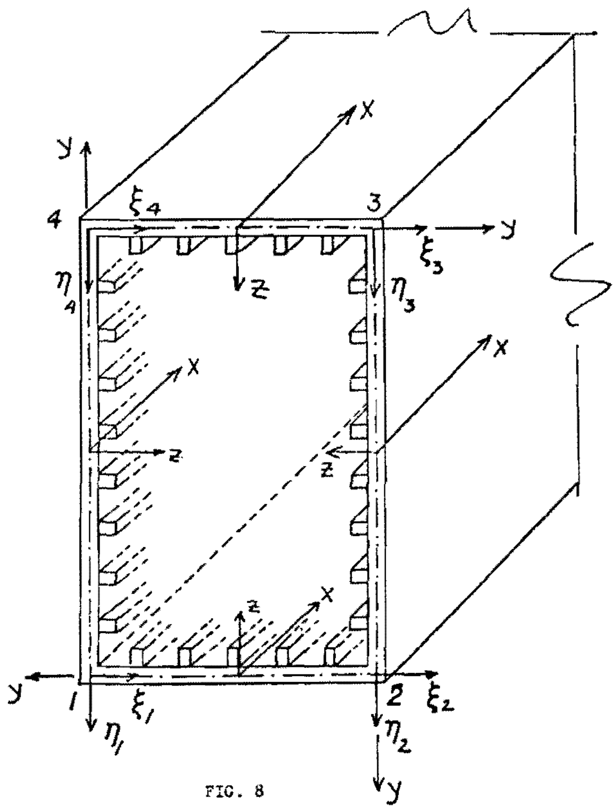
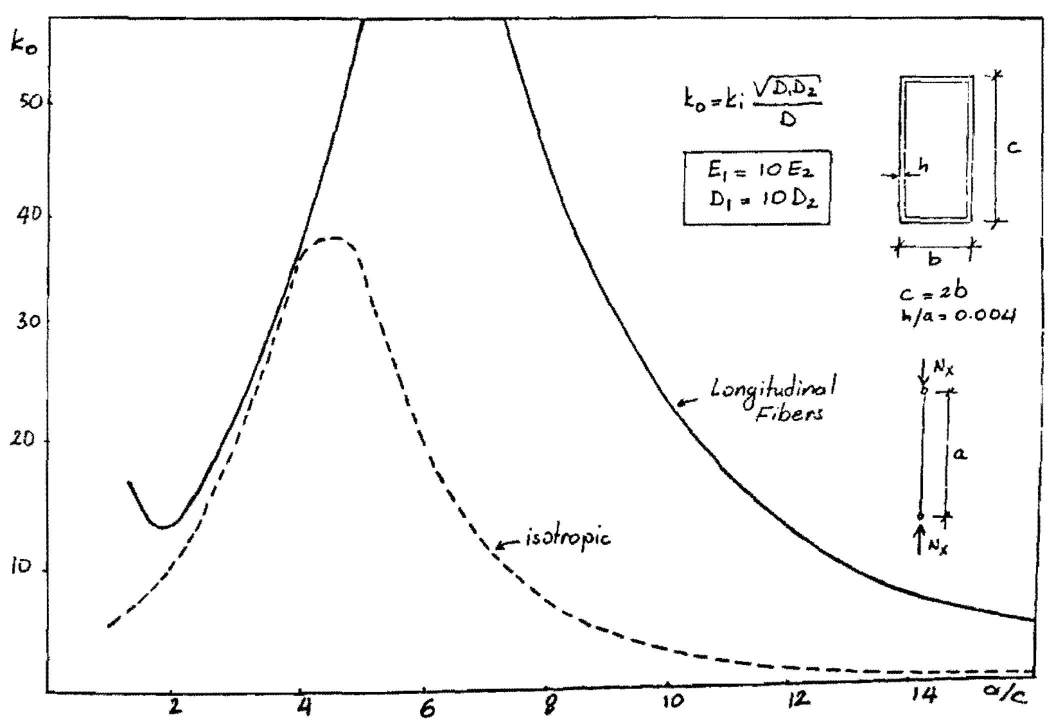


FIG. 7



y
x
4
ξ4
3
y
ξ3
η4
η3
z
x
z
z
x
z
x
y
1
ξ1
η1
2
ξ2
η2
y

FIG. 8



k_o
50
40
30
20
10
2
4
6
8
10
12
14
a/c
$k_o = k_i \frac{\sqrt{D_1 D_2}}{D}$
$E_1 = 10 E_2$
$D_1 = 10 D_2$
h
c
b
c = 2b
h/a = 0.004
N_x
a
N_x
Longitudinal Fibers
isotropic

FIG. 9

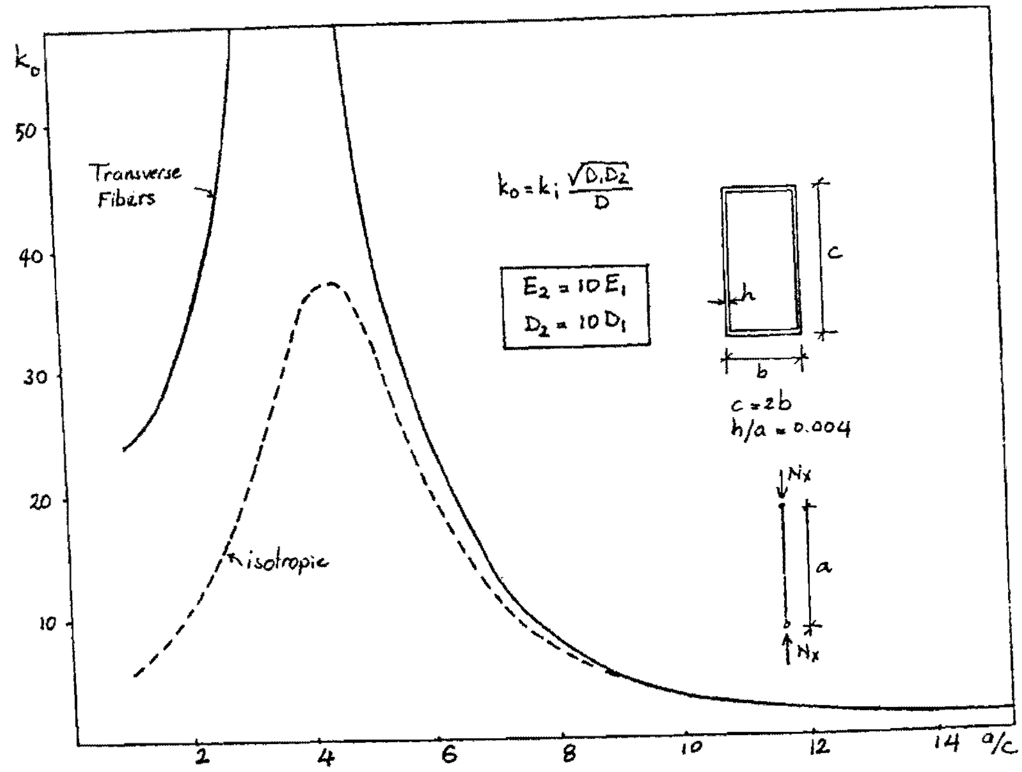

k_o
50
40
30
20
10
2
4
6
8
10
12
14
a/c
Transverse Fibers
isotropic
$k_o = k_i \frac{\sqrt{D_1 D_2}}{D}$
$E_2 = 10 E_1$
$D_2 = 10 D_1$
h
c
b
$c = 2b$
$h/a = 0.004$
N_x
a
N_x

FIG. 10

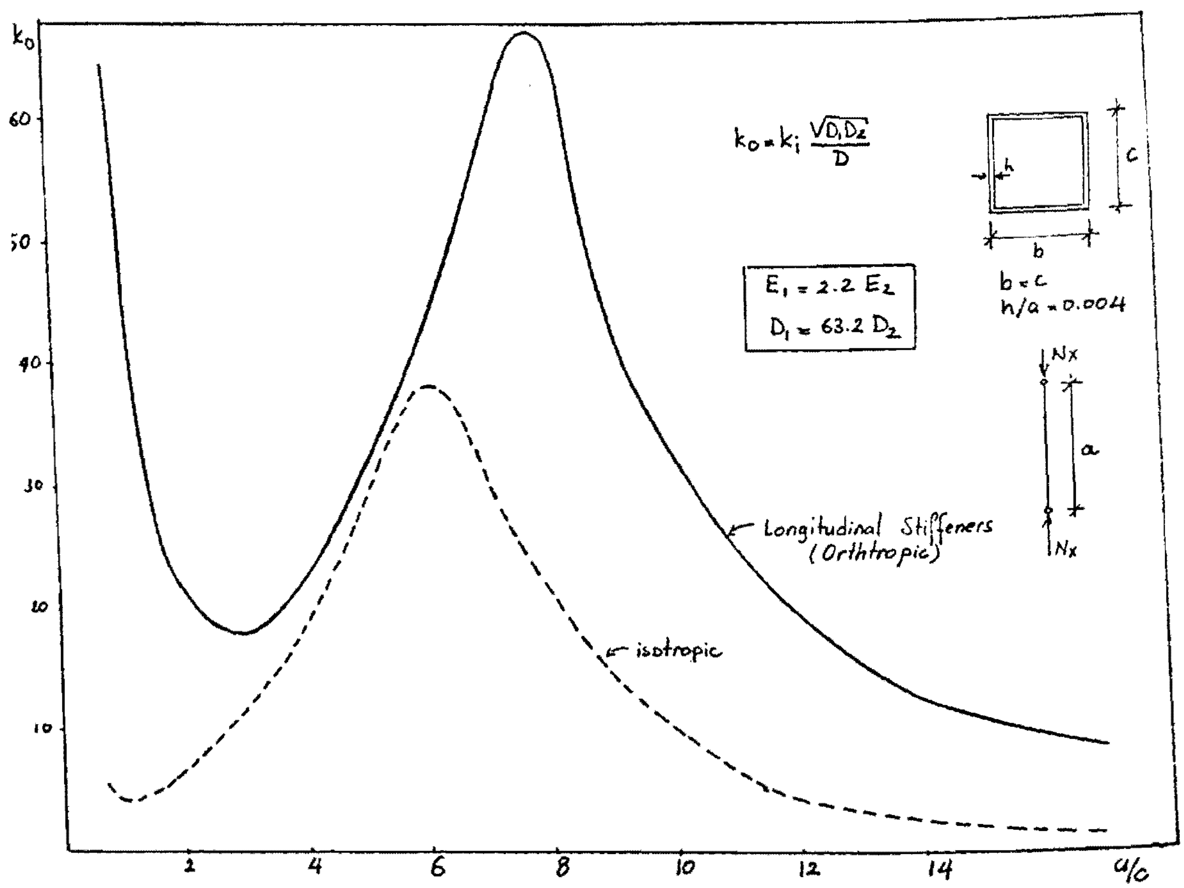

k_o
60
50
40
30
20
10
2
4
6
8
10
12
14
a/c
$k_o = k_i \frac{\sqrt{D_1 D_2}}{D}$
$E_1 = 2.2 E_2$
$D_1 = 63.2 D_2$
h
c
b
$b = c$
$h/a = 0.004$
N_x
a
N_x
Longitudinal Stiffeners (Orthtropic)
isotropic

FIG. 11

TASK REPORTER 14 - FIRE EFFECTS ON STRUCTURAL STABILITY

K. H. Klippstein, U. S. Steel Corporation

Update on 1980/81 Developments Related to the Stability of Structures Exposed to Fire

Since last reporting at the April 1980 Annual Technical Session and Meeting, significant research work and studies have appeared in the literature. A review of the eight listed references follows. The references cover the stability of structural members such as columns and beams, as well as composite floor slabs consisting of steel deck and concrete.

Specifically, the first reference deals with cold-rolled thin-walled beams and columns exposed to the ISO 834 fire standard, which in similar to the ASTM E119 standard. The specimens varied in thickness from 0.078 inch (2 mm) to 0.128 inch (3.25 mm) and were covered with three types of insulation such as sprayed mineral fibers, insulating board, and plaster. The analytical study and test results show that thin-walled steel members can be adequately protected with some of the currently available insulating materials to perform structurally for a predicted time period; however, several questions still need further research. The results of this study may provide valuable data for correlation with those of the AISI study on load-bearing studs, which was brought to your attention in the 1980 report.

The second reference introduces reliability concepts to the fire-safety analysis of steel columns. This analytical study compares the relative safety index of the protected hot-rolled columns in the upper stories of a high-rise building to the unprotected built-up columns in the ground floor or basement, which is said to be the common way high-rise buildings are designed. Obviously, the upper protected columns have a much higher safety index. Alternative insulating materials with different heat capacity, density, and coefficient of variation (COV) are suggested for the ground-floor or basement columns to achieve a more uniform safety index for all columns without losing usable floor space.

Two of the hypotheses which contributed to the development of the American Iron and Steel Institute (AISI) column-design guide described in the 1980 Task Report, are that (1) the temperature in the fire-exposed column is uniformly distributed, and (2) the column collapses or becomes unsafe at 1000°F (540°C). The authors of Reference 3 welcome the simplicity of design these hypotheses allow; however, they point out that efforts must be undertaken to assure that the design-guide procedures are also applicable to fire-exposed columns under loaded conditions and to column orientations for which slight temperature gradients in the column cross section do not adversely affect the column stability. Seventeen tests were conducted on column and beam-column substructures with various end-fixity and connection conditions when exposed to natural fires and axial loads. Test results showed that significant temperature gradients can occur in the fire-exposed or unexposed cross sections of the columns; the gradients tend to increase with the distance of the column to the facade when the column is not surrounded by flames. One of the tests resulted in a hot-flange temperature of 1460°F (795°C) and a temperature gradient of 500°F (280°C); although loaded to 27 ksi (185 N/mm^2), the column was still capable of sustaining the load without buckling because the column ends were relatively rigid. On the other hand, the same column could only be stressed to 12 ksi when the ends were pinned. The information collected is very valuable in helping to understand the performance of fire shields, high-strength bolts, and structural-failure mechanisms.

Eleven examples of water-filled structural members (columns and also some beams) are described in Reference 4. Ten of these structures involved are located in Europe. Round and rectangular columns are shown.

Reference 5 presents an analytical approach to predict the fire response of a structure, such as a composite or a noncomposite steel-deck structure. A finite-element computer program is used to predict the thermal and structural response, including the time-temperature history, mid-span deflection, end rotations, and elongations. The results compare well with ASTM E119 fire tests conducted at the Ohio State University. This AISI-sponsored work will be reviewed by D. C. Jeanes.

The results of a study on fire experience and fire exposure of structures for fixed-guideway transit systems are described in Reference 6. Only the larger rapid-transit systems in the United States and Canada were studied, but accident and fire reports were accumulated back to the time of construction before the turn of the century. Most recent and comprehensive data were obtained from the National Fire Protection Association (NFPA), Fire Information Data Organization (FIDO), which began its computerized collection of data in 1971. A total of 114 incidents were reviewed. The report concludes that ". . . the use of exposed steel (structural members) to support elevated fixed-rail guideway systems . . . or . . . in underground fixed-guideway rail systems on station platforms and in concourse areas . . . is effective from performance standpoints in all but the most unusual situations . . . provided the transit authority controls the combustible content loading in token booths and concessions . . ." Use of noncombustible construction materials such as structural steel is recommended.

The authors of Reference 7 propose a new method to analyze steel-framed structures exposed to fires. The method appears to go beyond determining the collapse temperature of a structural member, giving a full description of the behavior and development of stresses and strains by using "restricted basis linear programming." An example of the proposed method is provided, but the analytical results are not compared with physical test results.

The author of Reference 8 reviews the design philosophy of fire engineering and the design approaches that can be used to prevent collapse of a structure exposed to a fire. Statistical data on fires are examined to justify alternative design approaches such as a full or a partial structural analysis under real or standard fire conditions, consideration of sprinklers, and fire tests of the most vulnerable portion of a proposed structure.

As concluded by the author of the last reference, fire engineering of structures is a recently developed subject. Although it has progressed rapidly, there are many areas where further research and development is required before a complete understanding of the subject is achieved and meaningful code requirements are developed.

References

1. R. D. Homer, "The Resistance of Cold-Rolled Steel Profiles to Fire (Research Project)," Prepared for European Coal and Steel Community, Convention No. 6210/SB/8/801, June 1, 1975 through May 31, 1979.

2. Benjamin Koo, "A Reliability Based Fire-Safety Analysis for Steel Columns," Proceedings, ASCE/EMD Specialty Conference, University of Texas, Austin, Texas, September 17 through 19, 1979.

3. J. Kruppa, "Result of Research E.C.S.C.: Behavior of External Steel Columns in Fire," Report C.B.L.I.A. 55.3, Acier-Stahl-Steel, pp. 66-74, February 1980.

4. "The Hollow Section," English Edition by CIDECT (Comité International pour le Dévellopement et L'Etude de la Construction Tubulaire), Volume 3, ISSN 0143-4012, October 1980.

5. Boris Bresler and Robert Iding, "Response of Steel Buildings to Fire," Preprint 80-595, ASCE Convention and Exposition, Florida, October 27 through 31, 1980.

6. "Fire Experience and Exposure in Fixed-Guideway Transit Systems," Prepared by Schirmer Engineering Corporation and Rolf Jensen & Associates, Inc., for American Iron and Steel Institute, December 1980.

7. Roberto Contro and Gilvio Giacomini, "A Method for Analyzing Frame Structures Exposed to Fires," Construzioni Metalliche, No. 2, 1981.

8. B. R. Kirby, "The Philosophy of Fire Engineering," British Steel Corporation, Teesside Laboratories, Rails and Sections Department, Report No. T/RS/1189/15/81/C, July 10, 1981.

Stability of Fire Exposed Structural Steel Building Floors - A Computer Model and Full-Scale Test

D. C. Jeanes, American Iron and Steel Institute

Due to the technically complex interplay between a building fire and the surrounding structural frame, fire endurance has traditionally been assessed by the use of standardized tests (1) conducted on assemblies of representative construction. Although this approach provides a comparative measure of fire endurance between different types of construction it does not adequately address actual structural performance during exposure to a fire. Factors such as restraint against thermal expansion, redistribution of load, and moment resistance, although recognized to exist, are difficult to quantify and duplicate in a test. With a better understanding of building fire growth, changes in material properties, and structural performance, engineering problems of this magnitude are now solvable using computer aided design. A computer program has been developed (2, 3) and a research test program is now underway to evaluate the results. (4)

The structural computer model, FASBUS (Fire Analysis of Steel BUilding Systems), is a structural analysis program using the finite element method. It is specifically designed to model two dimensional structural floor systems using beam elements and nonconforming triangular plate bending elements, to represent the beams and floor slab respectively. The element stiffnesses, derived using temperature-dependent nonlinear stress-strain material models for both steel and concrete, contribute to the determination of the resulting structural stiffness of the system. Through an iterative process, the model determines the displacement conditions which satisfy the nonlinear equations of equilibrium at each of the element nodes. As a result, a determination is made of the deflected shape of the frame and the conditions of stress and strain throughout the depth of each structural element. A more detailed description of the computer model and the modeling techniques are contained in references 2 and 3.

The program was initially used to analyze existing test data on both composite and noncomposite beam designs, in "restrained" and "unrestrained" assemblies. (3) The prediction of structural fire endurance showed good agreement with the test results up to and including the point of instability. Such a condition was indicated by the development of a fully plastic section across the beam and the corresponding inability of the computer model to converge on a structurally stable solution.

Additional testing is now underway through a Research Associate Program between the American Iron and Steel Institute and the National Bureau of Standards. (4) The objective of this program is to develop data which will aid in a full evaluation of the computer model capabilities. In order to evaluate the interaction of a building framing system when partially exposed to fire, a large test structure was erected and instrumented. The test structure consists of a two-story four-bay frame with structural steel members sized to be representative of the mid-height of a mid-rise office building of stiffened core design. The test floor consists of a composite steel deck and two inch concrete topping. The details of the test frame were so chosen to provide a structure offering the least "built-in" restraint - considered to be in effect the "worst case" for analysis.

Performance of the structure during fire endurance testing was determined by recording vertical and lateral deflections, temperature measurements, and visual observations.

The first test consisted of exposing one bay of the structure to a 100 minute, ASTM E119 fire. The limited exposure was intentionally limited (ie less than the 2 hour rating) during this early test in order to "save" the frame for additional testing.

As a result of this test, the computer analysis was found to demonstrate good agreement with the measured performance. The analysis provided a reasonably accurate prediction of the overall influence of the fire on the frame by close comparison with the actual deflected shape of the structure, both vertically and horizontally. In addition, conditions of material failures, such as cracking and crushing of the concrete slab and local overstressed conditions in the steel framing, indicate agreement in general with those conditions observed upon inspection after the test.

As with any finite element model, the predictions of FASBUS II are dependent on proper modeling of the structure, accurate material models and temperature dependent properties, and consideration of structural changes with respect to time so that numerical convergence is optimized.

The initial evaluation of the FASBUS II computer model, based on the results of this test, suggests that this approach to structural fire endurance will provide the designer with a better understanding and evaluation of structural performance of steel floor framing systems exposed to fires.

References

1. American Society for Testing and Materials, "Fire Tests of Building Construction and Materials," Standard Method E119, Part 18, 1980, Annual Book of ASTM Standards, ASTM, Philadelphia, PA.

2. Chiapetta, R. L., Longinow, A., and Stepanek, O. J., "The Effect of Fire Temperatures on Buildings with Steel Frames," Final Report IITRI Project J8096, IIT Research Institute, April 1972.

3. Iding, R. H. and Bresler, B., "Effect of Fire Exposure on Steel Frame Buildings," Final Report (Draft) WJE #78124, Wiss, Janney, Elstner and Associates, Inc., September 1981.

4. Jeanes, D. C., "Predicting Fire Endurance of Steel Structures," Preprint 82-draft, ASCE Convention, Las Vegas, April 26-30, 1982.

TASK REPORTER 16 - STIFFENED PLATE STRUCTURES

A. Mansour, University of California, Berkeley

Structural Reliability and Strength of Stiffened Plates

One of the barriers to the use and implementation of reliability methods in the stiffened plates of ship and offshore structure is the lack of adequate methodology to incorporate and reflect properly the material and fabrication imperfections in an actual panel. Statistical data pertaining to factors such as initial deformations, initial stresses, yield strength variability, fabrication tolerances and corrosion are in general available, but not in sufficient quantities to permit unquestionable and reliable statistical estimates of their probability distributions and other relevant parameters. The safety levels of stiffened panels as predicted from reliability methods have to be calibrated in order to establish acceptable level by current practice.

In this paper, some of the above barriers in implementing the reliability concepts to the analysis of ship and offshore stiffened plating will be addressed. The nominal strength of a stiffened panel is evaluated under biaxial compressive loads. The mechanisms leading to failure by major instability of the stiffened plate or its components are identified and discussed. Such mechanisms include column buckling, stiffener tripping and overall grillage instability.

The factors which influence the strength of as-built stiffened panel are identified and will be briefly discussed. Approximate formulations of the ultimate strength coefficients of variation (C.O.V.) for the different failure modes as functions of the C.O.V. of their constitutive factors will be also presented. Reference is made to some available methods for calculating the nominal strength in the different modes of failure [1-6] as well as to some of the available engineering data [7-10].

References

1. Mansour, A., "Charts for the Buckling and Post-buckling Analyses of Stiffened Plates Under Combined Loading," Technical and Research Bulletin No. 2-22, Society of Naval Architects and Marine Engineers, July 1976.

2. Mansour, A., "Gross Panel Strength Under Combined Loading," Ship Structures Committee Report, SSC-270, 1977.

3. Smith, C. S., "Compressive Strength of Welded Steel Ship Grillages," RINA Spring Meeting, 1975.

4. Dow, R. S.; Hugill, R. C.; Clark, J. D. and Smith, C. S., "Evaluation of Ultimate Ship Hull Strength," Proceedings of the Extreme Load Response Symposium, SNAME and SSC, October 1981.

5. Mansour, A., "Postbuckling Behavior of Stiffened Plates with Small Initial Curvature under Combined Loads," International Shipbuilding Progress, June 1971.

6. Mansour, A. and Thayamballi, A., "Ultimate Strength of a Ship's Hull Girder in Plastic and Buckling Modes," Ship Structure Committee Report No. SSC-299, July 1980.

7. Baker, M. J., "Variations in the Strength of Structural Materials and Their Effect on Structural Safety," Imperial College, London, July 1970.

8. Basar, N. S., and Stanley, R. F., "Survey of Structural Tolerances in the United States Commercial Ship Building Industry," Ship Structure Committee Report SSC-273, 1978.

9. NAVSHIPS 0900-060-4010, "Fabrication, Welding, and Inspection of Metal Boat and Craft Hulls," Department of the Navy, Washington, D.C., January 1971 and the previous NAVSHIPS 0900-000-1000 dated March 1966 and June 1968.

10. Mansour, A.; Faulkner, D., "Hull Girder Reliability," Chapter 14 of Ship Structural Design Concepts, Cornell Maritime Press, Cambridge, Md., J. Harvey Evans, Ed., also (short version) "On Applying the Statistical Approach to Extreme Sea Loads and Ship Hull Strength". Trans. RINA, Vol. 115, 1973, pp. 277-314.

TASK GROUP REPORTS

TASK GROUP 14 - HORIZONTALLY CURVED GIRDERS

Chairman: C. H. Yoo, Auburn University

Out-of-Plane Buckling of Circular Rings

C. H. Yoo, Auburn University

Derived is a new fourth order system of differential equations describing the elastic buckling behavior of spatial thin-walled curved girders including the warping contribution to the buckling load of anti-symmetric sections. The effect of antisymmetry in cross-section to the critical load has been found significant. As a special case of curved members, the stability of circular rings is presented.

