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STRUCTURAL STABILITY RESEARCH COUNCIL

Established in 1944 by Engineering Foundation

Proceedings 1981

About the Council

The Structural Stability Research Council (formerly Column Research Council) was founded in 1944 to review and resolve the conflicting opinions and practices that existed at that time with respect to solutions to stability problems, and to facilitate and promote economical and safe design.

The Council offers guidance to specification writers and practicing engineers by developing both simplified and refined calculation procedures for the solution of stability problems, and assessing the limitations of these procedures.

The membership of the Council is made up of representatives from organizations (both governmental and private) concerned with specifications and design procedures for metal structures, representatives of consulting firms engaged in engineering practice, members-at-large selected from universities and design offices, and corresponding members from various countries who are in touch with stability research in their region.

The Council provides support and technical counsel for stability research, holds regular meetings to report on research activities, and publishes the definitive work "Guide to Stability Design Criteria for Metal Structures".

The Annual Technical Session not only provides up-to-date information on specific topics, but it also indicates where deficiencies exist in our present understanding of structural behavior. These Proceedings form a permanent record of the Council's activities and represent a primary source of the highlights of the latest solutions to structural problems before they are eventually published in technical journals.

STRUCTURAL STABILITY RESEARCH COUNCIL

Established in 1944 by Engineering Foundation

Proceedings 1981

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

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Foreword

Recently, I developed data and prepared analyses for informational purposes for our Sponsors and Participating Organizations to show the substantial returns that result from contributions and financial support of the SSRC. I was impressed with the results of this study and I want to share the following conclusions with the membership:

1. The volunteer contributions of Council members to SSRC work is estimated at $285,000 per year. Put another way, this means that for every one dollar given to support SSRC, almost 8 dollars is donated by voluntary contributions of time and out-of-pocket expenses by Council members and their supporting organizations. If SSRC were to go out of business, almost $300,000 per year of volunteer research support would be lost.

2. SSRC "seed money" for research generates $23,000 of research money from other sources for every $1,000 contributed by SSRC.

3. The cost of running the Council has increased four fold in the last 15 years.

Because costs have been increasing substantially each year, mainly due to inflation, a significant change has resulted in the make-up of organizations providing our financial support. We now have over 50 private firms and businesses contributing each year to support the Council. We also have received substantial support from outside organizations for our Annual Technical Session and Meeting - UMTA and FHWA for 1980 and seven firms in the offshore oil industry for 1982. I think it is appropriate that we not only thank this new group of supporters as well as our traditional supporters but to also realize that we have additional responsibilities. We now serve a wider market and a greater segment of the industry.

My goal for the coming year is two-fold. First, I want to expand our support from consultants and private businesses. This hopefully will result in an expanded research budget for SSRC. Where we are spending two to four thousand dollars a year, we should be spending ten to twenty. Secondly, I would like to direct some of our research efforts towards serving these new supporters. Determine what these needs are and see if SSRC can help them.

This year we had a successful Annual Technical Session and Meeting, concluding with a panel session arranged and chaired by George Winter on the subject of "Stability of Tall Buildings", thanks to the sponsorship of the National Science Foundation. The Council also cooperated in Council related sessions in both the Spring and Fall Annual Conferences in 1981.

This is my last year as your Chairman. May I close this Foreword with thanking all of you for your help and cooperation during my term of office. Special thanks should be given to the Headquarters Staff, our Director Lynn S. Beedle, our Administrative Secretary Lesleigh G. Federinic, and our Technical Secretaries Zu-Yan Shen and Gulay Askar.

Jerome S. B. Iffland

Jerome S. B. Iffland, Chairman
Structural Stability Research Council
New York, New York
1981