An examination of a wide variety of examples reveals fairly significant results in that the out-of-plane buckling mode shapes are coupled in terms of flexure and torsion. Depending on the unfavorable orientation of members in terms of their cross-sectional properties, the critical loads of out-of-plane buckling of rings are frequently lower than those of in-plane buckling, thus necessitating further evaluation of lateral bracings on such structures as circular coffer dams. Comparative studies were made with a few existing solutions and the possible sources of discrepancies were traced.

TASK GROUP 16 - PLATE GIRDERS

Chairman: M. Elgaaly, Bechtel Associates

Stress Distribution in Buckled Shear Webs

C. March, Concordia University

A diagonal tension field, as proposed by Wagner (1), is an inappropriate model for the post-buckling behaviour of stiffened shear webs. Basler (2) showed clearly that there is no requirement for a strong flange to resist tension forces in order to develop a shear capacity well in excess of that causing initial buckling.

Traditionally, the buckling stress has been calculated on the basis of a uniformly distributed shear stress along the boundaries. If a distribution of stress is assumed which varies from a value equal to the initial buckling stress, at the corners at the ends of the long compression diagonals, to the limiting shear stress at the other corners, it can be shown that there is a stable system of shear stresses which can provide a capacity beyond buckling with no requirement for tension stresses normal to the flanges.

A photoelastic model of a square shear web has been used to demonstrate the existence of this stress field. The model, composed of a 10" x 0.045" web bonded to 2" x 0.125" boundary flanges, is seen in Fig. 1 carrying a load six times that required to cause initial buckling. Failure occurred in the web at the point of highest boundary shear stress, with the flanges still essentially straight.

It has been shown (3) that, with flexible flanges, diagonal tension will not occur until the web yields in shear at the most highly stressed corner, after which the flanges will be compelled to bend and contribute to the total shear capacity. This increase in capacity due to flange strength was shown to exist by Rockey (4). A photoelastic material is brittle, no yielding occurs and hence the flanges will not be subjected to lateral tension force prior to the rupture of the web.

Figure 2 shows the stress distribution obtained from analysis of the photoelastic model.

For a mathematical model, for a brittle web, the shear stress distribution along an edge is assumed to be given by:

$$\tau = \tau_u/[1 + (N - 1)k]^2$$

in which $N = (\tau_u/\tau_c)^{\frac{1}{2}}$, $k = x/b$, b = panel width, x = the distance along the edge, τ_c = initial buckling stress, τ_u = ultimate shear stress.

The stress contours corresponding to this model are shown in Fig. 3. The assumed stress distribution along the boundary is also shown, together with the values obtained from the test. Using this assumed distribution, the total shear force to cause a brittle failure is predicted to be given by:

$$V = (\tau_c \ \ \tau_u)^{\frac{1}{2}}\, bt$$

For the specimen tested this gave a value of 330 compared to the test value of 400 lbs.

<u>REFERENCES</u>

1. Wagner, H., "Ebene Blecktragen mit dunnen Stegblech", Zeitshrift fur Flugtechnik Motorluftschiffart Vol. 20, 1929.

2. Basler, K., "Strength of Girders in Shear," ASCE, Proc. No. 2967, Part 1, ST7, 1961.

3. Marsh, C., "A Theoretical Model for the Collapse of Shear Webs." ASCE, Journal. Eng. Mech., (Accepted).

4. Rockey, K. C., Skaloud, M., "Influence of Flange Stiffener upon the Load Carrying Capacity of Webs in Shear", IABSE, 8th. Cong., Final Report, New York, 1968.

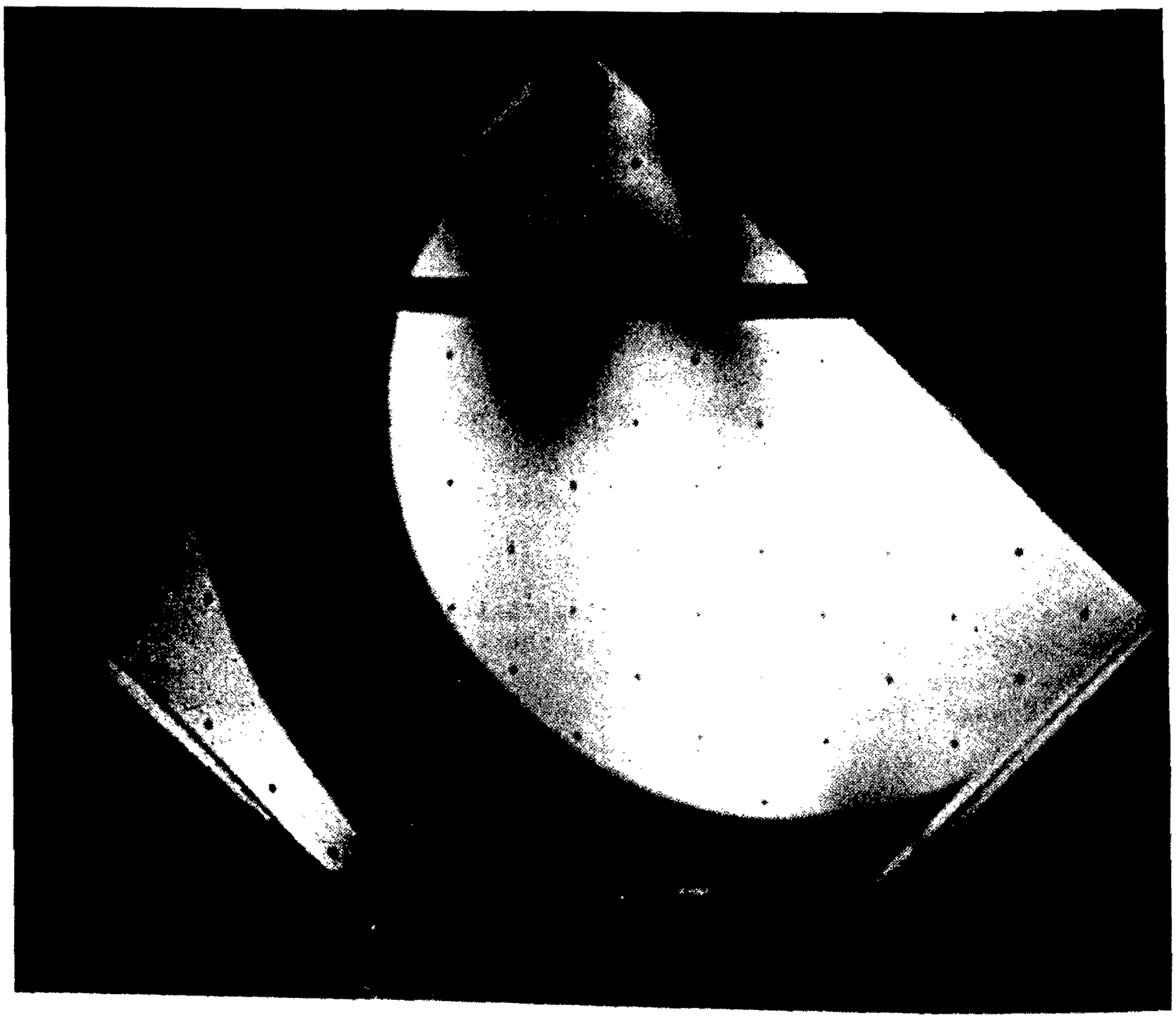


Fig. 1 Photoelastic model of the post-buckled shear web

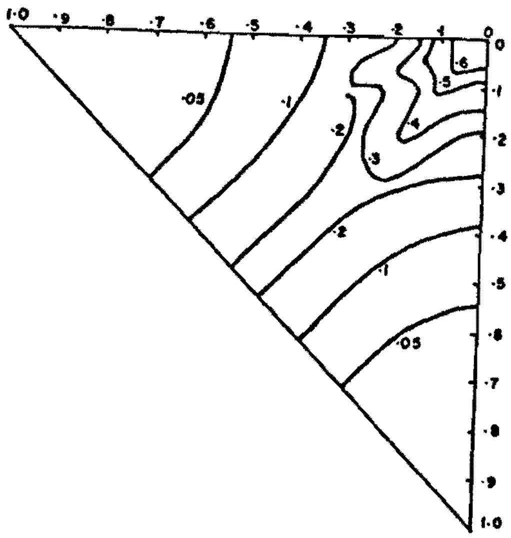


Fig. 2 Contours of shear stress from test

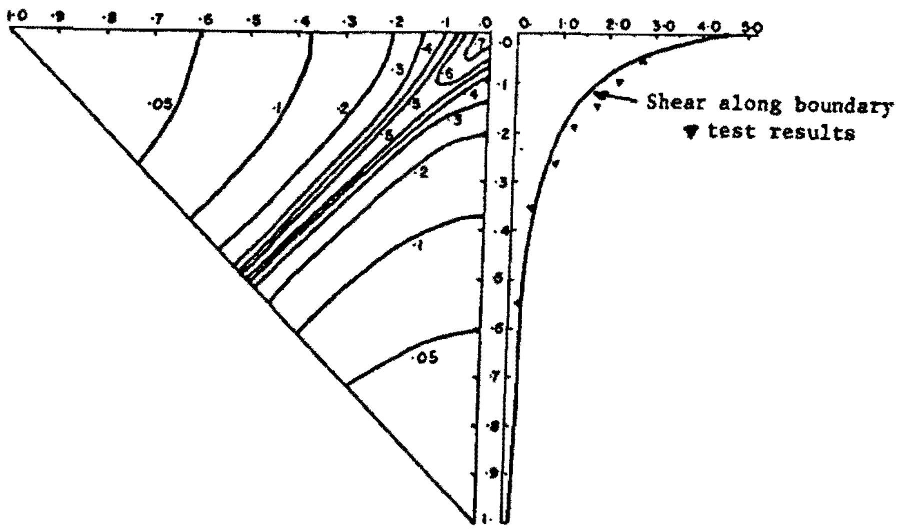


Fig. 3. Theoretical stress contours and shear stress distibution along the edge with experimental values.

TASK GROUP 21 - BOX GIRDERS

Chairman: R. C. Young, Iffland Kavanagh Waterbury, P.C.

Steel Box Girders Subjected to Torsion, Bending and Shear

A. Ostapenko, Lehigh University

A method was developed for analyzing the ultimate strength behavior of steel box girders subjected to torsion, bending and shear [1, 2]. The box girder segment between two transverse frames or diaphragms has a single-cell rectangular cross section and is composed of longitudinally stiffened flanges and webs. The effect of torque on such a segment is to redistribute the shear force between the two webs so that they are subjected to different levels of shear. The method applies to the pre- and postultimate ranges of loading and its principal features are the following:

1. The compression flange is treated as a series of separate beam-columns whose behavior is described by a stress-strain relationship. The strain is the total shortening of the beam-column divided by the original length. The total shortening consists of the axial and curvature contributions. The plate is affected by its buckling and postbuckling deformations, and the stiffener is taken to be elastic-plastic. Two computer programs were used to obtain the stress-strain relationships of such beam-columns [3, 4].

2. The stiffeners in the tension flange and the webs behave according to the material stress-strain relationship. Thus, their yield stress is the limit of capacity.

3. The webs are analyzed by considering buckling and tension field strengths of the web sub-panels under shearing and normal stresses [5, 2]. The total strength of a web is computed as a sum of the contributions of the subpanels. The normal and bending stresses are assumed to remain constant after buckling whereas shearing stresses increase due to the tension-field action. Compatibility of deformations of the sub-panels in a web is enforced by keeping the shearing deformations to be the same. [1, 2]

4. The cross section can be subjected to any degree of warping or remain plane.

Figure 1 shows the distribution of normal stresses in the cross section.

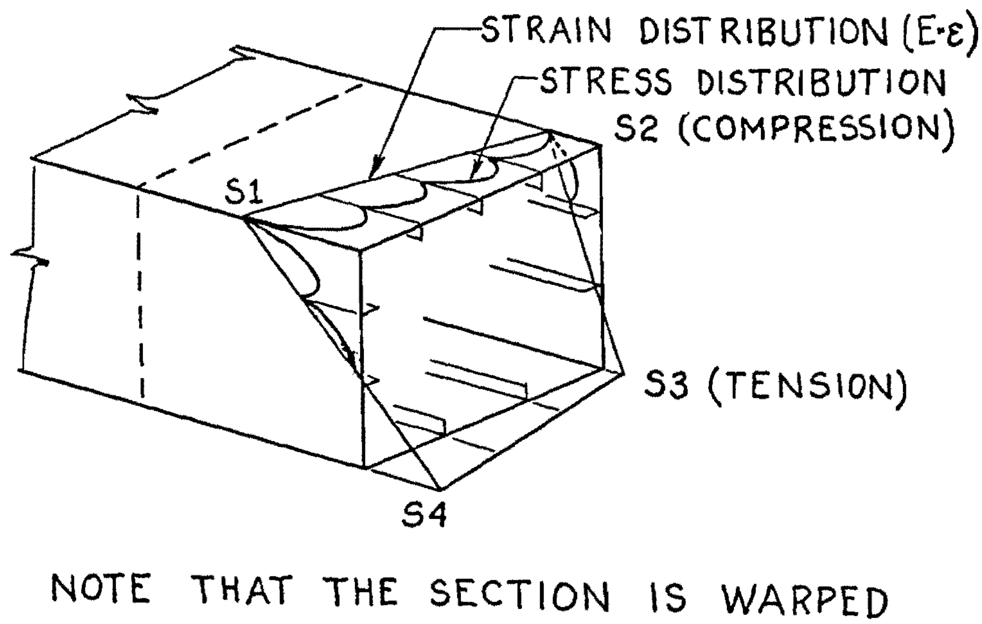


FIG. 1 - Distribution of Stresses and Strains in Cross Section of Box Girders

Strains are assumed to vary linearly across the width of each component and in the figure they are shown multiplied by the modulus of elasticity for a direct comparison with the stresses which are not necessarily proportional to the strains in the compression portion of the cross section. Distribution of the strains over the cross section is defined by the corner strains. For a specified value of the mid-width curvature of the cross section, these strains are adjusted to make the moment about the vertical axis and the axial force equal to zero, while the moment, shear and torque are kept in the same constant proportion to each other. This highly non-linear iterative process finally gives the load parameter for the curvature. By incrementing the curvature, a complete load vs. curvature relationship is computed for the pre-ultimate, ultimate and post-ultimate ranges of loading.

The method was compared with the available tests on box girder specimens [1, 6]. For test specimens under pure bending or bending and shear,the agreement was within 10%. However, for the four test specimens under moment, shear and torque, the method was too optimistic up to 70%.[1] It appears from the photographs in the publications [1, 6] that the test capacity in this case was lower than computed due to the inadeqate longitudinal stiffeners in the "heavier loaded" web.

Work is still in progress on further development of the method. Consideration is given, for example, to the inclusion of the effect of shearing stresses on the buckling, ultimate and post-ultimate behavior of the plate components in the compression flange and on the increase of the effective torsional eccentricity due to the shifting of the shear center as one web becomes "weaker" than the other.

References

1. Ostapenko, A., and Vaucher, A., ``Ultimate Strength of Ship Hull Girders under Moment, Shear and Torque,'' Fritz Engineering Laboratory Report No. 453.6 (MARAD Report MA-RD-940-80077), Lehigh University, July 1980.

2. Ostapenko, A., ``Strength of Ship Hull Girders under Moment, Shear and Torque,'' Extreme Loads Response Symposium, Society of Naval Architects and Marine Engineers, New York, October 1981, pp. 149-166, Proceedings of the SSC-SNAME Symposium held in Arlington, VA, on October 19-20, 1981

3. Rutledge, D.R., ``Computer Program for Ultimate Strength of Longitudinally Stiffened Panels,'' Fritz Engineering Laboratory Report No. 248.23, Lehigh University, 1968.

4. Smith, C.S., Kirkwood, W.C., McKeeman, J.L., ``Evaluation of Ultimate Longitudinal Strength of A Ship's Hull,'' AMTE(S) R 671, Admiralty Marine Technology Establishment, November 1977.

5. Ostapenko, A., ``Shear Strength of Longitudinally Stiffened Plate Girders,'' Proceedings, Structural Stability Research Council, Lehigh University, Bethlehem, Pa., 1980, pp. 36-40.

6. Dowling, P.J., Chatterjee, S., Frieze, P.A., Moolani, F.M., ``Experimental and Predicted Collapse Behavior of Rectangular Steel Box Girders,'' Steel Box Girder Bridges, Penelope Cartledge, BSc, ed., The Institution of Civil Engineers, London, February 1973, pp. 77-94.

[1] Later improvements of the computer program have resulted in much better correlation percentages: within 5% for pure bending and bending and shear, and within 38% for bending, shear and torque.

TASK GROUP 23 - EFFECT OF END RESTRAINT ON INITIALLY CROOKED COLUMNS

Chairman: W. F. Chen, Purdue University

Strength of H-Columns with Small End Restraints

E. M. Lui and W. F. Chen, Purdue University

Introduction

The study of the strength and behavior of columns has long been the subject of research for many years. There are many factors that influence the load-carrying capacity of columns. However, the initial geometric imperfections, the magnitude and distribution of residual stresses and the unavoidable end restraints of a column from beam-to-column connects far outweigh all other considerations. The first two factors have been the subject of extensive and systematic research by a number of researchers and investigators. As a result, analysis procedures and design criteria for columns in in-plane [2] and in-space [3,4] behavior have been formulated and proposed for general use. Nevertheless, the influence of end restraints on the strength and behavior of columns has not been systematically studied in depth. The combined effect of these three factors (initial crookedness, residual stresses, end restraints) will undoubtedly alter the load-carrying capacity of columns [1].

Up to the present moment, AISC specifications for the design of steel frames are based on the assumptions that the joints are either completely flexible (pinned condition) or completely rigid (fixed condition). The extensive research on pinned-end columns with initial imperfections and residual stresses has led to the development of multiple column curves [5] which allows the strengths of these columns to be assessed quite accurately, resulting in a more rational and economical design. However, experimental investigations of actual joint behavior conducted at various times during the past five decades have shown that typical simple connections do possess a certain amount of rotational rigidity that may influence the behavior of columns. The importance of end-restraint was first realized over fifty years ago when research workers [6,7,9] measured the relationship between the end-moment and the relative rotation of the beam to column at the connections. Methods of incorporating end-restraint into analysis and design were also proposed by these early investigators. Generally speaking, a more economical design will result if the effect of end-restraint is taken into consideration. Therefore, to design a column with its ends restrained as if it were pinned will be conservative but by no means economical.

The investigation included in this report was limited to nonsway steel columns of hot-rolled wide-flange shapes and flame-cut H-shapes. Initial out-of-straightness was taken as 0.001 L at mid-height in the plane of bending (L = length of the column). End restraints were provided by four simple beam-to-column connections (single web angle, double web angle, header plate and top and sear angle) (see Fig. 1). No lateral loads were present. All external forces were acting at the ends of the columns. Both strong and weak axes bending were considered. Five different patterns of actually measured residual stress distributions and several different steel grades were used in the study.

Effective length factor

The effective length factor, K, used here is defined as that length (slenderness) which gives on the basic column curve for pinned ends the same strength as the failure load for the actual end restrained. The determination of this factor is shown schematically in Fig. 2.

The values of K at several different load levels P_{max}/P_y, were determined for all 83 column curves. The results show that for each curve, the values of K do not vary significantly over the various load levels. This observation was also indicated by the analytical work of Sugimoto and Chen [8], among others. The values of K were plotted against the coefficients of end restrain, α, in Fig. 3. The end restraint factor α is defined in Fig. 4 where M_{pc} denotes the plastic moment capacity of the column section including the effect of axial force on the column. It can be seen that there is an inverse relation between K and α for a certain range of α. For simplicity, a linear relation is assumed (see Fig. 3).

$$K = 1.0 - 0.017\alpha \geq 0.60$$

The significance of this expression is that it enables us to get a rough estimate of the strength of columns with restrained ends once the strength of the columns with pinned ends are known.

Conclusions

1. Maximum load-carrying capacity of H-columns is increased if small end restraint due to shear connections is incorporated into the analysis of an initially crooked column with residual stresses.

2. Increased connection stiffness increases the maximum load-carrying capacity of columns except at very low λ-values when yielding is the primary cause of failure.

3. The influence of end restraints on column strength is more noticeably at high λ-values when stability is the critical criterion of failure.

4. The influence of residual stresses on the strength of columns at high λ-values will become negligible when compared with initial crookedness and end restraint. This justifies the use of elastic analysis to assess the strength of very slender columns.

5. Comparison of restrained-ended column curves with pinned-ended column curves reveals that the effective length factor K for a given column does not vary significantly over different maximum load levels. Therefore, a linear relation between the effective length factor K and the coefficient of end restraint α can be established for a given column with known value of rotational stiffness.

References

1. Chen, W. F., "End Restraint and Column Stability," Journal of the Structural Division, ASCE, Vol. 106, No. ST11, November 1980, pp. 2279-2295.

2. Chen, W. F., and Atsuta, T., "Theory of Beam-Columns," In-Plane Behavior and Design, Vol. 1, McGraw-Hill, New York, N.Y., 1976.

3. Chen, W. F., and Atsuta, T., "Theory of Beam-Columns," Space Behavior and Design, Vol. 2, McGraw-Hill, New York, N.Y., 1977.

4. Chen, W. F., and Cheong-Siat-Moy, F., "Limit States Design of Steel Beam-Columns - A State-of-the-Art Review," Solid Mechanics Archives, Vol. 5, Issue 1, Noordhoff, Leyden, The Netherlands, February, 1980, pp. 29-73.

5. Johnston, B. G. (ed.), "Guide to Stability Design Criteria for Metal Structures," 3rd edition, John Wiley and Sons, New York, N.Y., 1976.

6. Rathbun, J. C., "Elastic Properties of Riveted Connections," Transactions of ASCE, 1936, Vol. 101, pp. 524-563.

7. Steel Structures Research Committee. First, second and final repots, Department of Scientific and Industrial Research, HMSO, London, 1931, 1934, 1936.

8. Sugimoto, H., and Chen, W. F., "Small End Restraint Effects on Strength of H-Columns," Journal of the Structural Division, ASCE, Vol. 108, No. ST3, March, 1982, pp. 661-681.

9. Young, C. R., and Jackson, K. B., "The Relative Rigidity of Welded and Riveted Connections," Canadian Journal of Research 1934, 11, No. 1, 62-100, and 1934, 11, No. 2, pp. 524-563.

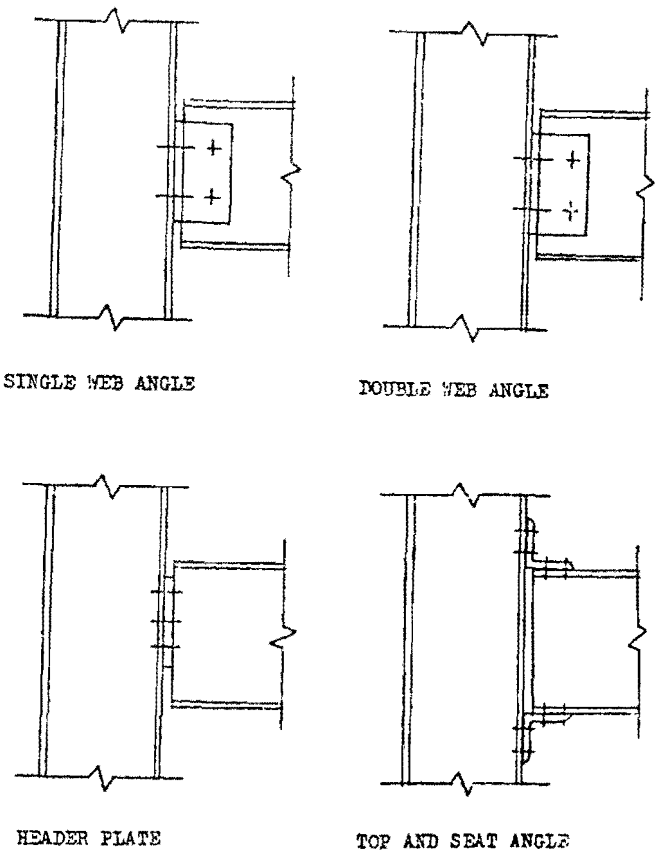


Figure 1. Simple Beam-to-Column Connections used in the Study

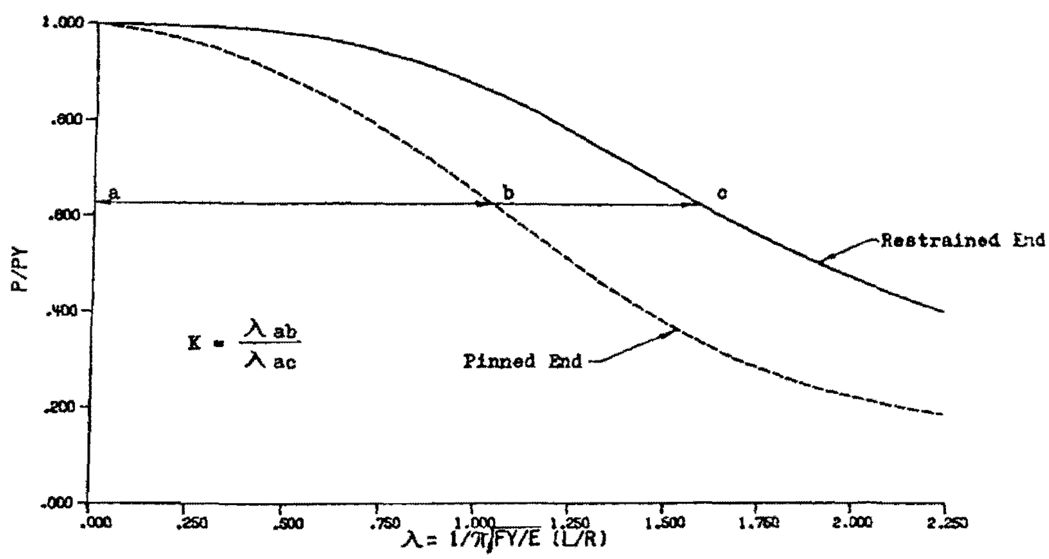


Figure 2. Determination of Effective Length Factor, K

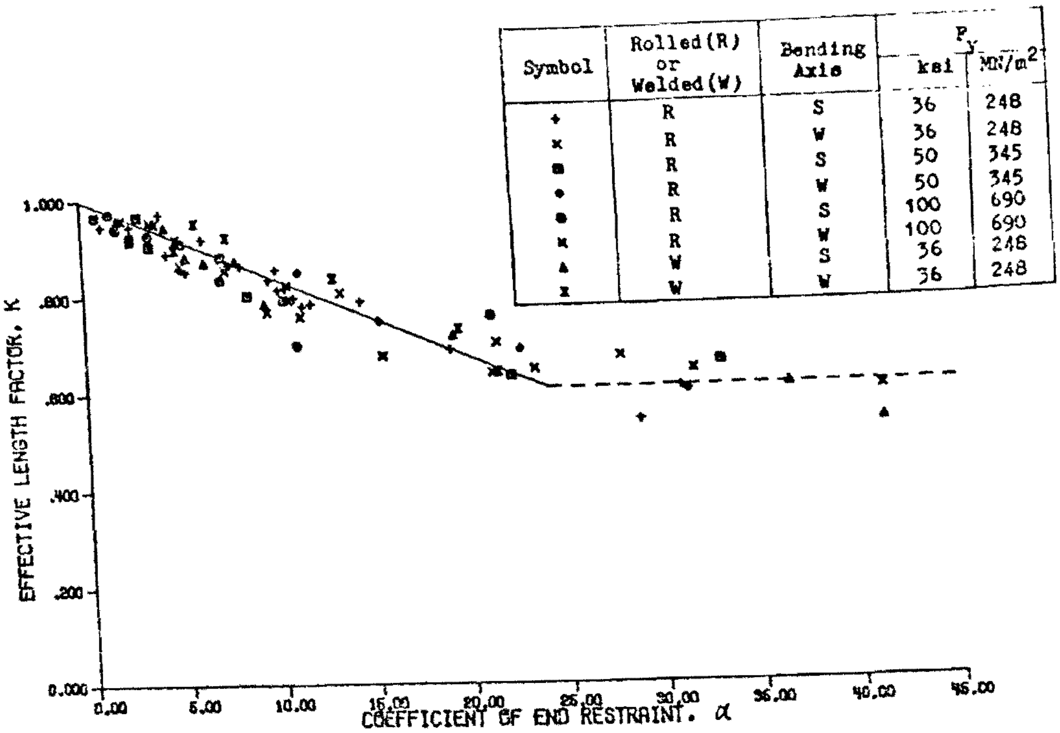


Figure 3. Relationship between K and α for All 83 Column Curves

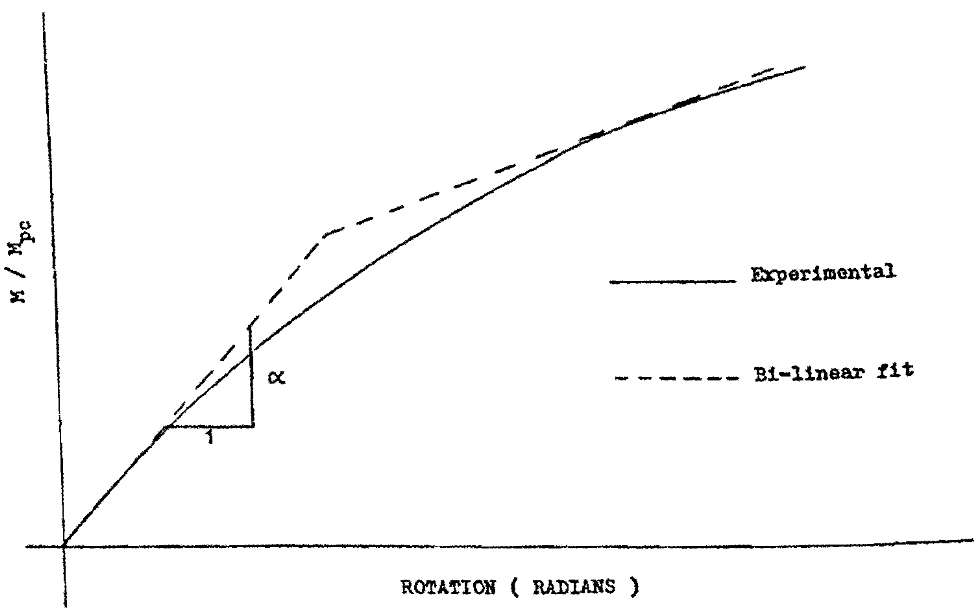


Figure 4. Determination of Coefficient of End Restraint, α

Limit States of Flexibly-Connected Steel Building Frames

M. H. Ackroyd, Rensselaer Polytechnic Institute

An interactive computer graphics program has been developed to predict the ultimate capacity of planar flexibly-connected steel building frames under proportional loads. Specification of frame characteristics and loading is done interactively in generic terms and then converted automatically by the program into the appropriate mathematical model for an ultimate load analysis. The analysis increments loads proportionally and accounts for nonlinear moment-rotation curves for beam-to-column connections, formation of zones of partial plastification in overloaded members (accounting for the presence of residual stresses), and stability effects in columns. Results of the analysis are presented in graphics displays of sequences of deformed shapes showing the development of regions of diminished structural integrity and the progression of the frame to its limit state.

A parametric study of a set of 11 frame topologies representative of typical building configurations focused on the influence of flexible beam-to-column connections on the ultimate capacities of the frames. Member properties were assigned to be consistent with current building designs. Beam-to-cloumn connection stiffnesses were assigned to cover maximum feasible ranges for the framing schemes considered: thus most of the frames studied were analyzed with several different connection stiffnesses.

Each frame was subjected to realistic combinations of gravity and wind loads. Loads were increased proportionately up to the limit state of the frame. In all the frames studied, the mode of failure was inelastic frame sidesway (rather than single-story sidesway instabilities). In particular, two "non-classical" modes of failure were observed: multiple-story sway failure, and column stack plastification.

Multiple-Story Sway. Figure 1 shows the limit state for a three-bay, ten-story frame with beam-to-column connections having initial rotational stiffnesses of 3,290,000 inch-kip/radian throughout the frame. As load intensities were incremented, the connection stiffnesses decreased in the prescribed nonlinear manner, and eventually the bases of the first-story columns began to yield (as shown by the shaded triangles). With subsequent load increments, the connection stiffnesses continued to decrease and, in the lower stories where wind moments were the largest, the connections on the leeward ends of girders "softened" to about half of their original stiffnesses. These reductions were sufficiently large to permit rotational relaxation of the column ends so that column plastification did not occur across the first-story level (as would be expected in a classical single-story sway failure), but rather, the plastification occured across the fifth-story level because the relative stiffnesses of girder/connection/column did provide the rotational restraint of the column tops. This five-story sway failure was the limit state for this frame.

Column Stack Plastification. Figure 2 shows a limit state for a low, squat frame which also does not fall into a "classical" mode. In this particular frame, all plastification occurred in the leeward column stack due to the combined action of gravity bending moments from the relatively long girders and due to the moments induced by wind acting from the left. Also, relatively heavy curtain wall loads were specified. Under no lateral sway, these curtain wall weights would be carried directly down the column stack to the foundation. However, due to sway, the vertical loads act through an induced eccentricity causing additional P-Delta moments in the column stack. These moments accelerated the formation of plastic zones throughout the stack until the ultimate flexural capacity of the stack was exhausted. At that point, the role of this column stack changed from that of a resisting structural element to that of an element causing additional loading on the remainder of the frame to its left, thus precipitating an overall sidesway instability.

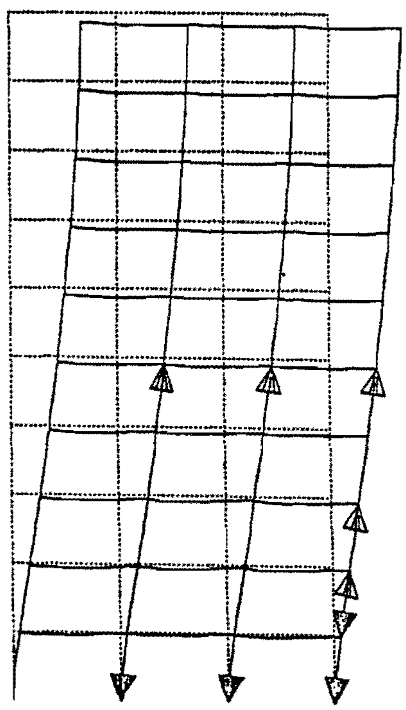


Figure 1. Five-story Sway Failure

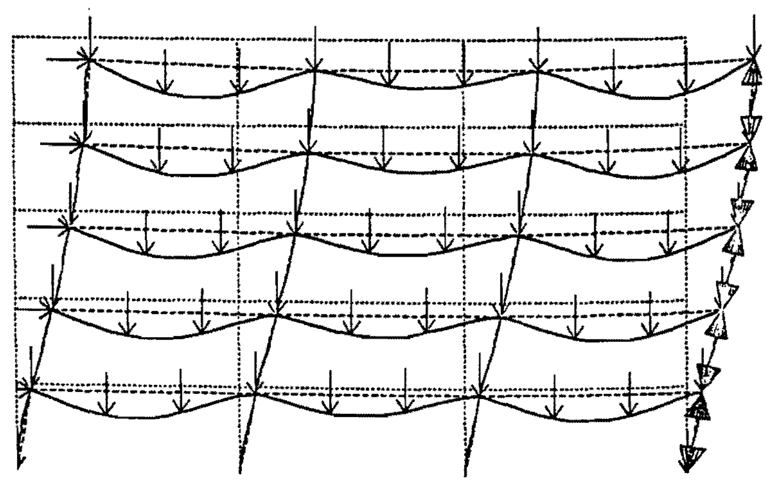


Figure 2. Column Stack Plastification

Acknowledgements. The development of the above-described interactive graphics package was part of a research project funded by the American Iron and Steel Institute. Their continued interest and support is gratefully acknowledged. Computer facilities were provided by the Center for Interactive Computer Graphics, RPI, Troy, N.Y.

References.

1. Ackroyd, M. H., "Nonlinear Inelastic Stability of Flexibly-Connected Steel Building Frames", Ph.D. thesis submitted to the University of Colorado, Boulder, Colorado, supervised by K. H. Gerstle, May, 1979.

2. Frye, M. J. and Morris, G. A., "Analysis of Flexibly-Connected Steel Frames", Canadian Journal of Civil Engineering, Vol. 2, 280, 1975.

3. Ackroyd, M. H., "Analysis of Steel Building Frames Using Interactive Graphics", report to American Iron and Steel Institute under Project No. 311, Ultimate Capacity of Flexibly-Connected Steel Building Frames, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N.Y., August, 1981.

Construction of Variable End Restraints for Small-Scale Testing of Tubular Columns

D. A. Ross, University of Akron

There is considerable need to be able to determine the effect of end restraints on the buckling and post-buckling behavior of tubular columns. For this reason, a small-scale experimental program has been undertaken in the Structural Engineering Laboratory of The University of Akron.

Since columns are typically parts of frames, the beams attached to the column ends serve as partial restraints on the rotation of column ends during buckling. It has been demonstrated that this end restraint may have significant influence on the buckling behavior of tubular columns, both analytically (1) and experimentally. However, there is little experimental data covering the post-buckling behavior of such columns. Furthermore, unlike wide flange or H-shaped columns, tubular columns do not have a readily obvious preferred buckling direction. Thus the buckling behavior of such columns is essentially three dimensional in nature - influenced by many factors, including the stiffnesses of perpendicular beams framing into each end of a column.

The beam stiffnesses may be modeled as tubular springs giving a column model similar to Fig. 1. To model such behavior two end fixtures were constructed, shown diagrammatically in Fig. 2. In each direction at each end a torsion bar of variable length is included so that a torsional restraint of variable stiffness may be considered. Futhermore the torsional springs are essentially uncoupled so that torsional restraints may be varied independently. It was intended that columns up to 28 inches long and having an external diameter up to 0.5 inches could be tested in this manner. This enabled a maximum slenderness ratio of 180 to be obtained. It was considered that loads up to 3000 pounds would be applied to column specimens.

The column axial load is transferred directly from the testing machine to the column specimen. Ideally a "frictionless" support should be provided, but this, of course, is impossible. After some experimental investigation, a rolling support was developed where the column end effectively rolled on a hemisphere attached to each column head. Friction of the ends was then shown to be minimal. Some effort was also required to minimize looseness of joints. This was achieved by driving tapered pins into connections wherever possible.

Calibration of the end fixtures was another important step in commissioning the apparatus. Ideally, the relationship between torsion bar rotation and torsional moment in the restraint is linear and independent of column axial load. However, unavoidable friction and joint looseness effectively prevent this relationship. It could not be assumed that the calibration achieved at no axial load would be correct when an actual experiment was being conducted. After a number of calibration tests had been conducted, a relationship was developed for each torsion bar of the form:

$$M_{restraint} = A\left[\frac{\theta}{\left(\frac{L_{TB}}{L_{Ref}}\right)}\right]^{B}$$

where

$M_{restraint}$ = torsional moment in restraint system (at one end in one direction)

θ = rotation of torsion bar (degrees)

L_{TB}, L_{Ref} = the length of the torsion bar and a reference torsion bar length, respectively.

A and B were shown to be constants for applied axial loads greater than a certain minimum (usually about 500 pounds). For perfect linearity B would be unity, but for the fixtures developed, B was in the range 0.7 to 0.8.

Fig. 3 shows a completed end fixture in position for a column buckling test. In each direction rotational variable differential transducers (RVDT's) measure torsion bar rotation. Successful column buckling tests have now been completed which should provide valuable data for comparison with analytical investigations.

References

1. Chen, W. F., "End Restraint and Column Stability", Journal of the Structural Division, ASCE, Vol. 106, No. ST11, Proc Paper 15796, November, 1980.

2. Ross, D. A., Chen, W. F., and Tall, L., "Fabricated Tubular Steel Columns", Journal of the Structural Division, ASCE, Vol. 106, No. ST1, Proc Paper No. 15122, pp. 265-282, January, 1980.

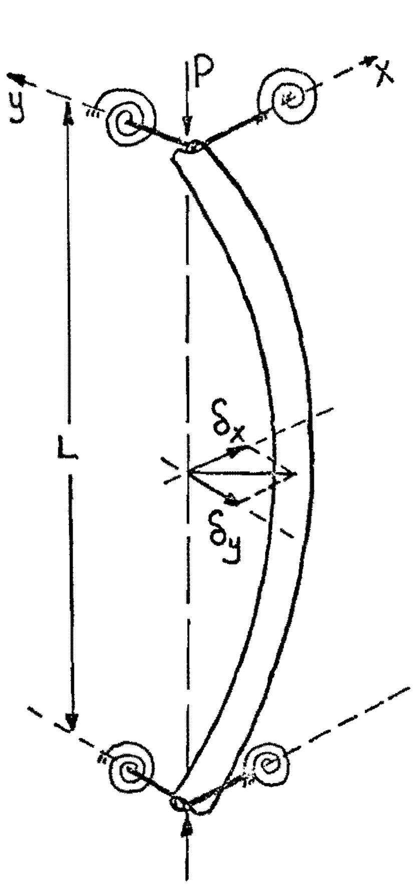


Fig. 1. Column Model

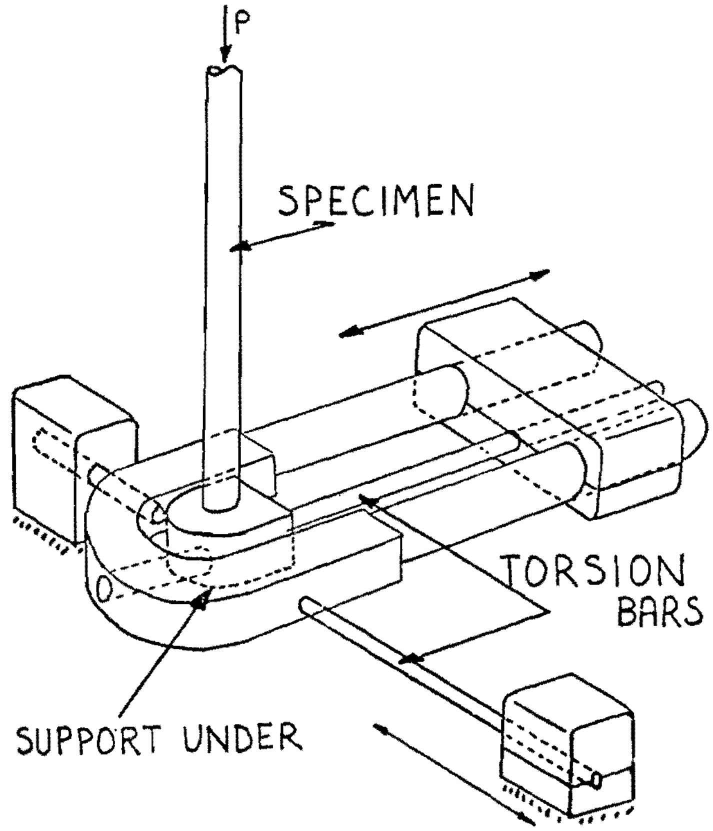


Fig. 2. Diagram of Column End Restraints

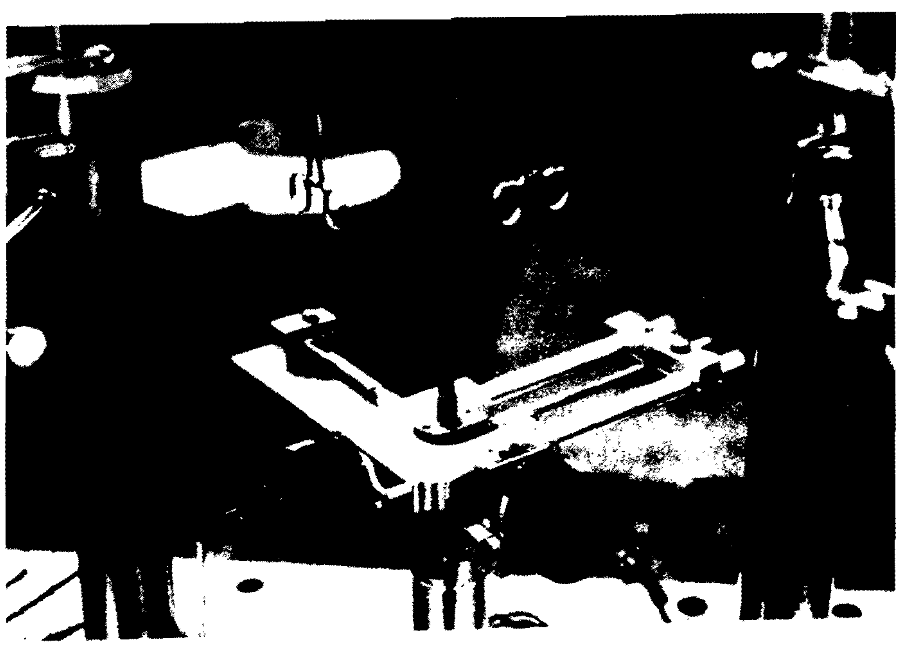


Fig. 3. Photograph of an End Fixture

TASK GROUP 7 - TAPERED MEMBERS

Chairman: G. C. Lee, State University of New York

Full Scale Testing of Tapered Structural Members

D. L. Johnson, Butler Manufacturing Company Research Center

Tapered members are used extensively in pre-engineered building systems. The tapered webs allow optimum usage of material in beams and rigid frames. Extensive research into the design aspects of tapered members has been sponsored by the American Institute of Steel Construction (AISC) and the Metal Building Manufacturers Association (MBMA). The results of this research are summarized in the recently published book[1] "Design of Single Story Rigid Frames" by Lee and Ketter and Hsu. Throughout this period the Butler Research Center has conducted a series of full scale tests studying a number of related design problems.

The Research Center has two facilities for testing building systems, the first is a 40' wide, 75' long and 4' high building. Two tapered beams span the 40' direction. After installation of the particular roof system being studied the entire structure is covered with a plastic sheet and a pressure differential is created by evacuating the air from inside the building using two 20 h.p. roots type superchargers. Instrumentation consists of linear transducers monitoring critical deflections and five video cameras to closely observe member behavior from the safety of the instrumentation building.

Four of the cameras are stationary while the fifth is a remote controlled robot capable of moving anywhere in the structure and with a zoom lens for examining developing problems with any of the members.

The other test method utilizes a reaction beam in the floor of the Research Center's main test building. The beam, a W36 x 300, can accommodate a 70' clear span frame or beam. The procedure normally followed is to construct a section of building over the reaction beam complete in all details, roofing, secondary structurals, compression flange bracing, etc. Spreader bars are mounted on the roof and rods pass through the roof to a series of hydraulic cylinders mounted on the reaction beam. A manifold system allows different loads to be applied to different parts of the structure simultaneously. There is also provision for applying a horizontal load to the frame independently or in combination with vertical loads. Instrumentation normally consists of strain gages on critical elements (including bolts), extensive use of displacement transducers and load cells to monitor reactions.

In the study of tapered frames and beams a number of specific problems have been addressed, a few of these include:

1. Confirmation of the results of Krishnamurthy's research in butt plate connections[2]. Full scale tests resulted in excellent correlation in bolt loads and splice plate thickness.

2. Web buckling due to a concentrated load acting on the web through a flange, AISC Specifications[3] Section 1.10.10.2. Testing of

a number of tapered webs (all unstiffened) with h/t values from 50 to over 200 indicated that this provision in the specification was unreasonably conservative. A research project at the University of New York at Buffalo is currently being sponsored by AISC and MBMA. Dr. George Lee will report on the progress of this project at this same conference. Hopefully the research will result in a more reasonable provision in the specifications.

3. Type 2 construction. A study of tapered beams using Type 2 construction was conducted to determine if a web tapered beam designed as a simple beam supported by columns designed to resist wind load moments only would satisfy all criteria required by the AISC specifications for structures using Type 2 construction, AISC Section 1.2.

Full scale tests of 60' beams verified the design assumptions. The columns and splices sustained no permanent deformation under full vertical load and attained full safety factor under combined vertical and lateral load.

4. Flange and web buckling. Tapered member design frequently results in combinations of high ratios of depth to web thickness and flange width to flange thickness. It was found that under these conditions the AISC provisions for flange buckling were not conservative. The formula in AISC Section C-2 is based on the assumption that the web stiffness provides considerable restraint to the flange and a K value of 0.7 is used compared to the simply supported case where K equals 0.425. The tests showed that where h/t was very high the effective K could actually be less than 0.4 and precipitate premature buckling. Subsequently Butler frames and beams have been designed using a flange buckling stress formula using a variable K factor equal to

$$\frac{4.44}{\sqrt{h/t}}$$

This results in a modification to AISC Equation C2-3 from

$$Q_s = 1.415 - 0.00437\ (b/t)\sqrt{F_y}$$

to

$$Q_s = 1.293 - .00309\ (h/t)\frac{\sqrt{F_y}}{\sqrt{K_s}}$$

where

$$K_s = \frac{4.44}{\sqrt{h/t}}$$

More recent tests have demonstrated the viability of this approach.

References

1. Lee, Ketter, Hsu "Design of Single Story Rigid Frames" MBMA 1981.

2. N. Krishnamurthy, "A Fresh Look at Bolted End-Plate Behavior and Design" 2nd Quarter 1978 AISC Engineering Journal.

3. American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", November 1978.

Web Crippling of Unstiffened Thin-Webs Under Concentrated Load

G. C. Lee and J. H. Chern, State University of New York at Buffalo

By considering both geometrical and material nonlinearities, finite element solutions are obtained for the web crippling load of unstiffened thin-web girders under concentrated load. Comparisons of results are made with available experimental information and the newly-proposed AISC LRFD criteria for web crippling are examined.

This research is supported jointly by AISC and MBMA.

OTHER RESEARCH REPORT

Report on the ECCS Stability Program

C. Massonnet and R. Maquoi, University of Liege

Foreword

Professor Massonnet has been invited to present at this session a report as chairman of TC 8 of the European Convention for Constructional Steelwork, on the current progress in Europe. Till last November, he intended to attend the New Orleans session, but some personal and professional duties did not allow him to have the opportunity to meet here a lot of friends and/or colleagues. He has nevertheless asked me to present the following report on his behalf and charged me to convey his best regards to you.

Introduction

Since 1980, Technical Committee VIII - Stability (T.C.8) of ECCS has been reorganized. The main reason was that Technical Committee V - Plasticity - has been dissolved because the pending problems were so intimately connected with stability problems that it was better to integrate them in the field of activity of T.C.8. Presently T.C.8, chaired by Prof. Massonnet, is composed of four Task Working Groups, that are concerned with following aspcets:

TWG 8/1 : Components (of Bars)
TWG 8/2 : Systems (of Bars)
TWG 8/3 : Plated structures
TWG 8/4 : Shells.

Its aim is to update and/or improve the set of ECCS recommendations which were produced in 1975 and 1978 successively and explained by a second volume, commonly named "Manual of Stability".

In addition to its role of advancing the technical level of Steel Structures by coordinating all research efforts in Western Europe, ECCS Recommendations have paved the way for an European Code, that would give equal chances to all builders belonging to the common market. Thus, the Commission of the European Communities created as early as 1975 a Working Group "Stability of Structures" with the aim of preparing uniform technical texts concerning the general safety principles and the justification of the stability of the various structures. In 1979, a panel chaired by Prof. Dowling (Imperial College, U.K.) had to draft an Eurocode NO.3, dealing with steel structures. Presently the Eurocode 3 is nearly completed and will soon await the approval of the political authorities.