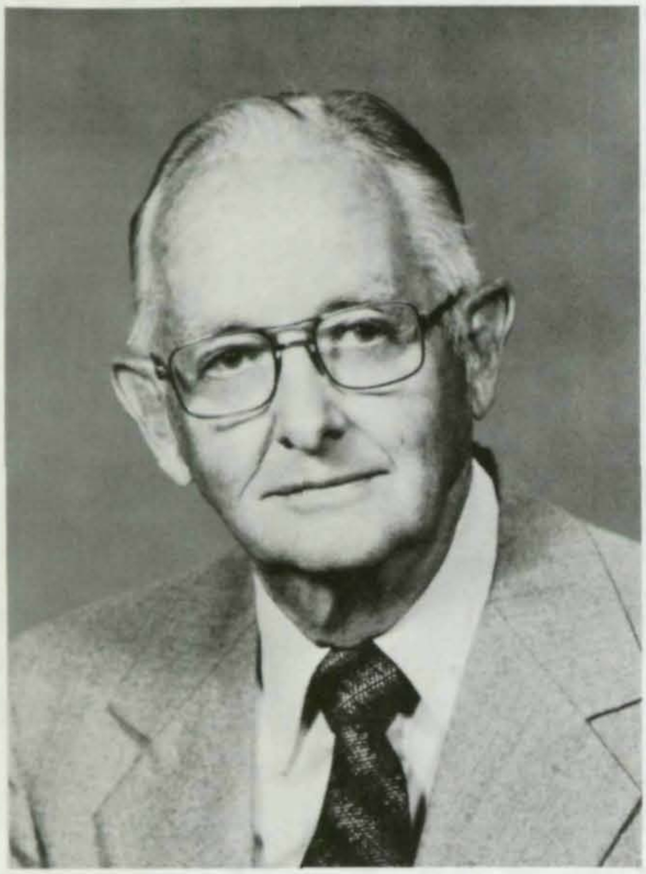
BRUCE G. JOHNSTON RECOGNITION SESSIONS

On May 12, 1981, in New York City, Bruce G. Johnston, a founding member of the Council, was honored at two special sessions during the ASCE International Conference and later at a banquet attended by many of his friends and associates.

In celebration of Bruce's 75th birthday, a special book was published containing 24 selected papers chosen from the more than 70 books and papers of which he was either sole author, coauthor, editor, or compiler. (It is available from SSRC headquarters.)

The SSRC is proud to salute Bruce Johnston in his 75th year. He has been a part of the Council for its entire 37 years of existance. The dedication, vision, persistance, and hard work that were required during the formation of the Council are still part and parcel of his devotion to its work. Such tireless efforts have served to inspire countless others.

Those who have had the privilege of serving with him on technical committees know of his abilities in Council work. Besides the obvious benefits of his vast reservoir of experience and knowledge, he has the ability to see the larger implications that others might overlook.



A generous sprinkling of delightful humor has served us well. Bruce's love of the game of golf led him to submit the following poem, subsequently published in the june 1981 GOLF DIGEST:

"The Numbers Game

When Ruth requests some household chore,
My age bears down at seventy-four
But when I stand out on the tee
I suddenly find I'm forty-three."

To us, Bruce is forever young, as is the lady who threw in her lot with him 42 years ago - Ruth Johnston.

For Bruce, Ruth and family - our love, highest esteem and continued best wishes.

SSRC Executive Committee

J. S. B. Iffland, Chairman	-	Iffland Kavanagh Waterbury
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T. V. Galambos	-	University of Minnesota
R. R. Graham	-	United States Steel Corporation
B. G. Johnston	-	Consultant
R. M. Meith	-	Chevron U.S.A. Inc.
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J. Springfield	-	Carruthers and Wallace Limited
G. Winter	-	Cornell University

BACK ROW: G. F. Fox, J. Springfield, R. M. Meith, J. L. Durkee, W. J. Austin, J. S. B. Iffland

FRONT ROW: Z. Y. Shen, R. R. Graham, S. J. Errera, B. G. Johnston, G. Askar

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 1981 Annual Technical Session was held on April 7 and 8 at the Conrad Hilton Hotel, Chicago. Eighty-eight persons attended the Session and twenty-six papers were delivered.

A panel discussion on "Stability of Tall Buildings" was held in the evening of April 7, 1981. The panelists were R. Shankar Nair, Fazlur R. Khan, and Jerry G. Stockbridge. The moderator was George Winter.

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

Summaries of the technical papers, the panel discussion and minutes of the business meeting are recorded in the following pages. The attendance list is also included.

PROGRAM OF TECHNICAL SESSION

Tuesday, April 7, 1981

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

SSRC Guide Reports

Chairman: B. G. Johnston, Consultant
Editor: T. V. Galambos, Washington University

. Overview of Current Stability Problems

T. V. Galambos, Washington University

9:50-10:10 a.m. - BREAK

. Finite Element Analysis of Stability Problems

R. H. Gallagher, University of Arizona

. Box Girders

D. H. H. Tung, The Cooper Union

. Open Discussion

Task Group 13 - Thin-Walled Metal Construction

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

. Effect of Insulation on Cold-Formed Z-Purlin Strength

T. M. Murray, University of Oklahoma

. Finite Strip Analysis of Postbuckling Behaviour of Plate Structures - Some Recent Results