In its present form, one can say that Eurocode 3 is basically founded on the ECCS Recommendations for its spirit but, as it intends to serve as a standard, it has introduced some simplified design rules and/or amendments. It is hoped that, despite some national divergences, the result will be valuable.

Concerning, now, the prospects of ECCS T.C.VIII itself, let me briefly summarize the state of development as follows.

Technical Working Group 8/4: Stability of Shells.

This group, chaired by Prof. Vandepitte (Gent University, Belgium), is probably the most active but if the field is no more virgin, a lot of well defined problems remain unsolved or badly solved. TWG 8/4 has already prepared design rules regarding the buckling analysis of unstiffened circular cylinders subject to meridional compression or to uniform external pressure, ring stiffened cylinders subject to external pressure, cylinders under combined axial load and external pressure and unstiffened spherical shells under uniform radial pressure. More recently, a new draft on axially loaded stringer stiffened cylinders has been prepared. Nowadays an extensive test programme on hydrostatically loaded conical shell models is in progress in Gent University and draft design rules allow for appraising the ultimate load of these structures with a very good accuracy.

A symposium on the purpose "Buckling of Shells" will take place in Stuttgart in next May at the occasion of Prof. Bornscheuer's retirement. Prof. Ramm is in charge of the corresponding organization.

Technical Working Group 8/3: Plated Structures.

This working group was formed in 1973 under the chairmanship of Prof. Massonnet (Liège University, Belgium). Since the latter was elected as chairman of T.C.8, Prof. Dubas (ETH, Zürich) succeeded him at the chair. Many papers have been produced and the members agreed three years ago on the preparation of a book on the behaviour and design of steel plated structures. This field of activity is one for which it is rather difficult to obtain a unanimous agreement on design rules and recommendations; this is probably due to the fact that, during the last decade, some of the main Western countries - namely United Kingdom, West Germany, Switzerland - have completely modernized their standards, as a consequence of well known resounding accidents.

Previous old codes were mainly based on a common background: the linear plate buckling theory, which had the merit of simplicity but was not representative of the actual behaviour and thus not able to give an homogeneous safety against collapse. The modern viewpoint is to use a more realistic approach: the ultimate carrying capacity. In this respect, both approaches are unfortunately very different and design rules resulting from extensive parametric study appear most often in the form of cooking recipes, whose physical background has nearly vanished.

The aforementioned prospective book would deal with the behaviour of stiffened and unstiffened plates, the design of stiffened - transversely of transversely and longitudinally - plated structures, and, last, the interaction between overall column buckling and local plate buckling in thin-walled columns. The design of transversely stiffened webs is based on the Cardiff model for the diagonal tension field, with some recent improvement for the design of intermediate and bearing stiffeners and the effect of concentrated loads. At present, ECCS believes that the generalization of the diagonal tension field model to the deep longitudinally and transversely stiffened webs is not yet sufficiently established and proved, so that the design of these plated structures could be presented as follows. First, if the longitudinal stiffeners are designed so as to remain rigid up to collapse ($\gamma > m\gamma^*$), the longitudinal stiffeners are designed for rigidity only and the plate subpanels checked for buckling and plasticity, taking account of postcritical strength behaviour through non linear interaction equations at collapse. Secondly, if the assumption of rigid longitudinal stiffeners is not satisfied, one continues to refer to linear plate buckling behaviour; this second approach is obviously little satisfactory, but it could serve to accustom the designer progressively to the modern concept of ultimate strength method.

Interactive buckling between overall column buckling and local plate buckling in thin-walled hollow square or rectangular sections will be based on a large and extensive experimental and theoretical research work undertaken in Liège since about five years. Because of the small time I have for this presentation, I shall just say that this design method refers to a plate buckling curve that gives the ultimate load of the walls and to a modified buckling curve, in which the ordinates and abscissae are changed by substituting to the squash load (or yield stress) the ultimate wall load (or mean ultimate compressive stress).

Task Working Groups 8/1 and 8/2: Components and Systems.

Because of the intimate connections between the problems with which these two TWG are concerned, I shall make no distinction between both kinds of activities.

First of all, the multiple european column buckling curves have recently received analytical expressions, which have already been adopted not only by ECCS but also by the drafting panel of the Eurocode No. 3 and by several national codes. The opinion is that the mathematical formulation of the buckling curves is a valuable improvement and, as far as we know, the new American curves could be formulated on the base of a similar concept. More recently, it was suggested to let depend the imperfection parameter of the analytical expressions on the yield stress in order to take account of the fact that the relative importance of the residual stresses decreases when the yield stress increases.

Amongst problems under discussion presently are:

- the beneficial effect of the rotorizing on the buckling strength, due to the strain hardening resulting from this alternate plastic bending treatment. A similar effect is obtained by cold forming and is thus directly dealing with thin-walled cold formed tubes.

- the evaluation of the buckling strength of H profiles obtained by automatic welding of plates. Because of the heat input, the residual stresses may be exceptionnally high and the question is to evaluate simply and safely the amount of these residual stresses and the loss of buckling strength which they induce.

A really pin-ended bar is rather unusual as most of the compressed bars are parts of structures and therefore restrained. The column buckling curves are rendered applicable to restrained bars through the concept of effective length. One of the main aims presently is the investigation of the limits of validity of this last concept for designing restrained bars, and more pecularly the study of what happens after the weakest bar of the structure has collapsed. Let us just mention that, presently, the length factor is usually determined in function of the rotational elastic restraints at the ends:

a) by Donnell's approximate formula for fixed ends (no sway);

b) by charts suggested by Johnston or by Wood, or by analytical expressions representing these charts, either for fixed ends or for ends permitted to sway freely.

The problem of bars in uniaxial bending and compression (usually called "beam-columns") is solved by a non linear interaction formula in which the effect of the geometrical imperfection on buckling is represented by an additional flexural term. It is commonly referred to the Dutch formula, instead of to the Belgo-American formula, for which the term of normal force contains the effect of buckling in its whole. Only the presentation of the formula will be somewhat arranged in order to include the lateral torsional buckling, that will be calculated according to the Lindner approach.

For what about biaxial bending and compression, the Dutch formula extended to biaxial bending will be followed, as closely as possible, despite the fact that valuable formulae due to Chen et al are available; the reason is that some doubts were raised about the safety of the Chen approach in some cases. Accordingly, further investigations have been recommended.

Angles for transmission towers is a subject which will merit much attention for future work. This is the perfect example of the interaction that does exist between activities of TWG 8/1 and 8/2; indeed, the behaviour of such angles depends mainly on the manner they are connected together in order to constitute a transmission tower. In fact, a transmission tower is one of the few repetitive objects that exists in Civil Engineering and it is astonishing to see how backwards the design methods are presently. Certainly, decent design rules do exist; but they are generally based on a buckling check of the angles with a slenderness ratio given by empirical formulae, which result from numerous full scale tests. One cannot miss to be struck by the large difference in sophistication between design methods of transmission towers and the methods used to attack other repetitive structures, such as the airplanes.

Inside TWG 8/1, some members work in the field of transmission towers and are very active so that some progress can be expected in a near future. I shall mention too that in Liège, a research work is under way for developing a bar finite element representing the behaviour of an angle and taking account of following characteristics:

- eccentricities at the nodes;
- postbuckling behaviour and warping phenomena in the elasto-plastic range;
- the large displacements and space deformations.

It is now well accepted that, in hyperstatic structures, the elastic buckling load has only an academic interest and it is obvious that the search for the collapse strength of hyperstatic structures postulates the knowledge of the plastic behaviour of these structures. One of the major problems facing TWG 8/2 is to assess beyond the collapse of the weakest bar, until general collapse occurs. This domain is very difficult and has been only partially explored. It is just possible to tackle first truss problems and then frame problems.

As the column buckling curves are collapse curves and are more favourable than most of the older regulations, one must be cautious when defining the effective length. More importantly, the effective length of compressed chords of trusses for buckling in the plane of the truss, should be raised from 0,8 l to 0, 9 l. In this field, let me mention an investigation performed in Liege on "statically determinate" trusses with finite elements using large displacements and plasticity. The main aim of this numerical simulation was to emphasize the difference in behaviour between a frame and a truss. In a sway frame, the stability of any bar depends on the degree of lateral and angular restraint of its end sections and requires thus the solution of the whole system; on the contrary, in a truss, the nodes are practically fixed because the truss is formed of rigid cells and the degree of restraint of each bar depends essentially on the adjacent bars. As, in addition, the loads generally act at the nodes and do not subject directly the bars to bending, the axial forces play therefore the governing role whilst the bending moment has a secondary effect. In other words, the type of collapse observed in trusses is such that a design "bar by bar" and the concept of effective width are acceptable.

One of the main topics of TWG 8/2 is the ultimate limit state calculation of unbraced frames. For the sake of safety and economy, it was decided to use design methods of decreasing complexity.

a) The exact ultimate strength analysis:

   It refers to actual elasto-plastic behaviour of the material, to the influence of deformations and to that of geometrical and structural imperfections. Such a method is restricted to computer programmes.

b) <u>The second-order plastic hinge theory:</u>
The plastic zones are here localized in plastic hinges. This method is much more simple than the first one and leads almost to similar results. In simple cases, hand calculations are possible.

c) <u>The second-order elastic theory:</u>
This method is already widely used in practice. Hand calculations are possible and computer programmes are widely available for small desk computers.

A table enabling the choice of the most simple method for a specified structure will probably be a double entry chart, the parameters of which being $\Delta\psi$ and ε. $\Delta\psi$ is the criterion for the susceptibility of the structure to lateral displacement; it corresponds to the slope of the columns and is compared to ratio Q/10 V, where Q is the sum of horizontal loads within one story, whilst V is the resultant of the vertical loads over this story, corresponding to Q and calculated by the first order theory. ε is the characteristic number $l\sqrt{\frac{N}{EI}}$, that is thus equal to π when $N = N_{cr}$; the boundary between the two domains is $\varepsilon = 1$.

Above mentioned values for the boundaries of the various approaches are still provisional and further investigations are in progress in order to improve these limits.

Last, I shall mention the peculiar problem of composite columns. Design methods for composite columns are already existing in the model code prepared jointly by IABSE - FIP - CEB - ECCS ; they are based on an ultimate load design philosophy. They cover encased columns and concrete filled tubular steel columns, which are loaded with any combination of end loads and moments. The basic assumptions are that, on the one hand, there is a full interaction between steel and concrete up to collapse and that, on the other hand, allowance must be made for imperfections which are consistent with those adopted for assessing the strength of the reference axially loaded steel column. On the basis of this last assumption, a new concept of column slenderness is introduced, which leads to the same expression as that used in the European buckling curves and enables these curves to be used as basic design curves for composite columns.

Within the short time devoted to the presentation of this report, I have tried to give you a survey of the ECCS work in progress and of the problems in which TC8 and its four TWG will focus. Let me finally add that only the design philosophy based on the limit states is adopted for presenting the design rules and corresponding recommendations. Most of the recent standards in Western Europe are presented in the same way, so that the old fashioned philosophy of allowable stresses disappears slowly but surely.

1982 ANNUAL BUSINESS MEETING

The Structural Stability Research Council holds an Annual Business Meeting the purposes of reporting activities, electing officers and members, and presenting the following fiscal year's proposed budget for approval. The 1982 Annual Business Meeting was held on March 31 in conjunction with the Annual Technical Session, at the Rault Center Hotel in New Orleans.

The minutes of the 1982 Annual Business Meeting follow.

CALL TO ORDER

The meeting was called to order at 11:50 a.m. by the SSRC Chairman, Jerome S. B. Iffland. Approximately 65 persons were present.

The Chairman introduced John Springfield, Vice Chairman; Lynn Beedle, Director; Gulay Askar, Technical Secretary; Lesleigh Federinic, Administrative Secretary; and John Clark, a Life Member and past Chairman.

The Chairman expressed the Council's appreciation to the following sponsors for their support of the Annual Technical Session:

Brown & Root, Inc.
Chevron U.S.A., Inc.
Chicago Bridge & Iron Company
Earl and Wright
McDermott Incorporated
Mobil Research & Development Corporation
Shell Oil Company

Special appreciation was expressed to Chevron for hosting the reception on the 30th, and to Chevron and McDermott for the Executive Committee tours of the fabrication yard and offshore platform on March 27th.

The Chairman recognized the special session recorders:

David Ross and Steve Gunzelman - Tuesday afternoon session.
Pete Birkemoe and Tom Bubenik - Tuesday evening panel discussion.

ELECTION OF OFFICERS

The Nominating Committee, chaired by Duane Ellifritt, nominated John Springfield for a three-year term as Chairman and Samuel Errera for a three-year term as Vice Chairman of the Council.

Voting had been conducted by letter ballot of the membership. It was announced that the nominees were elected, and they take office 1 October 1982.

ELECTION OF EXECUTIVE COMMITTEE MEMBERS

The Nominating Committee nominated Don Sherman, Clarence Miller, and incumbent Ted Galambos for three-year terms on the Executive Committee.

Voting had been conducted by letter ballot of the membership. It was announced that the nominees were elected, effective immediately.

ELECTION OF MEMBERS-AT-LARGE

The following persons were nominated by the Executive Committee for election to Member-at-Large:

Gulay Askar - Lehigh University/Technical University of Istanbul
William F. Baker - Skidmore, Owings & Merrill
Cuneyt Capanoglu - Earl and Wright
Luis F. Estenssoro - Wiss, Janney, Elstner & Associates
Albert J. Gouwens - Goodell-Grivas, Inc.
Ravi K. Kinra - Shell Oil Co.
Andrew Lally - Steinman, Boynton, Groquist & Birdsall
David A. Ross - University of Akron
C. H. "Jay" Yoo - Auburn University

The motion that the nominees by elected as Member-at-Large was carried.

ELECTION OF LIFE MEMBER

The nominating committee, chaired by Bruce Johnston, nominated Duiliu Sfintesco for life membership.

The motion that the nominee be elected was carried.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director on behalf of the Finance Committee chairman, Gerard Fox. The proposed budget for fiscal 82-83 was also presented and approved.

Budget 82-83 Summary:

Expected balance, 1 Oct 82	$34,960
Estimated Income	72,600
Estimated Expenditures	63,100
Expected balance, 30 Sep 83	$44,460

It was noted that through the efforts of the Finance Committee, additional Council revenues have been generated. This enables the Council to again increase the amount of "seed money" available for research grants. A list of research grant recipients and their subjects was requested and will be prepared.

DIRECTOR'S REPORT

Lynn Beedle, Council Director, summarized the Executive Committee activities and concerns, and highlighted the task group activities over the past year. Most of the task groups continue to be actively involved with the preparation of the 4th Edition of the "Guide", and a significant number of them have identified needed research in their reports to the Executive Committee. TG-11, International Cooperation on Stability Studies, is engaged in the preparation of the program for the 3rd International Colloquium. TG-23, Effect of End Restraint on Initially Crooked Columns, reported that they are on-target with their task and expect to finish in about two years. Six task groups have new chairmen.

The Director commended John Springfield, Gulay Askar, and Lesleigh Federinic for their continued dedication to the work of the Council. He especially thanked Jerry Iffland for his tireless efforts over the past 3 1/2 years as Chairman of the Council. Jerry has given freely of his time, talents and resources on behalf of SSRC, with most effective results.

NEXT ANNUAL TECHNICAL SESSION AND MEETING

The Chairman announced that the next Annual Technical Session and Meeting will be held in conjunction with the North American Session of the 3rd International Colloquium in Toronto, on 8-11 May 1983 at the Westin Hotel. The theme will be "Stability Design Considerations: New Approaches and Comparisons". The Introductory Report for the Colloquium will be the "World View" report now being published by AISC in four separate issues of the ENGINEERING JOURNAL.

ADJOURNMENT

The meeting was adjourned at 12:20 p.m.

1982 ANNUAL TECHNICAL SESSION & MEETING ATTENDANCE

Participant	Affiliation
Abrahams, M. J.	Parsons, Brinckerhoff, Quade & Douglas, New York, NY
Ackroyd, M. H.	Rensselaer Polytechnic Institute, Troy, NY
Angelides, D. C.	McDermott, Inc., New Orleans, LA
Anzai, T.	Petro-Marine Engineering, Inc., Gretna, LA
Arguello, J. G.	Texas A & M University, (Student), College Station, TX
Argus, B.	Argus Technical Systems, Inc., New Orleans, LA
Askar, G.	Lehigh University, Bethlehem, PA
Atsuta, T.	Kawasaki Heavy Industries, Ltd., Khikob, Japan
Austin, W. J.	Rice University, Houston, TX
Bankston, C. L.	McDermott, Inc., New Orleans, LA
Beedle, L. S.	Lehigh University, Bethlehem, PA
Beil, R. E.	Sverdrup & Parcel & Associates, Inc., St. Louis, MO
Bellis, G. B.	Chevron, U.S.A., New Orleans, LA
Bentley, J. S.	Tulane Engineering, (Student), New Orleans, LA
Bigham, J.	Chevron, U.S.A., New Orleans, LA
Birkemoe, P. C.	University of Toronto, Toronto, Ontario, Canada
Biswas, M.	Texas A & M University, College Station, TX
Bjorhovde, R.	University of Arizona, Tucson, AZ
Blessey, W. E.	Tulane University, New Orleans, LA
Boston, L. A.	McDermott, Inc., New Orleans, LA
Bruce, R. N.	Tulane University, New Orleans, LA
Bubenik, T. A.	Exxon Production Research, Houston, TX
Burke, T. M.	Gulf Oil Corp., New Orleans, LA
Burnett, R. M.	Cities Service Company, Tulsa, OK
Caldwell, D.	Tulane University (Student), New Orleans, LA
Capanoglu, C. C.	Earl and Wright, San Francisco, CA
Carubba, F. M.	Shell Offshore Inc., New Orleans, LA
Chandra, T. K.	Texaco, Inc., New Orleans, LA
Chang, B.	McDermott, Inc., New Orleans, LA
Charles, B. S.	McDermott, Inc., New Orleans, LA
Chen, M.	University of New Orleans, New Orleans, LA
Chen, W. F.	Purdue University, West Lafayette, IN
Chen, Y. N.	American Bureau of Shipping, New York, NY
Cheng, F. Y.	University of Missouri, Rolla, MO
Clark, J. W.	Consultant, Pittsburgh, PA
Craig, M. J. K.	Union Oil Co. of California, Brea, CA
Das, S. C.	Tulane University, New Orleans, LA
Dawsey, J. V.	McDermott, Inc., New Orleans, LA
Dortch, B.	Chevron, U.S.A., New Orleans, LA
Dowling, P. J.	Imperial College of Science & Technology, London, England
Durkee, J. L.	Consulting Structural Engineer, Bethlehem, PA
Dwyer, M. G.	McDermott, Inc., New Orleans, LA

Edlund, B.	Chalmers University of Technology, Goeteborg, Sweden
Elgaaly, M.	Bechtel Associates Prof. Corp., Ann Arbor, MI
Ellifritt, D. S.	Metal Building Manufacturers Assoc., Cleveland, OH
El-Metwally, S. E.	George Washington University (Student), Washington, DC
Errera, S. J.	Bethlehem Steel Corp., Bethlehem, PA
Faulkner, D.	Conoco Inc., Houston, TX
Fern, D. T.	McDermott, Inc., New Orleans, LA
Ferver, G. W.	Ferver Engineering Co., San Diego, CA
Figgers, R. F.	McDermott, Inc., New Orleans, LA
Fontova, R.	Shell Offshore Inc., New Orleans, LA
Foss, G.	Det Norske Veritas, Oslo, Norway
Frieze, P. A.	University of Glasgow, Scotland
Fu, S. L.	Gulf Oil Corp., Houston, TX
Gaines, E. T.	McDermott, Inc., New Orleans, LA
Galambos, T. V.	University of Minnesota, Minneapolis, MN
Gaus, M. P.	National Science Foundation, Washington, D.C.
Gehman, S. L.	Texaco, Inc., New Orleans, LA
Goldberg, J. E.	National Science Foundation, Washington, D.C.
Gonzalez, C. M.	Chevron, U.S.A., New Orleans, LA
Graham, R. R.	U. S. Steel Corp., Pittsburgh, PA
Grimm, D. F.	McDermott, Inc., Houston, TX
Gunzelman, S. X.	Brown & Root, Inc., Houston, TX
Hall, D. H.	Bethlehem Steel Corporation, Bethlehem, PA
Haney, J. P.	Shell Offshore Inc., New Orleans, LA
Hartmann, A. J.	Westinghouse Electric Co., Monroeville, PA
Hayes, D. J.	Sohio Construction Co., San Francisco, CA
Henry, C. N.	Chevron, U.S.A., New Orleans, LA
Hill, C. M.	Tulane University (Student), New Orleans, LA
Hotchkies, J. W.	Algoma Steel Corp., Ltd., Toronto, Ontario, Canada
Howard, W. C.	Texaco, Inc., New Orleans, LA
Huete, D. A.	Shell Offshore Inc., New Orleans, LA
Iffland, J. S. B.	Iffland Kavanagh Waterbury, New York, NY
Illanne, C. M.	Chevron, U.S.A., New Orleans, LA
Jan, H. Y.	American Bureau of Shipping, New York, NY
Jeanes, D. C.	American Iron and Steel Institute, Washington, D.C.
Johnson, A. L.	American Iron and Steel Institute, Washington, D.C.
Johnson, D. L.	Butler Manjfacturing Co., Grandview, MO
Johnson, M.	McDermott, Inc., New Orleans, LA
Johnston, B. G.	Consultant, Tucson, AZ
Jolissaint, R. E.	University of New Orleans (Student), New Orleans, LA
Keyder, E.	Brown & Root, Inc., Houston, TX
Khader, G. S.	Texas A & M University (Student), College Station, TX
Kinra, R. K.	Shell Oil Company, Houston, TX

LaBoube, R. A.	Butler Manufacturing Company, Grandview, MO
Lai, D. C.	Amoco Production Co., Tulsa, OK
Laurendine, T. T.	Chevron, U.S.A., New Orleans, LA
Lee, G. C.	State University of New York, Amherst, NY
Levert, F.	Texaco, Inc., New Orleans, LA
Liu, S. C.	National Science Foundation, Washington, D.C.
Llorente, C.	McDermott, Inc., New Orleans, LA
Lloyd, J. R.	Exxon Production Research Co., Houston, TX
Lu, L. W.	Lehigh University, Bethlehem, PA
Manguno, S. L.	Texaco, Inc., New Orleans, LA
Mansour, A.	University of California, Berkeley, CA
Maquoi, R. J. H.	Universite of Liege, Liege, Belgium
Marsh, C.	Concordia University, Montreal, Quebec, Canada
Marshall, P. W.	Shell Oil Company, Houston, TX
Martin, K.	McDermott, Inc., New Orleans, LA
Matsui, C.	Lehigh University, Bethlehem, PA
May, C.	Chevron, U.S.A., New Orleans, LA
McDermott, R. J.	McDermott Marine Engineering, Houston, TX
Meith, R. M.	Chevron, U.S.A., New Orleans, LA
Milek, W. A.	American Institute of Steel Construction, Chicago, IL
Miller, C. D.	Chicago Bridge & Iron Company, Plainfield, IL
Moore, E. D.	Bethlehem Steel Corp., Beaumont, TX
Morrison, D. G.	McDermott, Inc., New Orleans, LA
Moses, F.	Case Institute, Cleveland, OH
Moughler, C. A.	Texaco, Inc., New Orleans, LA
Moynan, D. A.	Walk, Haydel & Assoc., Inc., New Orleans, LA
Ohmart, R. D.	Conoco, Inc., Ponca City, OK
Ostapenko, A.	Lehigh University, Bethlehem, PA
Palmer, F. J.	Copperweld Corp., Pittsburgh, PA
Pan, C. S.	Tulane University (Student), New Orleans, LA
Patron, A. S.	Walk, Haydel & Assoc., Inc., New Orleans, LA
Pillai, S. U.	Royal Military College of Canada, Kingston, Ontario, Canada
Polyzois, D.	University of Texas, Austin, TX
Ramey, R.	Chevron, U.S.A., New Orleans, LA
Rathke, R. M.	Texaco, Inc., New Orleans, LA
Reab, K.	Tulane University (Student), New Orleans, LA
Regl, R.	McDermott, Inc., New Orleans, LA
Roberson, W.	McDermott, Inc., New Orleans, LA
Roberts, W.	Hallrow Ewbank Corp., Houston, TX
Ross, D. A.	University of Akron, Akron, OH
Roussel, H. J.	Roussel Engr., Inc. & Tulane University, Kenner, LA
Saccone, C.	University of New Orleans (Student), New Orleans, LA
Sahrmann, G. J.	McDermott, Inc., New Orleans, LA
Sebesta, R. J.	Chevron, U.S.A., New Orleans, LA
Serena, A. L.	McDermott, Inc., New Orleans, LA
Serrahn, C. S.	Earl and Wright, San Francisco, CA
Sherman, D. R.	University of Wisconsin-Milwaukee, Milwaukee, WI

Simitses, G. J.	Georgia Institute of Technology, Atlanta, GA
Singer, J.	Israel Institute of Technology, Haifa, Israel
Springfield, J.	Carruthers & Wallace Limited, Rexdale, Ontario, Canada
Sridharan, S.	Washington University, St. Louis, MO
Stanley, S. B.	Bethlehem Steel Corp., Beaumont, TX
Stelly, C. W.	Pennzoil Company, Houston, TX
Stevens, D.	Chevron, U.S.A., New Orleans, LA
Stiansen, S. G.	American Bureau of Shipping, New York, NY
Stringer, D. C.	AMCA International, Ottawa, Ontario, Canada
Stroup, C.	McDermott, Inc., New Orleans, LA
Supple, W. J.	J P Kenny & Partners, London, England
Supornsilaphachai, B.	E. B. Ludwig Steel Corp., Harahan, LA
Sybert, J. H.	Chevron, U.S.A., New Orleans, LA
Tall, L.	Florida International University, Miami, FL
Temple, M. C.	University of Windsor, Windsor, Ontario, Canada
Testa, R. B.	Columbia University, New York, NY
Tucker, O.	University of New Orleans (Student), New Orleans, LA
Velez, P. K.	Shell Offshore Inc., New Orleans, LA
Versowsky, P.	Chevron, U.S.A., New Orleans, LA
Vinnakota, R. S.	Marquette University, Milwaukee, WI
Wang, C.	Texaco, Inc., New Orleans, LA
Wankmuller, R.	Shell Offshore Inc., New Orleans, LA
Warden, Y. S.	Mobil Research and Development Corp., Dallas, TX
Whitley, J. O.	Bethlehem Steel Corp., Beaumont, TX
Winter, G.	Cornell University, Ithaca, NY
Wisch, D.	Texaco, Inc., New Orleans, LA
Wolchuk, R.	Wolchuk & Mayrbaurl, New York, NY
Wong, J.	E. B. Ludwig Steel Corp., Harahan, LA
Yoo, C. H.	Auburn University, Auburn, AL
Yu, W. W.	University of Missouri-Rolla, Rolla, MO
Zellin, M. A.	Sverdrup & Parcel & Associates, Inc., St. Louis, MO
Zimmer, R.	Conoco, Inc., Houston, TX

List of Publications

The following papers and reports have been received at Headquarters and have been placed in the SSRC library.

Chapuis, J. and Galambos, T. V.
RESTRAINED CROOKED ALUMINUM COLUMNS, Journal of the Structural Division, ASCE, Vol. 108, No. ST3, Proc. Paper 16937, March, 1982

Chapuis, J. and Galambos, T. V.
RELIABILITY OF ALUMINUM BEAM-COLUMNS, Journal of the Structural Division, ASCE, Vol. 108, No. ST4, Proc. Paper 17019, April, 1982

Cuk, P. E. and Trahair, N. S.
BUCKLING OF BEAMS WITH CONCENTRATED MOMENTS, The University of Sydney, School of Civil and Mining Engineering, Research Report Series, Research Report No. R401, September, 1981

Dux, P. F. and Kitipornchai, S.
INELASTIC BEAM BUCKLING EXPERIMENTS, University of Queensland, Department of Civil Engineering, Research Report Series, Research Report No. CE24, May, 1981

Ellis, J. S.
PRESTRESSED LATTICED BEAM-COLUMNS WITH OFFSET DIAGONALS, Canadian Journal of Civil Engineering, Vol. 7, Number 4, 1980, pp. 573-587

Ely, J. F.
STRENGTH OF DAMAGED END-POSTS (Recollections of Truss Bridge Research Project), 1982

European Convention for Constructural Steelwork, COMPOSITE STRUCTURES, The Construction Press Ltd, New York, 1982

Goschy, B.
STABILITY OF CORE-SUPPORTED STRUCTURES, Hungarian Academy of Sciences, Tomus 87 (1-2), pp. 59-68 (1978)

Lui, E. M. and Chen, W. F.
STRENGTH OF H-COLUMNS WITH SMALL END RESTRAINTS, School of Civil Engineering, Purdue University, West Lafayette, IN 47907, November, 1981

Lui, E. M. and Chen, W. F.
END RESTRAINT AND COLUMN DESING USING LRFD, Structural Engineering CE-STR-82-24 (1982), Purdue University

Mateescu, D., Appeltauer, L. and Cuteanu, E.
STABILITATEQ LA COMPRESIUNE A STRUCTURILOR DIN BARE DE OTEL, (STABILITY OF COMPRESSION MEMBERS IN STEEL STRUCTURES), Editura Academiei Republicii Socialiste România R-79717 Bucuresti, Calea Victoriei nr. 125, (Summary in English), 1980

Nylander, B.
DIMENSIONERING AV STANG MED TUNNVÄGGIGT SLUTET TVÄRSNITT MED HÄNSYN TILL KNÄCKNING OCH BUCKLING, (DIMENSIONING OF STEEL BOX COLUMNS), Meddelande nr 136, Institutionen för Byggnadsstatik, Kgl Tekniska Högskolan, Stockholm 1980, (Summary in English)

Rotter, J. M.
RAPID EXACT INELASTIC BIAXIAL BENDING ANALYSIS, The University of Sydney, School of Civil and Mining Engineering, Research Report Series, Research Report No. R405, December, 1981

Simitses, G. J.
RESPONSE OF SUDDENLY-LOADED STRUCTURAL CONFIGURATIONS, ASCE Preprint No. 82-509, ASCE Convention, New Orleans, 1982

Simitses, G. J., Sheinman, I., and Shaw, D.
STABILITY OF LAMINATED COMPOSITE SHELLS SUBJECTED TO UNIFORM AXIAL COMPRESSION AND TORSION, Georgia Institute of Technology, Interin Report, June 1981-June 1982

Structural Stability Research Council - Task Group 6
DETERMINATION OF RESIDULA STRESSES IN STRUCTURAL SHAPES, Reprinted from EXPERIMENTAL TECHNIQUES, Vol. 5, No. 3, 4-7, Sept. 1981

Structural Stability Research Council, European Convention for Constructural Steelwork, Column Research Committee of Japan, and Council of Mutual Economic Assistance
STABILITY OF METAL STRUCTURES - A WORLD VIEW, Engineering Journal, AISC, Second Quarter, 1982, Vol. 19, No. 2, pp. 101-138

Structural Stability Research Council, European Convention for Constructural Steelwork, Column Research Committee of Japan, and Council of Mutual Economic Assistance
STABILITY OF METAL STRUCTURES - A WORLD VIEW (PART 2), Engineering Journal, AISC, Fourth Quarter, 1981, Vol. 18, No. 4, pp. 129-196

Structural Stability Research Council, European Convention for Constructural Steelwork, Column Research Committee of Japan, and Council of Mutual Economic Assistance
STABILITY OF METAL STRUCTURES - A WORLD VIEW (PART 3), Engineering Journal, AISC, First Quarter, 1982, Vol. 19, No. 1, pp. 27-62

Wolchuk, R. F.
"PROPOSED SPECIFICATIONS FOR STEEL BOX GIRDER BRIDGES," JOURNAL OF THE STRUCTURAL DIVISION, ASCE, Vol. 106, No. ST12, Proc. Paper 15942, December, 1980, pp. 2463-2474

Finance

	Fiscal Year 10/81-9/82 Budget (approved 4/8/81)	Fiscal Year 10/81-9/82 Cash Statement 10/1/81-9/30/82	Fiscal Year 10/82-9/83 Budget (approved 3/3
BALANCE at Beginning of Period	$28,408.00	$31,657.92 (a)	$34,960.00
INCOME			
Contributions			
Sponsors			
AISC	4,000.00	4,500.00	6,000.00
AISI	5,000.00	5,000.00	9,000.00
API	1,000.00	1,000.00	1,500.00
CISC	1,000.00	1,500.00	1,500.00
MBMA	1,000.00	1,500.00	2,000.00
Participating Organizations	2,000.00	1,900.00 (b)	3,500.00
Participating Firms	4,500.00	5,800.00 (c)	7,000.00
Annual TS&M/Colloq (83)	---	---	34,000.00
Total Contributions	$18,500.00	$21,200.00	$64,500.00
Registration Fees (Annual Meeting)	4,500.00	4,400.00	5,000.00
Member-at-Large Fees	200.00	2,555.00 (d)	100.00
Guide Royalties	500.00	1,062.03	900.00
Sale of Publications	500.00	179.76	100.00
Interest	200.00	2,508.31	2,000.00
TOTAL INCOME	$24,400.00	$31,905.10	$72,600.00
EXPENDITURES			
Technical Services (Hqtrs)			
Staff Salaries	20,300.00	20,997.59 (e)	23,500.00
Supply, phone, mailing	1,800.00	2,062,61	2,000.00
Travel	500.00	317.07	200.00
Total Technical Services	$22,600.00	$23,377.27	$25,700.00
Research Support	2,000.00	1,800.00 (f)	10,000.00
Annual Meeting & Proceedings			
Annual Proceedings	3,000.00	3,016.00	3,500.00
Expenses & Services	7,500.00	7,710.37	7,000.00
Travel	6,000.00	7,065.59	12,800.00
Total Annual Mtg & Proceedings	$16,500.00	$17,791.96	$23,300.00
SSRC Guide (4th Edition)			
Expenses & Services	1,500.00}	4.42	{ 3,000.00
Travel		468.19	
Total SSRC Guide	$ 1,500.00	$ 472.61	$ 3,000.00
United Engineering Trustees	100.00	100.00	---
Travel	1,000.00	403.59 (g)	700.00
Publications	600.00	718.00 (h)	200.00
Contingencies	200.00	39.17 (i)	200.00
TOTAL EXPENDITURES	$44,500.00	$44,702.60	$63,100.00
BALANCE at End of Period	$ 8,308.00	$18,860.42 (a)	$44,460.00

EXPLANATORY NOTES

(a) Depositories

	10/1/81	9/30/82
General Account (UET)	$ 2,773.20	-0-
Technical Services (Lehigh University)	685.34	(1,125.75)
4th Edition Guide Account (Royalties)	2,892.76	3,482.18
NSF Grant (1981 ATS Chicago)	4,325.11	-0-
New Orleans Grant (1982 ATS&M)	6,379.83	-0-
Capital Preservation Fund (CPF)	14,601.71	16,503.99
	$31,657.92	$18,860.42

(b) Aluminum Association ($500); Canadian Institute of Steel Construction ($100); European Convention for Constructional Steelwork ($250); General Services Administration ($100); NASA ($350); Naval Ship Research and Development Center ($250); Structural Engineers Association of California ($100); Steel Joist Institute ($250)

(c) $125 each - Amirikian Engineering Company; Balke Engineers; Beiswenger, Hock & Associates ($100); Alfred Benesch & Company; Black & Veatch Consulting Engineers; Blauvelt Engineering Company; Bovay Engineers ($100); Butler Manufacturing Company; Brown & Root, Inc.; Carruthers & Wallace Ltd.; Chevron U.S.A. Inc.; Conoco Inc.; Copperweld Tubing Group; Edwards and Kelcey, Inc.; Feld, Kaminetzky & Cohen, P.C.; Gannett Fleming Corddry and Carpenter, Inc.; Gilbert Associates, Inc.; Greiner Engineering Sciences, Inc.; Hazelet & Erdal; Howard Needles Tammen & Bergendoff; Iffland Kavanagh Waterbury, P.C.; J P Kenny & Partners Ltd.; LeMessurier Associates/SCI; Lev Zetlin Associates, Inc.; A. G. Lichtenstein & Associates, Inc.; Loomis and Loomis, Inc.; McDermott Incorporated ($500); Modjeski and Masters; Walter P. Moore & Associates, Inc.; Parsons, Brinckerhoff, Quade and Douglas, Inc.; Richardson, Gordon and Associates; Rummel, Klepper & Khal; Sargent & Lundy; Seelye, Stevenson, Value & Knecht, Inc.; Shell Oil Company; Skidmore, Owings & Merrill; Skilling, Helle, Christiansen, Robertson, P.C.; Steinman, Boynton, Gronquist & Birdsall; Sverdrup & Parcel and Associates, Inc.; URS/John A. Blume & Associates, Engineers; URS Company; Vollmer Associates, Inc.; Weiskopf & Pickworth; Wiss, Janney, Elstner and Associates, Inc.

(d) Includes Corresponding Member contributions

(e) Staff salaries

	SSRC FUNDS	ANNUAL MEETING GRANTS NSF 81(CHI)	ANNUAL MEETING GRANTS 1982 (NO)
Director	$ 2,743.64	$ ---	$ ---
Technical Sercetary	3,331.56	---	820.45
Administrative Secretary	4,127.69	1,381.53	3,614.56
Secretarial/Clerical (includes employee benefits)	3,557.71	356.04	1,064.41
	$13,760.60	$1,737.57	$5,499.42

(f) Auburn University (C. H. Yoo)

(g) Executive Committee Meeting, Bethlehem, PA, Oct. 81, AISC subcommittee on columns (Chi), Apr. 82

(h) TM-6 reprints; Publications Brochure

(i) CPF administrative fees; foreign check adjustments

Register

Chairman:	J. S. B. Iffland	Treasurer:	J. L. Durkee
Vice Chairman:	J. Springfield	Secretaries:	G. Askar, Technical
Director:	L. S. Beedle		L. G. Federinic, Admin.

EXECUTIVE COMMITTEE

J. S. B. Iffland (82)
L. S. Beedle (Director)
J. L. Durkee**
S. J. Errera (83)
G. F. Fox (84)
T. V. Galambos (85)
R. R. Graham (84)
B. G. Johnston (84)
R. M. Meith (83)
W. A. Milek, Jr. (82)
C. D. Miller (85)
D. R. Sherman (85)
J. Springfield (83)
G. Winter*

* Past Chairman
** Past Vice Chairman

STANDING & AD HOC COMMITTEES

A. Committee on the "Guide" (84)

B. G. Johnston, Chairman
T. V. Galambos, Editor
L. S. Beedle J.S.B. Iffland
G. Winter

B. Committee on Finance (84)

G. F. Fox, Chairman
J. L. Durkee J.S.B. Iffland
L. S. Beedle R. E. Beil
J. W. Marsh

C. Committee on Technical Sessions Program (84)

S. J. Errera, Chairman
L. S. Beedle J. L. Durkee

D. Ad Hoc Committee on Bylaws (84)

J. L. Durkee G. F. Fox

E. Ad Hoc Committee on Column Formula (85)

J. Springfield, Chairman
R. Bjorhovde R. K. Kinra
W. F. Chen L. W. Lu
T. V. Galambos W. A. Milek
D. H. Hall G. Winter

TASK GROUPS

Task Group 1 - Centrally Loaded Columns

R. Bjorhovde, Chairman
L. S. Beedle
P. C. Birkemoe
W. F. Chen
J. W. Clark
J. L. Durkee
J. A. Gilligan
R. R. Graham*
D. H. Hall
B. G. Johnston
C. Marsh
T. Pekoz
L. Tall
R. Zandonini

Scope: To define the strength of centrally loaded columns, taking due account of the influence of the column geometry, the column cross-sectional geometric properties, the mechanical properties of the column material, and the variables associated with manufacture of column components and with column fabrication.

Task Group 3 - Beam-Columns

M. J. Abrahams, Chairman
W. F. Chen
L. W. Lu
D. A. Nethercot
S. U. Pillai
Z. Razzaq
J. Springfield*
S. Vinnakota

Scope: To investigate the behavior of columns subjected to uniaxial & biaxial bending, and to develop rational stability criteria based on the ultimate strength of such members.

Task Group 4 - Frame Stability and Columns as Frame Members

J. S. B. Iffland, Chairman*
M. H. Ackroyd
C. Birnstiel
M. Biswas
F. Y. Cheng
H. de Clercq
A. J. Gowens
P. Grundy
I. M. Hooper
T. Kanchanalai
L. W. Lu
H. R. Lundgren
W. A. Milek
Z. Razzaq
S. Vinnakota
C. K. Wang
J. A. Yura
M. A. Zellin

Scope: To develop procedures for investigating the stability of structural frameworks and the stability of columns as frame members.

Task Group 6 - Test Methods for Compression Members

L. Tall, Chairman
P. C. Birkemoe
R. Bjorhovde
S. J. Errera*
B. G. Johnston
T. Pekoz
D. A. Ross
R. B. Testa
H. H. Spencer
D. R. Sherman

Scope: To prepare technical memoranda on test apparatus and on techniques for testing structural members subject to buckling, and to develop procedures for interpreting the associated test data.

* Executive Committee Contact Member

Task Group 7 - Tapered Members (joint Task Group with WRC)

G. C. Lee, Chairman
A. Amirikian
D. S. Ellifritt
C. R. Felmley, Jr.
R. R. Graham*
N. Iwankiw
D. L. Johnson
G. W. Oyler
M. Yachnis

Scope: To develop practical procedures for determining the strength of tapered structural members and of frames made therefrom.

Task Group 8 - Dynamic Stability of Compression Members

D. Krajcinovic
J. Amazigo
S. S. Chen
S. M. Holzer
B. G. Johnston*
R. H. Plaut
D. Shilkrut
M. A. J. G. da Silva
G. J. Simitses
J. C. Simonis
A. E. Somers

Scope: To define the strength of columns and other compression members subjected to time-dependent loading.

Task Group 11 - International Cooperation on Stability Studies

D. Sfintesco, Chairman
W. A. Milek, V. Chairman*
G. A. Alpsten
L. S. Beedle
A. Carpena
J. H. Chen
M. Crainicescu
T. V. Galambos
M. P. Gaus
O. Halasz
J. S. B. Iffland
B. Kato
P. Marek
D. Mateescu
J. J. Melcher
G. W. Schulz
J. Strating
L. Tall
R. Zandonini

Scope: To provide liaison between national and regional research groups and to organize international colloquia in the field of stability problems. In particular, to provide liaison between SSRC task groups, the Japanese Column Research Committee, Committee 8 of the European Convention for Constructural Steelwork, and similar groups in other countries. To suggest joint research projects.

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

R. B. Testa, Chairman
G. A. Alpsten
G. F. Fox*
A. Gjelsvik
A. L. Johnson
B. G. Johnston
L. W. Lu
E. P. Popov
F. D. Sears

Scope: To obtain and interpret data on the mechanical properties in steel in the inelastic range that are of particular importance to stability problems, including the determination of the average value and variation of the following: yield stress level, yield strength, tangent modulus, secant modulus, strain-hardening modulus, and magnitude of strain at incipient strain hardening.

* Executive Committee Contact Member

Task Group 13 - Thin-walled Metal Construction

W. W. Yu, Chairman	C. Marsh	S. Sridharan
S. J. Errera	T. M. Murray	W. P. Vann
A. L. Johnson	A. Ostapenko	S. T. Wang
R. A. LaBoube	T. Pekoz	G. Winter*

Scope: To investigate the stability of flat plates and behavior of thin-walled members made of carbon steels, alloy steels, stainless steels, or aluminum alloys; and to develop stability criteria for such members, taking due account of the effects of manufacturing and the fabrication processes.

Task Group 14 - Horizontally Curved Girders

C. H. Yoo, Chairman	J. L. Durkee*	E. R. Latham
C. G. Culver	M. Elgaaly	W. A. Milek, Jr.
		M. Ojalvo

Scope: To investigate the behavior of horizontally curved girders, taking due account of the effects of rolling and fabrication practices; and to develop criteria for adequate bracing for such girders.

Task Group 15 - Laterally Unsupported Beams

Y. Fukumoto	S. Kitipornchai	M. Ojalvo
T. V. Galambos*	C. P. Mangelsdorf	N. S. Trahair
A. J. Hartman	D. A. Nethercot	J. A. Yura

Scope: To study the behavior of the develop stability criteria for laterally unsupported beams, including those in framed structures; and to determine bracing requirements for such beams.

Task Group 16 - Plate Girders

M. Elgaaly, Chairman	E. Karamuk	H. H. Spencer
P. B. Cooper	A. Lally	D. C. Stringer
J. L. Durkee*	C. Massonnet	B. T. Yen
R. S. Fountain	A. Ostapenko	R. C. Young
W. Hsiong	F. D. Sears	H. E. Waldner

Scope: To develop practical procedures for determining the ultimate strength of stiffened plate girders, and to extend these procedures to include plate girders with multiple longitudinal stiffeners.

* Executive Committee Contact Member

Task Group 17 - Doubly Curved Shells and Shell-Like Structures

C. Birnstiel, Chairman
W. J. Austin
A. Chajes
A. C. T. Chen
M. Crainicescu
W. K. Gillespie
S. X. Gunzelman
D. Krajcinovic
C. D. Miller
N. F. Morris
E. P. Popov
D. T. Sherman*
H. H. Spencer

Scope: To study the behavior of and develop stability criteria for doubly curved structures formed with continuous membranes, stiffened membranes, or reticulated frameworks.

Task Group 18 - Unstiffened Tubular Members

P. C. Birkemoe, Chairman
B. O. Almroth
M. D. Bernstein
C. Capanoglu
A. Chajes
J. W. Cox
E. D. George, Jr.
R. R. Graham
S. X. Gunzelman
C. Kao
P. W. Marshall
R. M. Meith*
C. D. Miller
F. J. Palmer
R. Regl
D. A. Ross
D. R. Sherman

Scope: To develop stability criteria for manufactured and fabricated unstiffened cylindrical tubular members, and to study the behavior of unstiffened non-cylindrical tubular members.

Task Group 20 - Composite Members & Systems

S. H. Iyengar, Chairman
P. J. Dowling
R. W. Furlong
B. Kato
J. W. Roderick
D. Sfintesco
B. Taranath
M. Wakabayashi
G. Winter*
J. Zils

Scope: To develop stability criteria for various types of composite columns, beam-cloumns and mixed steel-concrete systems.

Task Group 21 - Box Girders

R. C. Young, Chairman
G. F. Fox*
F. Moolani
B. Morgenstern
D. R. Schelling
F. D. Sears
H. H. Spencer
M. C. Tang
D. H. H. Tung
R. Wolchuk

Scope: To review, organize and interpret available information on the behavior of box girders, cooperating with other groups working on this subject; and to develop stability criteria as needed.

* Executive Committee Contact Member

Task Group 22 - Stiffened Tubular Members

C. D. Miller, Chairman
C. Babcock
M. D. Bernstein
C. Capanoglu
J. W. Cox
R. C. DeHart
P. J. Dowling
N. W. Edwards
G. Foss
S. X. Gunzelman
R. K. Kinra
R. M. Meith*
K. Minhas
R. Regl
G. J. Simitses
W. J. Supple

Scope: To investigate the stability of circular cylindrical and conical shells with longitudinal or circumferential stiffening alone or in combination. Stability criteria will be developed for local buckling and general instability type failures of cylinders and cones under axial load, external or internal pressure, beam type bending and torsion. Available test data will be compared with suggested stability criteria. Recommendations will be made for research where insufficient data is available.

Task Group 23 - Effect of End Restraint on Initially Crooked Columns

W. F. Chen, Chairman
M. H. Ackroyd
R. Bjorhovde
F. Cheong-Siat-Moy
R. O. Disque
T. V. Galambos
J. S. B. Iffland
B. Koo
D. A. Nethercot
Z. Razzaq
D. A. Ross
J. Springfield
S. Vinnakota
G. Winter*
R. Zandonini

Scope: To study the effect of end restraint on individual, initially crooked columns for which residual stress patterns are generally known.

* Executive Committee Contact Member

TASK REPORTERS

Task Reporter 11 - Stability of Aluminum Structural Members

M. L. Sharp, Aluminum Company of America

Task Reporter 13 - Local Inelastic Buckling

L. W. Lu, Lehigh University

Task Reporter 14 - Fire Effects on Structural Stability

K. H. Klippstein, U. S. Steel Corporation

Task Reporter 15 - Curved Compression Members

W. J. Austin, Rice University

Task Reporter 16 - Stiffened Plate Structures

A. E. Mansour, University of California - Berkeley

Task Reporter 17 - Laterally Unsupported Restrained Beam-Columns

L. W. Lu, Lehigh University

Task Reporter 18 - Application of Finite Element Methods to Stability Problems

R. H. Gallagher, University of Arizona

Task Reporter 19 - Creep Buckling

T. H. Lin, University of California - Los Angeles

Task Reporter 20 - Large Deflection Buckling

A. Chajes, University of Massachusetts

SPONSORS

	Representatives	Officers
American Institute of Steel Construction	J. L. Durkee R. O. Disque W. A. Milek	W. A. Milek Vice President Director of Engineering
American Iron and Steel Institute	A. L. Johnson	R. T. Willson Sr. Vice President
American Petroleum Institute	J. E. Ubben R. Regl F. P. Dunn	R. K. Kinra
*Brown & Root, Inc.	S. X. Gunzelman	D. I. Karsan Manager
Canadian Institute of Steel Construction	M. I. Gilmor D. Stringer	H. A. Krentz President
*Chevron U.S.A. Inc.	W. W. Wingerter	W. W. Wingerter Mgr. Public Affairs
*Chicago Bridge & Iron Company		C. D. Miller Dir., Structural Research
*Earl & Wright	W. M. Martinovich	W. R. Schmidt
*McDermott Incorporated	L. A. Boston S. S. Chang	G. C. Lee Vice President
Metal Building Manufacturers Association	D. L. Johnson D. S. Ellifritt C. M. Stockinger	C. M. Stockinger General Manager
*Mobil Research & Development Corporation		D. M. Coleman
*National Science Foundation		M. P. Gaus Program Manager, ASRA
*Shell Oil Company	P. W. Marshall R. K. Kinra	F. P. Dunn Manager

* Annual Technical Session Sponsor

PARTICIPATING ORGANIZATIONS

	Representatives	Officers
Aluminum Association	G. S. Gresham M. L. Sharp W. W. Pritsky	P. V. Mara Vice President Technical
American Institute of Architects	P. E. Kirven T. F. Mariani J. S. Hawkins, Jr.	H. Lawrence Director, Codes & Standards
American Society of Civil Engineers	B. G. Johnston J. S. B. Iffland J. A. Yura	E. Zwoyer Executive Director
American Society of Mechanical Engineers	M. D. Bernstein	B. E. Nelson Executive Director
Canadian Society for Civil Engineering	M. C. Temple	M. S. Mirza Executive Director
Corps of Engineers, U. S. Army	G. M. Matsumura	LTG J. K. Bratton Chief
Earthquake Engineering Research Institute	H. J. Degenkolb R. D. Hanson	J. A. Blume President
European Convention for Constructional Steelwork	A. Carpena	A. Carpena Tech. Secy. General E. B. Evers Admin. Secy General
Federal Highway Administration	R. F. Varney F. D. Sears C. Napier	L. P. Lamm Executive Director
General Services Administration	W. D. Rust	R. O. Haase Commissioner
Institution of Engineers, Australia	C. O'Connor D. S. Mansell B. E. Gorenc	J. Harding Committee Secretary
Langley Research Center, NASA	M. Stein M. M. Mikulas, Jr.	D. P. Hearth Director
National Bureau of Standards	H. S. Lew R. D. Marshall	R. N. Wright Dir., Center for Bldg Tech.
Naval Ship Research and Development Center	M. O. Critchfield R. F. Jones	CAPT M. V. Ricketts Commander
Steel Joist Institute	E. T. E. Sprague	E. T. E. Sprague Managing Director
Structural Engineers Association of California	J. O. Robb E. P. Popov G. W. Ferver	J. A. Willis President
Welding Research Council	G. C. Lee N. Iwankiw	G. W. Oyler Executive Director

PARTICIPATING FIRMS

	Representative(s)
Amirikian Engineering Co.	A. Amirikian
Ammann & Whitney	E. Cohen
Balke Engineers	C. R. Shah
Basil Engineering Corporation	C. D. Morrissey
Beiswenger, Hoch, & Associates	H. J. Graham
Alfred Benesch & Company	J. E. Barrett/R. S. Nair
Martin Berkowitz Associates	M. Berkowitz
Black & Veatch Consulting Engineers	V. H. Snell/R. P. McBean
Blauvelt Engineering Company	C. McCormack
Bovay Engineers, Inc.	K. W. Corey
Brown & Root, Inc.	S. X. Gunzelman
Butler Manufacturing Company	R. A. LaBoube
Carruthers and Wallace Limited	J. Springfield
Caudill Rowlett Scott, Inc.	
Chevron U.S.A., Inc.	W. W. Wingerter
Conoco, Inc.	N. D. Birrell/T. H. Yang
Copperweld Tubing Group	F. J. Palmer
DRC Consultants, Inc.	M. C. Tang
De Leuw Cather & Company	W. G. Horn
Delon Hampton & Associates	R. Varadarajan
Dravo Van Houten, Inc.	S. P. Lin
Edwards and Kelcey, Inc.	P. A. Olivo
Feld, Kaminetzky & Cohen, P. C.	D. Kaminetzky
Gannet Fleming Corddry and Carpenter, Inc.	M. H. Soto
Gilbert Associates	G. P. Rentschler
Green International, Inc.	I. S. Shina
Greiner Engineering Sciences, Inc.	J. M. McCummings/T. D. Jenkins
Hardesty & Hanover	H. E. Drugge
Hazelet & Erdal	F. B. Wylie, Jr.
Howard Needles Tammen & Bergendoff	G. F. Fox
Iffland Kavanagh Waterbury, P. C.	A. C. Shroff

	Representative(s)
J P Kenny & Partners Ltd.	A. C. Walker/W. J. Supple
Le Messurier Associates/SCI	W. J. Le Messurier
Lev Zetlin Associates, Inc.	R. C. Tomasetti
A. G. Lichtenstein & Associates, Inc.	A. G. Lichtenstein
Lockwood, Andrews & Newnam, Inc.	J. R. Broderick
Loomis and Loomis, Inc.	R. S. Loomis/R. W. Loomis
Chas T. Main, Inc.	V. Izbickas
McDermott Incorporated	L. A. Boston/S. S. Chang
Modjeski and Masters	J. E. Prickett
Walter P. Moore & Associates, Inc.	L. Griffis
Parsons Brinckerhoff Quade & Douglas, Inc.	M. J. Abrahams
Richardson, Gordon and Associates	A. W. Hedgren, Jr./ H. L. Burmeister, Jr.
Rummel, Klepper, & Kahl	A. L. Deen, Jr.
Sargent & Lundy	J. M. McLaughlin
Seelye Stevenson Value & Knecht, Inc.	I. M. Hooper
Shell Oil Company	P. W. Marshall/R. K. Kinra
Skidmore, Owings & Merrill	S. H. Iyengar
Skilling, Helle, Christiansen, Robertson, P. C.	L. E. Robertson
Steinman, Boynton, Gronquist & Birdsall	C. C. Ulstrup
Sverdrup & Parcel and Associates, Inc.	R. E. Beil/M. A. Zellin
Tippetts-Abbett-McCarthy-Stratton	
URS/John A. Blume & Associates, Engineers	N. Owen
URS/Company, Inc.	K. Diao
Vollmer Associates, Inc.	E. J. Moloney
Weiskopf & Pickworth	E. Pisetzner
Wiss, Janney, Elstner, and Associates, Inc.	A. T. Derecho

M E M B E R S - A T - L A R G E

M. J. Abrahams
P. F. Adams
R. J. Alvarez
G. Askar

W. F. Baker
P. C. Birkemoe
C. Birnstiel
R. Bjorhovde
D. O. Brush
K. P. Buchert

C. C. Capanoglu
A. Chajes
W. F. Chen
F. Y. Cheng
F. Cheong-Siat-Moy
P. B. Cooper
J. W. Cox

P. J. Dowling
G. C. Driscoll, Jr.
J. L. Durkee

M. Elgaaly
S. J. Errera
L. F. Estenssoro

G. F. Fox
R. W. Furlong

T. V. Galambos
R. H. Gallagher
M. P. Gaus
S. C. Goel
A. J. Gouwens
R. R. Graham

G. Haaijer
D. H. Hall
A. J. Hartmann
G. Herrmann
I. M. Hooper
T. Huang

J. S. Iffland
L. Ingvarsson
S. H. Iyengar

R. K. Kinra
S. Kitipornchai
D. Krajcinovic

A. Lally
G. C. Lee
W. J. LeMessurier
F. J. Lin
T. H. Lin
L. W. Lu
H. R. Lundgren

J. R. Maison
C. Marsh
P. W. Marshall
B. M. McNamee
R. M. Meith
M. L. Morrell
T. M. Murray

M. Ojalvo
A. Ostapenko

T. Pekoz
S. U. Pillai
C. W. Pinkam

Z. Razzaq
B. C. Ringo
L. E. Robertson
D. A. Ross

G. W. Schulz
Z. Y. Shen
D. R. Sherman
G. J. Simitses
H. H. Spencer
J. Springfield
F. W. Stockwell

L. Tall
M. C. Temple
R. B. Testa
S. S. Thomaides

F. Van Der Woude
W. P. Vann
S. Vinnakota

C. K. Wang
S. T. Wang
R. Wolchuk
E. W. Wright

B. T. Yen
C. H. Yoo
R. C. Young
W. W. Yu
J. A. Yura

R. Zandonini

CORRESPONDING MEMBERS

G.	Augusti	Italy
M. N.	Aydinoglu	Turkey
M.	Crainicescu	Romania
H.	de Clercq	South Africa
J. B.	Dwight	England
L.	Finzi	Italy
Y.	Fujita	Japan
M. S.	Gregory	Australia
P.	Grundy	Australia
O.	Halasz	Hungary
T.	Kanchanalai	Thailand
A. N.	Kounadis	Greece
R. W.	Kowalczyk	Poland
P. K.	Larsen	Norway
C.	Massonnet	Belgium
J.	Melcher	Czechoslovakia
D.	Sfintesco	France
K.	Thomsen	Denmark
B.	Thurlimann	Switzerland
N. S.	Trahair	Australia
U.	Vogel	West Germany

LIFE MEMBERS

	Year Elected
A. Amirikian	1978
W. J. Austin	1976
L. S. Beedle	1976
J. W. Clark	1980
E. L. Erickson	1976
E. H. Gaylord	1976
J. A. Gilligan	1976
J. E. Goldberg	1976
T. R. Higgins	1976
N. J. Hoff	1976
L. K. Irwin	1976
B. G. Johnston	1976
R. L. Ketter	1976
D. Sfintesco	1982
B. Thurlimann	1976

S S R C A D D R E S S E S

* ABRAHAMS, Michael J., Parsons, Brinckerhoff, Quade & Douglas, One Penn Plaza, 250 West 34th Street, New York, New York 10119

ACKROYD, Prof. Michael H., Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, New York 12181

* ADAMS, Dr. Peter F., Dean, Faculty of Engineering, The University of Alberta, Edmonton, Alberta T6G 2G8 Canada

ALMROTH, Dr. B. O., Lockheed Research Laboratory, 3251 Hanover Street, Palo Alto, California 94304

ALPSTEN, Dr. Goran A., Stalbyggnadskontroll AB, Tralgatan 16, S-133 00 Saltsjobaden, Sweden

* ALVAREZ, Prof. Ronald J., Engineering & Computer Sciences, Hofstra University, Hempstead, New York 11550

AMAZIGO, Prof. J., Department of Mathematics, University of Nigeria, Nsukka, Nigeria

* AMIRIKIAN, Dr. Arsham, Amirikian Engineering Co., 35 Wisconsin Circle, Chevy Chase, Maryland 20015

ARGUS, Ballard, Argus Technical Systems, Inc., 2709 Erato Street, P.O. Box 50066, New Orleans, Louisiana 70150

ARNDT, Arthur P., Vice-President - Engineering, American Bridge Division, Airport Office Park - Building 3, 420 Rouse Road, Coraopolis, Pennsylvania 15108

* ASKAR, Dr. Gulay, I. T. U. Muh-Mim, Fakultesi, Insaat Bolumu, Macka-Istanbul, Turkey

ATSUTA, Dr. Toshio, Assistant Manager, Welding Research Laboratory, Technical Institute, Kawasaki Heavy Industries, Ltd., 1-1, Hagashikawasaki-Cho, 3-Chome, Chuo-Ku, Kobe 650-91 Japan

* AUGUSTI, Prof. Guiliano, Facolta di Ingegneria, Universita Degli Studi Di Firenze, Via Di S Maria, 3, I 50139 Florence, Italy

* AUSTIN, Prof. Walter J., Department of Civil Engineering, P.O. Box 1892, Rice University, Houston, Texas 77001

* AYDINOGLU, Dr. M. Nuray, Istanbul Teknik Universitesi, Macka Insaat Fakultesi, Macka/Besiktas, Istanbul, Turkey

BABCOCK, Prof. Charles D., Professor of Aeronautics, California Institute of Technology, 105-50, Pasadena, California 91125

*** SSRC Member**

* BAKER, William F., Skidmore, Owings & Merrill, 33 West Monroe Street, Chicago, Illinois 60603

* BARRETT, Jack E., Alfred Benesch & Company, 233 North Michigan Avenue, Chicago, Illinois 60601

BARTON, Dr. Cliff S., Dean, College of Engineering Sciences & Technology, 270 Clyde Building, Brigham Young University, Provo, Utah 84602

* BEEDLE, Dr. Lynn S., Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015

* BEIL, Ralph E., Vice-President, Sverdrup & Parcel & Associates, Inc., 801 North 11th Street, St. Louis, Missouri 63101

* BERKOWITZ, Martin, Martin Berkowitz Associates, Consulting Engineers, 1896 Morris Avenue, Union, New Jersey 07083

* BERNSTEIN, Martin D., Foster Wheeler Energy Corporation, 110 South Orange Avenue, Livingston, New Jersey 07039

* BIRKEMOE, Prof. Peter C., Department of Civil Engineering, Galbraith Building, University of Toronto, Toronto, Ontario M5S 1A4 Canada

* BIRNSTIEL, Dr. Charles, Consulting Engineer, 108-18 Queens Boulevard, Forest Hills, New York 11375

* BIRRELL, N. D., Chief Marine Engineer, Production Engineering Services, Conoco Incorporated, P. O. Box 2197, Houston, Texas 77252

BISWAS, Dr. Mrinmay, Department of Civil Engineering, Texas A & M University, College Station, Texas 77843

* BJORHOVDE, Prof. Reider, Department of Civil Engineering, University of Arizona, Tuscon, Arizona 85721

BLUME, John A., President, Earthquake Engineering Research Institute, 2620 Telegraph Avenue, Berkeley, California 94704

* BOSTON, Lawrence A., McDermott Incorporated, P. O. Box 60035, New Orleans, Louisiana 70160

BRANNON, H. R., Jr., Senior Research Scientist, Exxon Production Research Company, P. O. Box 2189, Houston, Texas 77001

BRATTON, LTG J. K., Chief of Engineers, U. S. Army Corps of Engineers, Washington, D. C. 20314

* BRODERICK, J. Richard, Lockwood, Andrews & Newnam, Inc., 1900 St. James Place, Houston, Texas 77056

BRUEGGING, Jerry, Butler Manjfacturing Company Research Center, 135th & Botts Road, Grandview, Missouri 64030

* BRUSH, Prof. Don O., Department of Civil Engineering, University of California, Davis, Davis, California 94616

BUBENIK, Dr. Thomas A., Platform Analysis Section, Offshore Structures Division, Exxon Production Research Company, P. O. Box 2189, Houston, Texas 77001

* BUCHERT, Dr. Kenneth P., Bechtel Power Corporation, TPO, (50) 11 A1, P. O. Box 3965, San Francisco, California 94119

* BURMEISTER, Harold L., Jr., Richardson, Gordon and Associates, Inc., Three Gateway Center, Pittsburgh, Pennsylvania 15222

CANTY, Don, Editor, AIA Journal, The American Institute of Architects, 1735 New York Avenue, N. W., Washington, D. C. 20006

* CAPANOGLU, Cuneyt C., Earl and Wright, One Market Plaza, Spear Street Tower, San Francisco, California 94105

* CARPENA, Dr. Ing. A., ECCS Tech. Genl. Secretariat, Avenue Louise 326, Bte 52, B-1050 Brussels, Belgium

* CHAJES, Prof. Alexander, Department of Civil Engineering, University of Massachusetts, Amherst, Massachusetts 01003

* CHANG, S. S., McDermott Incorporated, P. O. Box 60035, New Orleans, Louisiana 70160

CHEN, Dr. Andrie C. T., Exxon Production Research Company, P. O. Box 2189, Houston, Texas 77001

CHEN, Dr. Jian-Hei, Deputy Chief Research Engineer, General Research Institute of Building and Construction, Ministry of Metallurgical Industry, 43 Xuc Yuan Road, Beijing, People's Republic of China

CHEN, Prof. Shao-Fan, Professor of Structural Engineering, Xian Institute of Metallurgy and Construction Engineering, Xian, Shaanxi, People's Republic of China

CHEN, Dr. Shoei-Sheng, Components Technology Division, Argonne National Laboratory, 9700 South Cass Avenue, Argonne, Illinois 60439

* CHEN, Prof. Wai-Fan, School of Civil Engineering, Civil Engineering Building, Purdue University, West Lafayette, Indiana 47907

* CHENG, Dr. Franklin Y.,Department of Civil Engineering, University of Missouri, Rolla, Missouri 65401

* CHEONG-SIAT-MOY, Dr. Francois, Department of Civil Engineering, California State University, Sacramento, Sacramento, California 95819

CHONG, Dr. Ken P., Department of Civil & Architectural Engineering, College of Engineering, The University of Wyoming, P. O. Box 3295, University Station, Laramie, Wyoming 82071

* CLARK, Dr. John W., Technical Advisor, Alcoa Labs, 904 Farragut Street, Pittsburgh, Pennsylvania 15206

* COHEN, Edward, Ammann & Whitney, Two World Trade Center, New York, New York 10048

COLEMAN, D. M., Mobil Research & Development Corporation, P. O. Box 900 (FRL), Dallas, Texas 75221

CONNOR, Sam, Director, Public Information, Alumni Building #27, Lehigh University, Bethlehem, Pennsylvania 18015

* COOPER, Prof. Peter B., Department of Civil Engineering, Seaton Hall, Kansas State University, Manhattan, Kansas 66506

* COREY, Ken W., Bovay Engineers, Inc., 5619 Fannin Street, P. O. Box 8098, Houston, Texas 77004

CORNELL, Prof. C. Allin, Department of Civil Engineering, Room 1-263, Massachusetts Institute of Technology, Cambridge, Massachusetts 02139

* COX, Dr. John W., Tera, Inc., P. O. Box 740038, Houston, Texas 77081

* CRAINICESCU, Ms. Magda, Institutul de Cercetari in Constructii si Economia Constructiilor, Sos. Pantelimon 266, Bucuresti, Romania

* CRITCHFIELD, Dr. Milton O., Code 1730.5, David W. Taylor Naval Ship Research & Development Center, Bethesda, Maryland 20084

CULVER, Dr. Charles G., Office of Federal Building, Technical Building 226, Room B244 National Bureau of Standards, Washington, D. C. 20234

DANIELS, Dr. J. Hartley, Fritz Engineering Laboratory #13, Lehigh University, Bethlehem, Pennsylvania 18015

DAVEY, Tom, Managing Editor, Canadian Consulting Engineer, 1450 Don Mills Road, Don Mills, Ontario, Canada

DAVIS, C. S., 19460 Burlington Drive, Detroit, Michigan 48203

* de CLERCQ, Dr. Hennie, Van Wyk & Louw Inc., P. O. Box 31798, Braamfontein, 2017 South Africa

* DEEN, Albert L., Jr., Rummel, Klepper & Kahl, 1035 N. Calvert Street, Baltimore, Maryland 21202

* DEGENKOLB, Henry J., H. J. Degenkolb & Associates, 350 Sansome Street, San Francisco, California 94104

DEHART, Robert C., Vice-President, Southwest Research Institute, 8500 Culebra Road, P. O. Drawer 28510, San Antonio, Texas 78283

* DERECHO, Dr. A. T., Wiss, Janney, Elstner and Associates, Inc., 330 Pfingsten Road, Northbrook, Illinois 60062

* DIAO, Kenneth, Vice-President, URS Company, Inc., 370 Seventh Avenue, New York, New York, 10001

* DISQUE, Robert O., American Institute of Steel Construction, Wrigley Building, 400 North Michigan Avenue, Chicago, Illinois 60611-4185

DORTON, Dr. Roger A., Chairman, CSCE Structural Division, Structural Office, Ministry of Transportation & Communications, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8 Canada

DOSHI, Kishar P., Iffland Kavanagh Waterbury, P. C., 1501 Braodway, New York, New York 10036

* DOWLING, Prof. Patrick J., Department of Civil Engineering, Imperial College of Science & Technology, London, SW7 2BU England

* DRISCOLL, Prof. George C., Fritz Engineering Laboratory #13, Lehigh University, Bethlehem, Pennsylvania 18015

* DRUGGE, H. Everett, Hardesty and Hanover, 1501 Broadway, New York, New York 10036

du BOUCHET, Prof. Andres V., 9 Meadowlark Lane, East Brunswick, New Jersey 08816

* DUNN, F. P., Manager, Civil Engineering Department, Shell Oil Company, P. O. Box 2099, Houston, Texas 77001

* DURKEE, Jackson L., Consulting Structural Engineer, 217 Pine Top Trail, Bethlehem, Pennsylvania 18017

* DWIGHT, John B., Reader in Structural Engineering, Engineering Department, Cambridge University, Cambridge CB2 1PZ England

EDLUND, Prof. Bo L. O., Chalmers University of Technology, Department of Structural Engineering, Steel and Timber Structures, S-41296 Goeteborg, Sweden

EDWARDS, Dr. Norman W., Nutech, Inc., 145 Martinvale Lane, San Jose, California 95119

* ELGAALY, Dr. Mohamed, Bechtel Associates Professional Corporation, 2342 Delaware, Ann Arbor, Michigan 48103

* ELLIFRITT, Dr. Duane S., Director of Engineering & Research, Metal Building Manjfacturers Association, 1230 Keith Building, Cleveland, Ohio 44115-2180

ELLIOTT, Arthur L., 3010 Tenth Avenue, Sacramento, California 95817

ELLIS, Dr. John S., Head, Department of Civil Engineering, Royal Military College of Canada, Kingston, Ontario, K7L 2W3 Canada

* ERICKSON, Eric L., 501 Dumbarton Drive, Shreveport, Louisiana 71106

* ERRERA, Dr. Samuel J., Room 1797 Martin Tower, Bethlehem Steel Corporation, Bethlehem, Pennsylvania 18016

* ESTENSSORO, Luis F., Wiss, Janney, Elstner & Associates, 330 Pfingsten Road, Northbrook, Illinois 60062

EVERS, E. B., ECCS Administrative Secretary General, Posthus 20714, NL-3001 JA Rotterdam, The Netherlands

FAIRWEATHER, Virginia, Editor, ASCE News, 345 East 47th Street, New York, New York 10017

FANG, Pen J., Department of Civil & Environmental Engineering, University of Rhode Island, Kingston, Rhode Island 02881

FEDERINIC, Mrs. Lesleigh G., SSRC Administrative Secretary, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015

FELDT, Prof. W. T., Division of Engineering Science, University of Wisconsin-Parkside, Kenosha, Wisconsin 53140

FELMLEY, C. R., Jr., Technical Secretary, Welding Research Council, 345 East 47th Street, New York, New York 10017

* FERVER, Greer W., Ferver Engineering Company, 3487 Kurtz Street, San Diego, California 92110

* FINZI, Prof. Leo, Viale Guistininiano 10, 20129 Milano, Italy

FISHER, Dr. Gordon P., School of Civil and Environmental Engineering, 309 Hollister Hall, Cornell University, Ithaca, New York 14853

FLEISCHER, Walter H., G S O 248, Bethlehem Steel Corporation, Bethlehem, Pennsylvania 18016

FOSS, Gunnar, Det Norske Veritas, Section for Steel Structures, Industrial & Offshore Division, P. O. Box 300, N-1322 Hovik Oslo Norway

FOUNTAIN, Richard S., Highway Construction Marketing, United States Steel Corporation, Pittsburgh, Pennsylvania 15222

FOWLER, Prof. David W., Department of Architectural Engineering, University of Texas at Austin, Austin, Texas 78712

FOX, Arthur J., Publisher, Engineering News Record, 330 West 42nd Street, New York, New York 10036

* FOX, Gerard F., Howard, Needles, Tammen & Bergendoff, 1345 Avenue of the Americas, New York, New York 10105

FREEMAN, Dr. B. Gail, Cogswell/Hausler Associates, P. O. Drawer 2678, Chapel Hill, North Carolina 27712

* FUJITA, Prof. Yuzuru, Department of Naval Architecture, The Faculty of Engineering, University of Tokyo, Bunkyo-Ku, Tokyo 113 Japan

FUKUMOTO, Dr. Yuhshi, Department of Civil Engineering, Nagoya University, Chiksusa-Ku, Nagoya, Japan

* FURLONG, Dr. Richard W., Department of Civil Engineering, University of Texas at Austin, Austin, Texas 78712

* GALAMBOS, Prof. Theodore V., Department of Civil & Mineral Engineering, University of Minnesota, 112 Mines and Metallurgy, 221 Church Street, S. E., Minneapolis, Minnesota 55455

GALIONE, Kenneth A., Managing Director, Society for Experimental Stress Analysis, 21 Bridge Square, P. O. Box 277, Saugatuck Station, Westport, Connecticut 06880

* GALLAGHER, Dr. Richard H., Dean, College of Engineering, University of Arizona, Tucson, Arizona 85721

* GAUS, Dr. Michael P., Program Manager, Design Research, Division of Problem Focused Research Application, Directorate ASRA, National Science Foundation, Washington, D. C. 20550

GAYLORD, Prof. Charles N., Department of Civil Engineering, Thornton Hall, University of Virginia, Charlottesville, Virginia 22901

* GAYLORD, Prof. Edwin H., 2129 Newmark Lab, Civil Engineering, University of Illinois, Urbana, Illinois 61801

GEORGE, Ernest D., Jr., Jordan, Apostal, Ritter Associates, Inc., Administration Building 7, Davisville, Rhode Island 02854

GILLESPIE, W. K., Chief Structural Engineer, Pittsburgh-Des Moines Steel Company, Neville Island, Pittsburgh, Pennsylvania 15225

* GILLIGAN, John A., Manager, Special Technical Service-Metallurgy, U. S. Steel Corporation, 600 Grant Street, Room 1780, Pittsburgh, Pennsylvania 15230

* GILMOR, Michael I., Manager of Engineering, Canadian Institute of Steel Construction, 201 Consumers Road, Suite 300, Willowdale, Ontario M2J 4G8 Canada

GJELSVIK, Prof. Atle, Department of Civil Engineering, S. W. Mudd Building, Columbia University, New York, New York 10027

GLASFELD, Dr. R., General Dynamics Corporation, 97 Howard Street, Quincy, Massachusetts 02169

GODFREY, Kneeland A., Jr., Editor, Civil Engineering, A S C E, 345 East 47th Street, New York, New York 10017

* GOEL, Prof. S. C., Professor of Civil Engineering, The University of Michigan, Ann Arbor, Michigan 48104

GOLAY, A., IABSE Bulletin, AIPC-IVBH-IABSE, ETH - Honggerberg, CH-8093 Zurich, Switzerland

* GOLDBERG, Dr. John E., Structural Mechanics Division, National Science Foundation, Washington, D. C. 20550

* GORENC, B. E., MacDonald, Wagner & Priddle, Consulting Engineers, 100 Miller Street, North Sydney 2060 Australia

* GOUWENS, Albert J., Manager of Structural Engineering, Goodell-Grivas, Inc., 17320 West Eight Mile Road, Southfield, Michigan 48075

* GRAHAM, Harry J., Beiswenger, Hoch & Associates, Inc., P. O. Box 1368, North Miami Beach, Florida 33160

* GRAHAM, Roland R., Consultant, 1020 Lakemont Drive, Pittsburgh, Pennsylvania 15243

* GREGORY, Dr. Malcolm S., Civil Engineering Department, University of Tasmania, Box 252C, G.O.P. Hobart, Tasmania, Australia 7001

* GRESHAM, G. S., Reynolds Metals Company, 5th & Cary Streets, Richmond, Virginia 23261

* GRIFFIS, Lawrence G., Walter P. Moore & Associates, Inc., 2905 Sackett Street, Houston, Texas 77098

* GRUNDY, Prof. Paul, Department of Civil Engineering, Monash University, Clayton, Victoria 3168 Australia

* GUNZELMAN, Stephen X., Design Engineer, Brown & Root, Inc., P. O. Box 3, Houston, Texas 77001

* HAAIJER, Dr. Geerhard, 110 Moonlight Drive, Monroeville, Pennsylvania 15146

HAASE, Richard O., Commissioner, General Services Administration, Washington, D. C. 20405

* HALASZ, Dr. Otto, Department of Steel Structures, Technical University Budapest, XI Muegyetem rkp 3, H-1512 Budapest, Hungary

* HALL, Dann H., Consulting Engineer, Room 1733 Martin Tower, Bethlehem Steel Corporation, Bethlehem, Pennsylvania 18016

* HANSON, Prof. Robert D., University of Michigan, Department of Civil Engineering, Ann Arbor, Michigan 48109

HARDESTY, Egbert R., Hardesty &Hanover, 1501 Broadway, New York, New York 10036

HARDING, John, Committee Secretary, The Institution of Engineers, Australia, National Headquarters, 11 National Circuit, Barton, A. C. T., Australia 2600

HARPER, C. D., Journal of Institution of Engineers, 11 National Circuit, Barton, A. C. T. Australia 2600

HARRIS, Dr. Harry G., Department of Civil Engineering, Drexel University, 32nd and Chestnut Streets, Philadelphia, Pennsylvania 19104

* HARTMANN, Dr. A. J., 1161 Colgate Drive, Monroeville, Pennsylvania 15146

* HAWKINS, Jasper S., Hawkins Lindsey Wilson Associates, 111 East Camelback Road, Phoenix, Arizona 85012

HAYASHI, Prof. Tsuyoshi, Department of Mathematics, Faculty of Science & Engineering, Chuo University, I-13-27 Kasuga, Bunkyo-Ku, Tokyo 112

HEALY, Dr. John J., Consulting Engineer, Ebasco Services, Inc., Two World Trade Center, 91st Floor, New York, New York 10048

HEARTH, Donald P., Director, Langley Research Center, N A S A, Hampton, Virginia 23665

HECHTMAN, Dr. Robert A., Environmental & Urban Systems, Virginia Polytechnic Institute and State University, Blacksburg, Virginia 24061

* HEDGREN, Arthur W., Jr., Richardson, Gordon and Associates, Inc., 3 Gateway Center, Pittsburgh, Pennsylvania 15222

* HERRMANN, Prof. G., Division of Applied Mechanics - Durand Building, Department of Mechanical Engineering, School of Engineering, School of Engineering, Stanford University, Stanford, California 94305

* HIGGINS, Dr. Theodore R., Epsom Manor Retirement Apartments, Apt. 378, Epsom, New Hampshire 13234

* HOFF, Dr. Nicholas J., 782 Esplanada Way, Stanford, California 94305

HOLLISTER, Dean S. C. - deceased 1982

HOLZER, Dr. Siegfried M., Civil Engineering Department, Virginia PolytechnicInstitute & State University, Blacksburg, Virginia 24061

HOOLEY, Dr. Roy F., Department of Civil Engineering, University of British Columbia, Vancouver, B. C. Canada

* HOOPER, Ira M., Vice-President, Seelye Stevenson Value & Knecht, 99 Park Avenue, New York, New York 10016

HOTCHKIES, James W., Algoma Steel Corporation Limited, Royal Bank Plaza, Suite 2850, P. O. Box 12, Toronto, Ontario M5J 2J1 Canada

HSIONG, Wei, MTA, Incorporated, 6420 South Sixth Street, Frontage Road, Springfield, Illinois 62703

* HUANG, Prof. Tseng, Department of Civil Engineering, University of Texas at Arlington, P. O. Box 19308, Arlington, Texas 76019

* IFFLAND, Jerome S. B., Iffland Kavanagh Waterbury, P. C., 1501 Broadway, New York, New York 10036

* INGVARSSON, Dr. Lars, Avd. DBF, Dobel, 781 84 Borlange, Sweden

* IRWIN, Lafayette K., P. O. Box 487, Camden, South Carolina 29020

* IWANKIW, Nestor, Assistant Director, Engineering & Research, American Institute of Steel Construction, 400 N. Michigan Avenue, Chicago, Illinois 60611

* IYENGAR, Srinivasa H., Skidmore, Owings & Merrill, 33 West Monroe Street, Chicago, Illinois 60603

* IZBICKAS, Vytautas, Chas. T. Main, Inc., Southeast Tower, Prudential Center, Boston, Maryland 02199

JACOBS, G. V., 24 Morse Lane, Woodside, California 94062

JANSEN, T. Paul, De Serio-Jansen Engineers, PC, 511 Root Building, 86 West Chippewa Street, Buffalo, New York 14202

* JENKINS, Thomas D., Greiner Engineering Sciences, Inc., One Village Square, Village of Cross Keys, Baltimore, Maryland 21210

JOHNS, Dr. Thomas G., BATTELLE, Houston Operations, 2223 West Loop South, Suite 320, Houston, Texas 77027

* JOHNSON, Dr. Albert L., American Iron & Steel Institute, 1000 16th Street, N. W., Washington, D. C. 20036

* JOHNSON, Donald L., Butler Manjfacturing Company, Research Center, 135th Street & Botts Road, Grandview, Missouri 64030

* JOHNSTON, Prof. Bruce G., 5025 East Calle Barril, Tucson, Arizona 85718

* JONES, Dr. Rembart F., Code 172.3, David W. Taylor Naval Ship Research and Development Center, Bethesda, Maryland 20084

* KAMINETZKY, Dov, Feld, Kaminetzky & Cohen, P. C., 60 East 42nd Street, New York, New York 10165

* KANCHANALAI, Dr. Tokul, 133 Sukumvit SOI 39, Sukumvit Street, Bangkok 11, Thailand

KAO, Carl S., Mobil Research & Development Corporation, P. O. Box 900, Dallas, Texas 75221

KARAMUK, Dr. Ergun, Basler & Hofmann, Forchstr. 395, CH-8029 Zurich, Switzerland

KARSAN, Demir I., Manager, Brown & Root, Inc., P. O. Box 3, Houston, Texas 77001

KATO, Prof. Ben, Faculty of Engineering, Department of Architecture, The University of Tokyo, 7-3-1, Hongo, Bunko-ku, Tokyo 113 Japan

* KEMP, Prof. Alan R., Department of Civil Engineering, University of Witwatersrand, 1 Jan Smuts Avenue, Johannesburg 2001, South Africa

* KETTER, Dr. Robert L., Leading Professor of Engineering and Applied Sciences, State University of New York at Buffalo, Engineering Building - West - Room 236, Buffalo, New York 14260

KHAN, Dr. Fazlur S. - Deceased 1982

KILLAM, Everett H., Manager - Product Planning, #5 Kenbell Plaza, 3075 Canal Road, Terre Haute, Indiana 47802

* KINRA, Ravindar K., Senior Civil Engineer, Shell Oil Company, TSP 1772, P. O. Box 2099, Houston, Texas 77001

* KIRVEN, Peyton E., American Institute of Architects, 1441 Benedict Canyon Drive, Beverly Hills, California 90210

* KITIPORNCHAI, Dr. Sritiwat, Department of Civil Engineering, University of Queensland, St. Lucia, Qld, Australia 4067

KLIPPSTEIN, Karl H., Applied Research Laboratory, MS66, U. S. Steel Corporation, Monroeville, Pennsylvania 15146

KOO, Prof. Benjamin, Department of Civil Engineering, The University of Toledo, Toledo, Ohio 43606

* KOUNADIS, Prof. Anthony N., Associate Professor of Structural Analyses and Steel Bridges, National Technical University, 42 - 28th October Avenue, Athens 147 Greece

* KOWALCZYK, Dr. Ryszard M., ul.Korotynskiego 19A, m.119, 02 123 Warsaw, Poland

* KRAJCINOVIC, Dr. Dusan, Department of Materials Engineering, Box 4338, University of Illinois at Chicago Circle, Chicago, Illinois 60680

KRENTZ, H. A., President, Canadian Institute of Steel Construction, 201 Consumer Road, Suite 300, Willowdale, Ontario, Canada M2J 4G8

* LA BOUBE, Roger A., Research Center, Butler Manjfacturing Company, 135th Street and Botts Road, Grandview, Missouri 64030

LAI, David C., Amoco Production Company, P. O. Box 591, Tulsa, Oklahoma 74102

* LALLY, Andrew, Steinman, Boynton, Gronquist & Birdsall, 50 Broad Street, New York, New York 10004

LAMM, L. P., Executive Director, Federal Highway Administration - HNG-30, Department of Transportation, Washington, D. C. 20590

* LARSEN, Prof. Per K., Division of Steel Structures, Norwegian Institute of Technology, 7034 Trondheim - NTH, Norway

LATHAM, Earl R., California Department of Public Works, P. O. Box 1499, Sacramento, California 95807

LAWRENCE, Henry, Director, Codes & Standards, The American Institute of Architects, 1735 New York Avenue, N. W., Washington, D. C. 20006

* LEE, Prof. George C., Dean of Engineering, State University of New York, 203 Fronczak Hall, Amherst, New York 14260

LEE, Griff C., Vice President & Group Executive, McDermott Incorporated, 1010 Common Street, P. O. Box 60035, New Orleans, Louisiana 70160

LEEDS, David J., Editor, EERI Newsletter, 11972 Chalon Road, Los Angeles, California 90049

* LE MESSURIER, William J., Le Messurier Associates/SCI, 1033 Massachusetts Avenue, Cambridge, Massachusetts 02138

* LEW, Dr. H. S., Building 226, Room A365, Building Research Division - IAT, National Bureau of Standards, Washington, D. C. 20234

LIBOVE, Prof. Charles, Department of Mechanical & Aerospace Engineering, 139 E. A. Link Hall, Syracuse University, Syracuse, New York 13210

* LICHTENSTEIN, Abba G., A. G. Lichtenstein & Associates Inc., Consulting Engineers, 17-10 Fair Lawn Avenue, Fair Lawn, New Jersey 07310

* LIN, Dr. Fung Jen, 1020 Arroyo Drive, South Pasadena, California 91030

* LIN, Samuel P., Dravo Van Houten, Inc., One Penn Plaza, New York, New York 10001

* LIN, Tung H., University of California, Mechanics & Structures Department, 5732 Boelter Hall, Los Angeles, California 90024

LIND, Prof. Nils C., Department of Civil Engineering, University of Waterloo, Ontario, N2L 3GI Canada

LLOYD, James R., Exxon Production Research Company, P. O. Box 2189, Houston, Texas 77001

* LOOMIS, Robert S., Loomis and Loomis Inc., Box 505, Windsor, Connecticut 06095

* LOOMIS, Richard W., Loomis and Loomis Inc., Box 505, Windsor, Connecticut 06095

* LU, Prof. Le Wu, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015

* LUNDGREN, Prof. Harry R., Civil Engineering Center, Arizona State University, Tempe, Arizona 85281

MacADAMS, J. N., Research & Technology, Armco Steel Corporation, Middletown, Ohio 45042

* MAISON, Jack R., Assistant Director, Department of Ocean Engineering and Structural Design Southwest Research Institute, P. O. Drawer 28510, San Antonio, Texas 78284

MANGELSDORF, Prof. Clark P., 949 Benedum Hall, University of Pittsburgh, Pittsburgh, Pennsylvania ,5261

* MANSELL, Dr. D. S., Civil Engineering Department, University of Melbourne, Parkville VIC 3052, Australia

MANSOUR, Alaa E., University of California, Department of Naval Architecture, Berkeley, California 94720

MARA, Paul V., Vice President - Technical, The Aluminum Association, Inc., 818 Connecticut Avenue, N. W., Washington, D. C. 20006

MAREK, Dr. Pavel, Czechoslovakia Technical University, SF CVUT ZIKOVA 4, Prague 6, Czechoslovakia

* MARIANI, Theodore F., American Institute of Architects, Mariani & Associates, 1600 20th Street, N. W., Washington, D. C. 20009

* MARSH, Prof ~~Dr~~. Cedric, Center for Building Studies, Concordia University, 1455 de Maisonneuve Boulevard West, Montreal, Quebec H3G 1M8 Canada

MARSH, James W., Western Regional Manager, American Institute of Steel Construction, Inc., 9440 Telstar Avenue, Suite 103, El Monte, California 91731

* MARSHALL, Peter W., Shell Oil Company, P. O. Box 2099, Houston, Texas 77001

* MARSHALL, Dr. Richard D., U. S. Department of Commerce, National Bureau of Standards, Washington, D. C. 20234

MARTINOVICH, W. M., Senior Vice President, Earl and Wright, One Market Plaza, San Francisco, California 94105

* MASSONNET, Prof. Charles E., Universite de Liege, Institut du Genie Civil, Quai Banning, 6, B-4000 Liege Belgium

MATEESCU, Prof. Dipl. Ing. Dan, Institutul Politechnic Timisoara, Str. Stadion No. 1, Timisoara, Romania

MATSUI, Prof. C., Department of Architecture, Faculty of Engineering, Kyushu University, Hakozaki, Fukuoka 812, Japan

* MATSUMURA, George M., Office of the Chief of Engineers, 20 Massachusetts Avenue, N.W., ASI-T, Room 3118, Washington, D.C. 20314

MATTOCK, Prof. A. H., University of Washington, Department of Civil Engineering, Seattle, Washington 98105

* MC BEAN, Dr. Robert P., Black & Veach Consulting Engineers, P. O. Box 8405 Kansas City, Missouri 64114

* MC CORMACK, Charles, Blauvelt Engineering Company, One Park Avenue, New York, New York 10016

* MC CUMMINGS, James M., Greiner Engineering Sciences, Inc. One Village Square, Village of Cress Keys, Baltimore, Maryland 21210

MC FALLS, R. K., Bell Telephone Labs, Room 4, 204 North Road, Chester, New Jersey 07930

* MC LAUGHLIN, J. M., Manager, Structural Department, Sargent & Lundy Engineers, 55 East Monroe Street, Chicago, Illinois 60603

* MC NAMEE, Prof. Bernard M., Drexel University, 32nd & Chestnut Streets, Philadelphia, Pennsylvania 19104

* MEITH, Robert M., Offshore Technology & Planning Staff, Standard Oil Company of California, 3 Embarcadero Center, Room 1073, P. O. Box 3069, San Francisco, California 94119

* MELCHER, Dr. Jindrich J., Technical University of Brno, VUT-FAST, Barvicova 85, 662 37 Brno, Czechoslovakia

* MIKULAS, Dr. Martin M., Jr., Mail Stop 190, NASA Langley Research Center, Hampton, Virginia 23665

* MILEK, William A., Jr., American Institute of Steel Construction, 400 North Michigan Avenue, 8th Floor, Chicago, Illinois 60611-4185

MILLER, Clarence D., Director of Structural Research, Chicago Bridge & Iron Company, Route 59, Plainfield, Illinois 60544

MINHAS, Khalid M., DMT-30, Room 8150, 400 - 7th Street, S. W., Department of Transportation, Washington, D. C. 20590

MIRZA, M. Saeed, Executive Director, The Canadian Society for Civil Engineering, 2050 Mansfield, Suite 700, Montreal, Quebec H3A 1Z2 Canada

* MOLONEY, Edward J., Vollmer Associates, Inc., 62 Fifth Avenue, New York, New York 10011

MOOLANI, Dr. Firoz, Ministry of Transportation and Communications of Canada, 1201 Wilson Avenue, West Building, Downsview, Ontario, Canada

MORGENSTERN, Brian, Buckland & Taylor, 1591 Bowser Avenue, North Vancouver, B. C., Canada V7P 2Y4

* MORRELL, Dr. M. L., Department of Civil Engineering, Clemson University, Clemson, South Carolina 29631

MORRIS, Prof. Nicholas F., Civil Engineering Department, Manhattan College, Riverdale, New York 10471

* MORRISSEY, Charles D., Basil Engineering Corporation, 41 East 42nd Street, New York, New York 10017

* MURRAY, Prof. Thomas M., University of Oklahoma, School of Civil Engineering, 202 West Boyd Street, Norman, Oklahoma 73019

MURRAY, W. W., Associate Technical Director for Structures, Naval Ship Research & Development Center, Bethesda, Maryland 20084

* NAIR, R. Shankar, Alfred Benesch & Company, 233 North Michigan Avenue, Chicago, Illinois 60601

* NAPIER, Claude, Federal Highway Administration, HNG-33, 400 - 7th Street, N. W., Washington, D. C. 20590

NAPPER, L. A., Engineering Department, Bethlehem Steel Corportation, Bethlehem, Pennsylvania 18016

NELSON, Dr. Burke E., Executive Director, American Society of Mechanical Engineers, 345 East 47th Street, New York, New York 10017

NETHERCOT, Dr. David A., Department of Civil & Structural Engineering, The University, Mappin Street, Sheffield, S1 3JD, United Kingdom

NEWSLETTER, Editor, National Research Council of Canada, Ottawa K1A OS2, Canada

* O'CONNOR, Prof. Colin, Department of Civil Engineering, University of Queensland, St. Lucia, Qld., Australia 4067

* OJALVO, Prof. Morris, Department of Civil Engineering, 470 Hitchcock Hall, The Ohio State University, Columbus, Ohio 43210

* OLIVO, Peter A., Vice President, Edwards and Kelcey, Inc., 70 South Orange Avenue, Livingston, New Jersey 07039

* OSTAPENKO, Prof. Alexis, Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania 18015

* OWEN, Dr. Norman, URS/John A. Blume & Associates, Engineer, 130 Jessie Street, San Francisco, California 94105

OYLER, Dr. Glenn W., Executive Director, Welding Research Council, 345 East 47th Street, New York, New York 10017

* PALMER, Frederick J., Manager-Engineering, Copperweld Tubing Group, Two Robinson Plaza, Route 60, Box 60, Pittsburgh, Pennsylvania 15230

PARMER, John F., Executive Director, Structural Engineers Association of Illinois, 55 East Washington Street, Room 1401, Chicago, Illinois 60602

* PEKOZ, Prof. Teoman, School of Civil & Environmental Engineering, Cornell University, Ithaca, New York 14850

* PILLAI, Prof. S. Unnikrishna, Professor of Civil Engineering, Regional Engineering College, Calicut, Kerala, India 673601

* PINKHAM, C. W., S. B. Barnes & Associates, 2236 Beverly Boulevard, Los Angeles, California 90057

* PISETZNER, Emanuel, Weiskopf & Pickworth, Consulting Engineers, 200 Park Avenue, New York, New York 10017

PLAUT, Prof. Raymond H., Department of Civil Engineering, Virginia Polytechnic Institute, Blacksburg, Virginia 24061

* POPOV, Prof. Egor P., University of California, 725 Davis Hall, Berkeley, California 94720

* PRICKETT, Joseph E., Modjeski and Masters, P. O. Box 2345, Harrisburg, Pennsylvania 17105

* PRITSKY, W. W., Technical Director, Engineering, The Aluminum Association, Inc., 818 Connecticut Avenue, N. W., Washington, D. C. 20006

RAY, Keith, Editor, Building Research, National Research Council, 2101 Constitution Avenue, N. W., Washington, D. C. 20418

* RAZZAQ, Dr. Zia, Department of Civil Engineering, University of Notre Dame, Notre Dame, Indiana 46556

* REGL, Dr. Robert, McDermott Inc., 1010 Common Street, P. O. Box 60035, New Orleans, Louisianna 70160

* RENTSCHLER, Dr. Gelnn P., Gilbert Associates, Inc., P. O. Box 1498, Reading, Pennsylvania 19603

RICKETTS, Capt. Myron V., Commander, David W. Taylor Naval Ship Research & Development Center, Bethesda, Maryland 20084

* RINGO, Dr. Boyd C., Civil & Environmental Engineering Department, 639 Baldwin #71, University of Cincinnati, Cincinnati, Ohio 45221

* ROBB, John O., 175 North Circle Drive, San Gabriel, California 91776

* ROBERTSON, Leslie E., Skilling, Helle, Christiansen, Robertson, P.C., 211 East 46th Street, New York, New York 10017

ROLF, Richard L., ALCOA Research Lab, P. O. Box 772, New Kensington, Pennsylvania 15068

ROMANESKI, Albert L., Executive Vice President, Sippican Consultants International, Inc., 1033 Massachusetts Avenue, Cambridge, Massachusetts 02138

* ROSS, Prof. David A., Civil Engineering Department, University of Akron, Akron, Ohio 44325

* RUST, William D., Jr., Professional Services Division, Office of Construction Management, Public Buildings Service, General Services Administration, Washington, D. C. 20405

* RUTENBERG, Prof. Avigdor V., Head of Structures, Department of Civil Engineering, TECHNION - Israel Institute of Technology, Technion City, Haifa 32000, Israel

SANDFORD, Paul G., Carruthers & Wallace Ltd., 34 Greensboro Drive, Rexdale, Ontario M9W 1E1 Canada

SCHEFFEY, Charles F., Director, Office of Research, Federal Highway Administration, Washington, D. C. 20590

SCHELLING, Dr. David R., Department of Civil Engineering, University of Maryland, College Park, Maryland 20742

SCHMIDT, William R., Earl & Wright, One Market Plaza, Spear Street Tower, San Francisco, California 94105

* SCHULTZ, Dr. Gerald W., Institute fur Baustatik, Universtitat Innsbruck, Technikerstrasse 13, A6020 Innsbruck, Austria

* SEARS, Frank D., Chief, Review Branch, Bridge Division - HNG-32, Federal Highway Administration, 400 Seventh Street, S. W., Washington, D. C. 20590

* SFINTESCO, Dr. Duiliu, 86 Avenue de Beaumont, 60260 Lamorlaye, France

* SHAH, Chandra R., President, Balke Engineers, 7762 Reading Road, Cincinnati, Ohio 45237

* SHARP, M. L., Section Head, Engineering Properties & Design Division, Alcoa Technical Center, Alcoa Center, Pennsylvania 15068

SHEINMAN, Dr. I., Department of Civil Engineering, TECHNION - Israel Institute of Technology, Technion City, Haifa 32000, Israel

* SHERMAN, Prof. Donald R., Department of Civil Engineering, P. O. Box 784, University of Wisconsin-Milwaukee, Milwaukee, Wisconsin 53201

SHILKRUT, Prof. Dov, Ben Gurion University of the Negev, Faculty of Engineering, P. O. Box 653, Beersheva 84120, Israel

* SHINA, Isaac S., Green International, Inc., 504 Beaver Street, Sewickley, Pennsylvania 15143

* SHROFF, Avanti C., Iffland Kavanagh Waterbury, 1501 Broadway, New York, New York 10036

SILVA, Prof. Raul Rosas, Department of Civil Engineering, Pontificia Universidade Catolica do Rio de Janeiro PUC-RJ, R. Marques de S. Vincente 225. Gavea, Rio de Janeiro, CEP 22453, Brazil

DA SILVA, Dr. Manual A. J. G., c/o Promon Engenharia S.A., Praia do Flamengo, 154 10° andar, 22210 Rio de Janeiro - RJ, Brazil

* SIMITSES, Prof. George J., School of Engineering Science and Mechanics, Georgia Institute of Technology, 225 North Avenue, N. W., Atlanta, Georgia 30332

SIMONIS, John C., Babcock & Wilcox, Power Engeration Group, P. O. Box 1260, Lynchburg, Virginia 24505

SINGER, Prof. Josef, President, TECHNION - Israel Institute of Technology, Technion City, Haifa 32000, Israel

SMITH, James R., Executive Director, Building Seismic Safety Council, 1015 15th Street, N.W., Suite 700, Washington, D. C. 20005

* SNELL, Virgil H., Black & Veach Consulting Engineers, P. O. Box 8405, Kansas City, Missouri 64114

SOMERS, Dr. Arnold E., Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Virginia 24061

* SOTO, M. H., Gannett Fleming Corddry and Carpenter, Incorporated, P. O. Box 1963, Harrisburg, Pennsylvania 17105

* SPENCER, Dr. Herbert H., 10-8M Landing Lane, New Brunswick, New Jersey 08901

* SPRANGUE, E. T. E., Managing Director, Steel Joist Institute, 1703 Parham Road, Suite 204, Richmond, Virginia 23229

* SPRINGFIELD, John, Vice President, Carruthers & Wallace Ltd., 34 Greensboro Drive, Resdale, Ontario M9W 1E1 Canada

SRIDHARAN, S., Department of Civil Engineering, Washington University, St. Louis, Missouri 63130

* STEIN, Dr. Manuel, MS-190, SDD-Analytical Methods Section, NASA Langley Research Center, Hampton, Virginia 23665

* STOCKINGER, Charles M., General Manager, Metal Building Manufacturers Association, 1230 Keith Building, Cleveland, Ohio 44115

STOCKWELL, Frank W., Jr., Northeastern Regional Manager, American Institute of Steel Construction, 225 West 34th Street, Suite 1413, New York, New York 10122

STRATING, Dr. J., Protech International BV, General Engineers & Consultants, Stationsplein 2, Schiedam, The Netherlands

* STRINGER, David C., AMCA International, Limited, P. O. Box 3246, Station C, Ottawa, Ontario K1Y 4J5 Canada

SULLIVAN, C. J., Director of Research, The American Society of Mechanical Engineering, 345 East 47th Street, New York, New York 10017

* SUPPLE, Dr. William J., J P Kenny & Partners, Limited, Burne House, 88-89 High Holborn, Lohdon WC1V6LS United Kingdom

* TALL, Dr. Lambert, Dean, School of Technology, Florida International University, Tamiami Campus, Miami, Florida 33199

* TANG, Dr. Man-Chung, Executive Vice President, DRC Consultants, 500 5th Avenue, New York, New York 10036

TARANATH, Bungale, Walter P. Moore & Associates, Inc., 2905 Sackett Street, Houston, Texas 77098

TATSA, Dr. E., Department of Civil Engineering, TECHNION - Israel Institute of Technology, Technion City, Haifa 32000, Israel

* TEMPLE, Prof. Murray C., Department of Civil Engineering, University of Windsor, Windsor, Ontario N9B 3P4 Canada

* TESTA, Prof. Rene B., Department of Civil Engineering and Engineering Mechanics, Columbia University, Seeley W. Mudd Building, New York, New York 10027

* THOMAIDES, Spiro. S., Room 1705, Martin Tower, Bethlehem Steel Corp., Bethlehem, Pennsylvania 18016

* THOMSEN, Dr. Kjeld, International Steel Consulting A/S, Øster Alle 31, DK-2100 Ø Copenhagen Denmark

* THURLIMANN, Prof. Bruno, Institute of Structural Engineering, ETH-Honggerberg, CH-8093 Zurich, Switzerland

* TOMASETTI, Richard L., Senior Vice President, Lev Zetlin Associates, Inc., 95 Madison Avenue, New York, New York 10016

* TRAHAIR, Prof. Nicholas S., University of Sydney, Civil Engineering, Sydney, N S W, Australia 2006

TROMBLEY, Kenneth, Editor, Professional Engineer, 2029 K. Street, N. W., Washington, D. C. 20006

TUNG, Prof. David H. H., School of Engineering, The Copper Union, Cooper Square, New York, New York 10003

TYSON, Christopher G., General Manager, Codes and Standards Activities, American Iron & Steel Institute, 1000 - 16th Street, N. W., Washington, D. C. 20036

* UBBEN, J. E., Assistant Production Director, American Petroleum Institute, 211 North Ervay, Suite 1700, Dallas, Texas 75201

* ULSTRUP, Carl C., Associate Engineer, Steinman, Boynton, Gronquist & Birdsall, 50 Broad Street, New York, New York 10004

* VAN DER WOUDE, Dr. Frank, University of Tasmania, Civil Engineering Department, Box 252 C GPO, Hobart, Tasmania 7001, Australia

* VANN, Dr. W. Pennington, Department of Civil Engineering, Box 4089, Texas Tech University, Lubbock, Texas 79409

* VARADARAJAN, Dr. R., Delon Hampton & Associates, 6001 Montrose Road, Suite 800, Rockville, Maryland 20852

* VARNEY, Robert F., Deputy Chief, Structures and Applied Mechanics Division, Office of Research - HRS-10, Federal Highway Administration, Washington, D. C. 20590

VIEST, Dr. Ivan M., Room 242 East Building, Bethlehem Steel Corporation, Bethlehem, Pennsylvania 18016

* VINNAKOTA, Prof. Sriramulu, Civil Engineering Department, Marquette University, 1515 West Wisconsin Avenue, Milwaukee, Wisconsin 53233

* VOGEL, Prof. U., Institut für Baustatik, Universität Karlsruhe, 75 Karlsruhe, Kaiserstr. 12, Federal Republic of Germany

WAKABAYASHI, Prof. Minoru, Disaster Prevention Research Institute, Kyoto University, Uji City, Kyoto Pref., Japan

WALDNER, H. Eugene, Associate, Modjeski and Masters, P. O. Box 2345, Harrisburg, Pennsylvania 17105

* WALKER, Prof. Alistair C., J P Kenny & Partners, Limited, Burne House, 88-89 High Holborn, London WC1V6LS United Kingdom

WALL, Donald R., Publisher, Building Design & Construction, 5 South Wabash Avenue, Chicago, Illinois 60603

* WANG, Dr. Chu-Kia, Department of Civil & Environmental Engineering, University of Wisconsin, Madison, Wisconsin 53706

* WANG, Shien T., Department of Civil Engineering, 210 Anderson Hall, University of Kentucky, Lexington, Kentucky 40506

* WATSON, Don R., Technical Director, International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, California 90601

* WELLER, Dr. T., Department of Aeronautical Engineering, Technion - Israel Institute of Technology, Technion City, Haifa 32000, Israel

WILLIS, James A., Blaylock-Willis and Associates, 1909 McKee Street, San Diego, California 92110

WILLSON, R. Thomas, Senior Vice President, American Iron & Steel Institute, 1000 - 16th Street, N. W., Washington, D. C. 20036

* WINGERTER, W. W., Manager, Public Affairs, Chevron U.S.A., Inc., 935 Gravier Street, New Orleans, Louisiana 70112

WINTER, Prof. George, Deceased - November 1982

* WOLCHUK, Roman, Weidlinger Associates, 110 East 59th Street, New York, New York 10022

WRIGHT, Dr. Douglas T., President, University of Waterloo, Waterloo, Ontario N2L 3G1 Canada

* WRIGHT, Dr. E. Whitman, 57 Sunnyside Avenue, Ottawa, Ontario K1S 0P9 Canada

WRIGHT, Dr. Richard N., Director, Center for Building Technology, Building 226, Room B260, National Bureau of Standards, Washington, D. C. 20234

* WYLIE, Frank B., Jr., Hazelet & Erdal, 405 Morrissey Building, 304 West Liberty Street, Louisville, Kentucky 40202

* YACHNIS, Dr. Michael, Chief Engineer, Code 04B, Naval Facilities Engineering Command, 200 Stovall Street, Alexandria, Virginia 22332

* YANG, T. H., Conoco Inc., P. O. Box 2197, Houston, Texas 77252

* YEN, Prof. Ben T., Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015

YIU, Chris S. C., E Lionel Pavlo, 114 Fifth Avenue, New York, New York 10011

* YOO, Prof. Chai Hong, Department of Civil Engineering, Auburn University, Auburn, Alabama 36849

* YOUNG, Robert C., Iffland Kavanagh Waterbury, 1501 Broadway, New York, New York 10036

YU, Dr. Ching K., URS/Madigan-Praeger Inc., 150 East 42nd Street, New York, New York 10017

* YU, Prof. Wei Wen, Department of Civil Engineering, University of Missouri, Rolla, Missouri 65401

* YURA, Dr. Joseph A., Department of Civil Engineering, University of Texas, Austin, Texas 78712

* ZANDONINI, Dr. Riccardo, Dip. Ingegneria Strutturale, Polite Cnico Di Milano, P.za Leonardo Da Vinci 32, 20133 Milano, Italy

* ZELLIN, Martin A., Sverdrup & Parcel and Associates, Inc., 801 North 11th Street, St. Louis, Missouri 63101

ZILS, John, Skidmore, Owings & Merrill, 33 West Monroe Street, Chicago, Illinois 60603

ZOBEL, RADM W. M., Commander, Naval Facilities Engineering Command, 200 Stovall Street, Alexandria, Virginia 22332

ZWOYER, Eugene, American Society of Civil Engineers, 345 East 47th Street, New York, New York 10017

By-Laws*

PURPOSES OF THE COUNCIL

The general purposes of the Structural Stability Research Council shall be:

1. To maintain a forum where the structural stability aspects of metal and composite metal and concrete structures and their components can be presented for evaluation, and pertinent structural research problems proposed for investigation.

2. To review the world's literature on structural stability of metal and composite metal and concrete structures and study the properties of materials available for their construction, and to make the results widely available to the engineering profession.

3. To organize, administer and guide cooperative research projects in the field of structural stability, and to solicit financial support for such projects.

4. To promote publication and dissemination of research information in the field of structural stability.

5. To study the application of the results of research to stability design of metal and composite metal and concrete structures, and to develop comprehensive and consistant strength and performance criteria and encourage consideration thereof by specification-writing bodies.

---

*Revised: August 21, 1947; October 1, 1948; November 1, 1949; August 15, 1951; May 20, 1955; October 1, 1960; May 7, 1962; May 21, 1965; May 31, 1968; March 27, 1974; May 7, 1975; November 15, 1976; April 30, 1980; and November 15, 1981.

COUNCIL MEMBERSHIP

The voting membership of the Council shall consist of Representatives of Sponsors, Representatives of Participating Organizations, Representatives of Participating Firms, Members-at-Large, Corresponding Members and Life Members.

Representatives are appointed by the Sponsor, the Participating Organization, or the Participating Firm subject to the approval of the Executive Committee, and continue to serve until replaced. A Sponsor may appoint up to five representatives, a Participating Organization may appoint up to three representatives, and a Participating Firm may appoint up to two representatives. Organizations concerned with investigation and design of metal and composite structures may be invited by the Council to become Sponsors, Participating Organizations, or Participating Firms, as appropriate.

An individual who has expressed interest in the work of the Council, and who is presently or has been involved in work germane to its interest, may be elected a Member-at-Large by the Council following nomination by the Executive Committee.

Corresponding Members are appointed by the Executive Committee to maintain contact with organization in other counctries that are active in areas of interest to the Council.

Council members of appropriate age and service may be elected Life Members by the Council following nomination by the Executive Committee.

Every three years the Secretary of the Council shall canvas the Sponsors, Participating Organizations and Participating Firms to determine their Representatives for the next three-year period.

Every three years the Secretary of the Council shall contact each Member-at-Large and each Corresponding Member to determine whether he wishes to continue his membership.

FEES

The minimum yearly fee for Sponsors, Participating Organizations and Participating Firms shall be determined by the Executive Committee.

Any Participating Organization whose Bylaws specifically prohibit payment of such a fee shall be exempted therefrom upon its request and following approval by the Executive Committee.

The fee for Members-at-Large shall be determined by the Executive Committee and shall be for a three-year period, billed concurrently with the regular triennial membership review.

Representatives, Corresponding Members and Life Members are exempted from the payment of fees, but may contribute on a voluntary basis.

COUNCIL MEETINGS

The Council shall hold at least one regular meeting in each fiscal year, and such additional meetings as deemed necessary by the Executive Committee. A meeting quorum consists of twenty Council members.

COUNCIL DUTIES

1. To establish policies and rules, and approve changes in the Bylaws.
2. To review and approve the annual budget.
3. To elect Council officers, members of the Executive Committee, Members-at-Large and Life Members.
4. To approve the appointment of salaried officers of the Council.
5. To encourage interest in and support of the work of the Council and to assist in publicizing its activities and findings.

COUNCIL OFFICERS AND STAFF

The officers of the Council shall be a Chairman and a Vice Chairman. The Chairman shall exercise general supervision over the technical and business affairs of the Council, subject to the direction of the Council, and shall perform all duties incidental to his office; and he shall be Chairman of the Executive Committee. The Chairman shall preside at meetings of the Council and of the Executive Committee. He shall be ex-officio a member of all Council committees and task groups. In the absence of the Chairman his duties shall be performed by the Vice Chairman.

The terms of office of the Chairman and Vice Chairman shall be three years and shall begin on October 1 of the year of election. They shall be eligible for immediate re-election for one term of one year. In the event of an unanticipated vacancy in the office of Chairman or Vice Chairman, a successor shall be appointed by the Executive Committee to serve for the remainder of the term.

A Director may be engaged by the Executive Committee, subject to the approval of the Council, to serve as the chief administrative officer of the Council. The Director shall be an ex-officio member of the Council and of the Executive Committee. Additional officers may be engaged by the Executive Committee as necessary, subject to the approval of the Council. The Director may engage appropriate staff and shall supervise their work. The salaries of all such officers and staff members shall be determined by the Executive Committee.

Working under the general direction of the Chairman and the Executive Committee, the Director shall conduct the regular business of the Council. He shall administer the financial affairs of the Council in accordance with an approved budget and good business practices, and shall prepare and execute all contracts authorized by the Executive Committee. The Director shall exert every effort to secure economy in the business administration of the Council.

COUNCIL EXECUTIVE COMMITTEE

The Executive Committee shall consist of the Chairman of the Council, the Vice Chairman, the Director, the most recent Past Chairman and Past Vice Chairman, and nine additional members elected by the Council from its membership. For the nine elected members the term of office shall be three years, with three members elected each year. Members whose terms are expiring shall be eligible for immediate re-election. Members shall take office immediately upon their election.

An unanticipated vacancy shall be filled by appointment by the Chairman from the membership of the Council, and the appointee shall serve for the remainder of the term.

The Executive Committee shall determine and implement policies and programs to support and advance the general purposes of the Council, and shall exercise general direction and supervision over the technical and business affairs of the Council. The specific duties and responsibilities of the Executive Committee shall include the following:

(a) Review and approve proposed research projects and contracts.

(b) Coordinate and give general supervision to research projects and contracts.

(c) Appoint a Committee on Finance, a Committee on "Guide to Stability Design Criteria for Metal Structures", a Committee on Technical Session Programs, and such other committees as may be deemed necessary from time to time.

(d) Set up task groups and appoint chairmen thereof, and approve nominees for membership therein; and appoint task reporters.

(e) Review, approve and disseminate reports and manuscripts.

(f) Sponsor and implement the preparation of successive editions of the "Guide" amd appoint the Editor thereof.

(g) Respond appropriately to inquiries relating to stability design criteria. Such inquiries may be referred to the appropriate task groups for evaluation and response.

(h) Exercise general supervision over preparation of the program for the Annual Technical Session and Meeting of the Council.

(i) Direct the financial and business management of the Council and assist the Committee on Finance in preparation of the annual budget.

From time to time the Executive Committee may ask consultants particularly interested in specific projects to serve in an advisory capacity with respect thereto.

Meetings of the Executive Committee shall be held in the spring and in the fall. Additional meetings may be held at the call of the Chairman, or at the written request of two of the Executive Committee or ten members of the Council. An Executive Committee quorum shall consist of seven members.

The minutes of the Executive Committee shall be transmitted promptly to all task group chairmen and furnished on request to any member of the Council. If no objection is made by any member within a reasonable period after the minutes have been issued, it shall be considered that the Council has no objection to the recorded actions of the Executive Committee. However, if objection to any Executive Committee action is entered by three or more Council members, then the action in question shall be submitted to the Council for vote, either at a special meeting called for that purpose or by letter ballot.

ELECTIONS

Each year at its fall meeting the Executive Committee shall appoint three members of the Council to serve as the Nominating Committee, with one of the three named as chairman thereof. Members of the Executive Committee or of the previous year's Nominating Committee shall not be eligible to serve.

The Nominating Committee shall prepare a slate of candidates for Chairman and Vice Chairman of the Council and for the Executive Committee to fill the anticipated vacancies, and shall transmit this slate to the Chairman of the Council by January 15.

The election of the Chairman and Vice Chairman of the Council and of members of the Exectuive Committee shall be by letter ballot. The results of the balloting shall be reported at the regular Annual Meeting of the Council. To be elected Chairman or Vice Chairman a candidate must receive a majority of the votes cast. In the event no candidate for Chairman or Vice Chairman receives such a majority, a run-off election between the two candidates receiving the largest number of votes shall be conducted.

STANDING AND SPECIAL COMMITTEES

Standing committees shall be a Committee on Finance, a Committee on the "Guide", and a Committee on Technical Session Programs. There shall be in addition such special committees as may be approved by the Executive Committee.

The Committee on Finance shall prepare the annual budget and solicit financial support for the work of the Council. The Chairman and the Vice Chairman of this committee shall be selected from the membership of the Executive Committee.

The Committee on the "Guide" shall direct the preparation and publication of successive editions of the "Guide".

The Committee on Technical Session Programs shall receive and review recommendations by task group chairmen and task reporters for Annual Technical Session papers and presentations, and determine the content of and guidelines for the Annual Technical Session program.

Chairmen and members of standing and special committees shall be appointed by and responsible to the Executive Committee, shall serve for three years, and shall be eligible for immediate reappointment.

TASK GROUPS

The Executive Committee may establish task groups, each for the study of a specific subject. The membership of each task group shall be only as large as needed for the work at hand. Task group members need not be members of the Council.

Task group chairmen shall be appointed by and responsible to the Executive Committee, shall serve for three years, and shall be eligible for immediate reappointment.

Prior to the Annual Meeting each task group chairman for the ensuing year shall review the task group membership with the objective of providing the most effective organization, and submit membership recommendations to the Executive Committee for approval.

The duties of a task group with respect to its designated area of responsibility shall include the following:

(a) Make recommendations for needed research.

(b) Review proposed research projects and render opinions as to their feasibility and suitability as Council projects.

(c) Furnish advice and guidance in connection with research projects, and suggest improvements in details of research programs within budgetary limitations.

(d) Make recommendations as to termination of projects.

(e) Prepare summary reports covering results of ongoing research projects, and final reports on completed projects.

(f) Prepare state-of-the-art reports summarizing existing knowledge, procedures and practices.

(g) Prepare material for the "Guide", as requested by the "Guide" Committee or the "Guide" Editor.

Each project handled by a task group shall be of definitive objective and scope.

Task groups shall be responsible to the Executive Committee for organizing and carrying out their projects, which shall be approved by the Executive Committee.

Each task group shall meet at least once in each fiscal year to review progress and plan activities for the ensuing year.

The Chairman of each task group shall submit an annual report to the Executive Committee at such other times as requested or as he deems necessary.

CONTRACTS AND AGREEMENTS

The Executive Committee may, within its budget, enter into contracts and agreements to implement the work of the Council. Contracts for research projects shall preferably be for a fiscal-year period. At the end of such a period the contract may be renewed or extended the next fiscal year.

Employment agreements with the Director and other salaried Council officers and staff may be for extended periods.

FISCAL YEAR

The fiscal year shall begin on October 1.

REVISION OF BYLAWS

These Bylaws may be revised by a majority vote of the entire membership of the Council conducted by letter ballot.

Rules of Procedure*

A. OUTLINE OF ROUTE OF A RESEARCH PROJECT FOR CONSIDERATION BY THE STRUCTURAL STABILITY RESEARCH COUNCIL

Projects are to be considered under three classifications:

Class (1) -- Projects originating within the Structural Stability Research Council.

Class (2) -- Those originating outside the Structural Stability Research Council or resulting from work at some institution and pertaining to general program of study approved by the Structural Stability Research Council.

Class (3)-- Extensions of existing SSRC sponsored projects.

Projects under Class (1) are to be handled as follows:

1. Project proposed.

2. Referred to Executive Committee for study and report to Council with recommendation.

3. If considered favorably by Council, the Executive Committee will take necessary action to set up the project.

4. Project Committee, new or existing, sets up project ready for proposals and refers back to Executive Committee for action.

5. Executive Committee sends out project for proposals.

6. Project Committee selects and recommends successful proposal to Executive Committee for action.

7. If awarded, the Project Committee supervises the project.

8. Project Chairman is to obtain adequate interim reports on project from laboratory.

9. Project Chairman advises Executive Committee adequately in advance of Annual Meeting as to report material available for Council presentation.

10. Executive Committee formulates program for presentation of reports at Annual Meeting.

11. Project Committee submits reports on any completed phase of the work for the Executive Committee.

12. Executive Committee determines disposition of report subject to approval of the Council before publication.

* Revised: Sep 22, 1975; May 16, 1977; Oct 22, 1981

Projects under Class (2) would be handled essentially the same except that steps 4, 5, and 6 would be omitted at the discretion of the Executive Committee. The procedure for items 7 - 12 would then be unchanged from that used for Class (1) projects.

With regard to Class (3) projects, an extension of an existing project which requires no additional funds or changes in supervisory personnel shall be approved by a majority of the Executive Committee, but need not be reported to the Council for its consideration or action. If an extension requires additional funds, such extensions may be approved by the Executive Committee subject to approval by a letter ballot from the Council.

B. OUTLINE OF A PATH OF A PROJECT THROUGH THE COUNCIL (FOR RECOMMENDED PRACTICE)

Task Group submits its findings to the Executive Committee.

Executive Committee acts and forwards to Recommended Practice Committee.

Recommended Practice Committee acts and forwards recommendations to Executive Committee.

Council votes on the matter.

Executive Committee transmits recommendations and findings to specification-writing bodies, and/or Publications Committee arranges for publication.

C. DISTRIBUTION AND PUBLICATION OF REPORTS

For the guidance of project directors and task group chairmen, the following policy is recommended with regard to the distribution of technical progress reports and with respect to the publication of reports. The scope of this procedure is intended to cover those reports that result from projects supported financially by the Structural Stabilty Research Council.

1. Distribution of Technical Progress Reports

Any duplicated report prepared by an investigator carrying out a research program may be distributed to the appropriate task group and to members of the Executive Committee with the understanding that the investigator may take further limited distribution with a view of obtaining technical advice. General distribution will only be made after approval by the task group.

2. Publication of Reports

Published reports fall into two categories and are to be processed as indicated:

a. Reports Constituted as Recommendations of the Council

(1) The report shall be submitted to the Executive Committee which after approval will circulate copies to members of the Structural Stability Research Council.

(2) Subject to approval of the Structural Stability Research Council, the Publications Committee takes steps to publish Council recommendations.

b. Technical Reports Resulting from Research Programs

(1) Universities or other organizations carrying out programs of research for the Structural Stability Research Council should make their own arrangements for publication of results.

(2) Assuming that the investigator wishes to arrange for such publication, approval must be obtained from the appropriate task group.

(3) Reprints are currently used as means of distributing reports of projects sponsored by or of interest to the Council. Investigator should order sufficient reprints for distribution by the Council. It is assumed that ear-marked project funds will be adequate for this purpose.

(4) When appropriate, reprints should be distributed under a distinctive cover.

(5) A statement of sponsorship should be included in all reports.

D. SSRC LIFE MEMBERS

1. Reason for Life Member Category - To facilitate continued participation in and contributions to the work of the SSRC on the part of active Council members who:

a. Have given exceptionally long service to SSRC or

b. Have given long service to SSRC and are on a reduced schedule of regular professional activity.

2. Guidelines for Nomination to Life Member Category

a. Candidate has given approximately 25 years of active service to SSRC, or approximately 15 years of active service and is not engaged full-time in regular employment; and

b. Has made significant contributions to the work of SSRC: and

C. Expects to continue active participation in the work of SSRC.

3. Nominating Procedure

a. SSRC Chairman will appoint Life Member Nominating Committee in the fall of each year, this committee to consist of two members of the Executive Committee (one of whom will be designated chairman) and the SSRC Secretary.

b. This committee will submit recommendations for Life Member nominees to the Executive Committee at its spring meeting.

c. Approved candidates will become Executive Committee nominees.

4. Election Procedure

The names of the Executive Committee nominees will be presented to the Council at its Annual Meeting, for election to Life Membership.

E. WORKING RELATIONSHIP BETWEEN EXECUTIVE COMMITTEE & TASK GROUPS

1. Executive Committee defines scope of task group assignment, selects task group chairman, and appoints Executive Committee contact member. SSRC Chairman sends letter of appointment to task group chairman and furnishes him with Statement of Scope, name of contact member, and procedural guidelines as appropriate.

2. Task group chairman can recommend changes to scope if he so desires.

3. Executive Committee recommends possible task group members, but task group chairman assembles his own list of prospects and determines their willingness to serve, and furnishes names to contact member.

4. Executive Committee approves task group members and SSRC chairman notifies them of their appointment.

5. Task group should meet at least once a year to remain in good standing. SSRC Chairman shall make this point clear to task group chairman when he is appointed.

6. Suitably in advance of Annual Technical Session, SSRC Secretary shall send instructions to each task group chairman regarding expected participation of his task group.

7. Suitably in advance of each Executive Committee meeting, SSRC Secretary shall send Executive Committee agenda (and relevant EC meeting minutes as necessary) to each task group chairman, requesting him to send one-page report to his contact member covering the following matters (and others as appropriate):

a. Task group progress.

b. Status of research projects being supervised or advised by task group.

c. Task group meeting minutes.

d. Comments on relevant matters on EC agenda.

e. Membership status and recommended changes.

f. (Prior to spring meeting of Executive Committee) Task group plans for SSRC Annual Technical Session.

8. It is contact member's responsibility to check regularly with task group chairman regarding task group progress, and particularly with respect to his duties and plans in connection with: (a) holding of task group meetings; (b) reports to Executive Committee; and (c) planning for and participation in Annual Technical Session.

9. In the event task group chairman will not be present at Executive Committee meeting or at Annual Technical Session, contact member will present task group report, or (if he is unable to attend) he shall arrange for an alternate to report, consulting in advance with SSRC Chairman or Secretary as appropriate.

10. In general, SSRC Chairman commissions and furnishes all necessary instructions to task group, and contact member renders follow-up services. Thus, task group chairman is ultimately responsible to Executive Committee, not to contact member.

F. GUIDELINES FOR SSRC TASK GROUP CHAIRMEN

1. Scope of Task Group Activities

Review the scope as approved by the Executive Committee and recommend changes if needed.

2. Task Group Membership

a. At the time the task group is formed, recommend task group membership to the Executive Committee. Task Group members will be approved by the Executive Committee and notified by the SSRC Chairman.

b. Review the task group membership at least once each year (before the annual meeting) and recommend new members or changes in the membership to the Executive Committee.

c. Endeavor to insure that members are active participants in the task group activities.

3. Conduct of Business

a. Direct the activities of the task group in the work required to carry out the assignment defined in the task group scope.

b. Carry out other tasks as may be assigned by the Executive Committee.

c. Hold a meeting of the task group at least once each year.

4. Investigator

An investigator is the project director for a research project under the advisory guidance of an SSRC task group. Such a person could either be directing a project sponsored financially by SSRC, or directing a program in the area of interest of the task group and for which both the task group and the investigator have agreed that such advisory guidance is desired and appropriate. Investigators are normally given priority when funds are available for travel to SSRC meetings.

5. Advisory Guidance

Advisory guidance is the activity that the committee carries out in providing suggestions to an investigator. Where financial support has been provided by SSRC (seed money for example), the Executive Committee normally would assign a task group to monitor the project. The task group members will provide the results of their experience to help an investigator. At the same time, the investigator will inform the task group of the most recent work so that the task group can get on with its other activities.

6. Reporting of Task Group Activities

Submit a written report of task group activities to the Executive Committee before each Executive Committee meeting. The deadline for the reports will be indicated to the task group chairman by correspondence from the SSRC secretary. Annual reports should cover:

a. Task group meetings.

b. Statement of purpose.

c. Task group membership.

d. Identify investigators (Roster C2).

e. Budget requests.

f. Projects receiving Task Group advisory guidance.

g. Research underway.

h. Needed research.

i. Guide activity.

j. Future Task Group plans.

k. Recommendations to Executive Committee for consideration and action.