S. Sridharan and T. V. Galambos, Washington University

12:10 p.m. - GROUP LUNCHEON

PROGRAM

1:10 p.m. - AFTERNOON SESSION

Presiding: J. L Durkee, Consulting Structural Engineer

SSRC Supported Research Project Reports

. Effect of End Restraints on the Stability of Geometrically Imperfect Columns as Parts of a Plane Frame

Z. Razzaq, University of Notre Dame

. Criteria, Analysis, and Design of Braced and Unbraced Frames

M. Biswas and R. Earwood, Texas A & M University

. Influence of Imperfections on the Maximum Strength of Restrained Beam Columns

S. Vinnakota, University of Wisconsin - Milwaukee

. The Effect of the Material Damage on the Buckling of Structures

D. Krajcinovic, University of Illinois at Chicago Circle

Task Group 1 - Centrally Loaded Columns

Chairman: R. Bjorhovde, The University of Alberta

. Local Buckling in Cyclically Loaded Built-Up Struts

E. P. Popov, University of California, Berkeley

Task Group 3 - Columns with Biaxial Bending

Chairman: J. Springfield, Carruthers and Wallace Limited

. Comparison of Test Results with Design Equations for Biaxially Loaded Steel Beam-Columns

S. U. Pillai, Royal Military College of Canada

Task Group 23 - Effect of End Restraint on Initially Crooked Columns

Chairman: W. F. Chen, Purdue University

. Effect of Small End Restraint on Strength of H-Columns

H. Sugimoto and W. F. Chen, Purdue University

. Initially Crooked, End Restrained Columns and Beam-Columns

Z. Y. Shen, Tong-Ji University, Shanghai
L. W. Lu, Lehigh University

PROGRAM

3:10-3:30 p.m. - BREAK

Task Group 4 - Frame Stability and Columns as Frame Members

Chairman: J. S. B. Iffland, Iffland Kavanagh Waterbury

. Buckling of Three-Story, Single-Span Rigid Frames

R. L. Ketter, State University of New York at Buffalo

. Stability of Tall Structures Restrained by Braces with Non-Linear Stiffness

L. F. Estenssoro, Wiss,Janney, Elstner and Associates, Inc.
A. J. Gouwens, Goodell-Grivas, Inc.

Task Group 7 - Tapered Members

Chairman: A. Amirikian, Amirikian Engineering Co.

. Inelastic Torsional Properties of Tapered Steel Channels

G. C. Lee and K. C. Chang, State University of New York at Buffalo

4:30-6:00 p.m. - SOCIAL HOUR

6:00 p.m. - PANEL DISCUSSION: STABILITY OF TALL BUILDINGS

Moderator: George Winter, Cornell University

Panelists: R. Shankar Nair, Alfred Benesch & Company, "Evaluation of Overall Stability Effects in Tall Buildings"

Fazlur R. Khan, Skidmore Owings & Merrill, "Structural Redundancy Criteria for Stability of Tall Buildings under Unforeseen Loads"

Jerry G. Stockbridge, Wiss, Janney, Elstner and Associates, "The Interaction Between Exterior Walls and Building Frames in Historic Tall Buildings"

8:00 p.m. - ADJOURN

Wednesday, April 8, 1981

8:30 a.m. - MORNING SESSION

Presiding: Bruce G. Johnston, Consultant

Task Group 8 - Dynamic Stability of Compression Elements

Chairman: D. Krajcinovic, University of Illinois at Chicago Circle

PROGRAM

. Dynamic Snap-Through of Shallow Structures

S. M. Holzer and R. H. Plaut, Virginia Polytechnic Institute and State University

. Stability of Circular Cylindrical Structures Subject to Fluid Flow

S. S. Chen, Argonne National Laboratory

. Dynamic Response and Stability of Rigid Frames Subjected to Time-Dependent Axial Forces

C. H. Tay, National University of Singapore
C. K. Wang, University of Wisconsin - Madison

Task Group 18 - Unstiffened Tubular Members

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

. Post Buckling Behavior of Tubular Beam-Columns

D. R. Sherman, University of Wisconsin - Milwaukee

. Column Strength of Cold-Formed Tubular Sections

R. G. Slutter and R. J. McDermott, Lehigh University

Task Group 22 - Stiffened Cylindrical Members

Chairman: C. D. Miller, Chicago Bridge & Iron Company

. Preliminary Results of Tension and Collapse Tests on Fabricated Steel Cylinders

C. D. Miller, Chicago Bridge & Iron Company

10:00-10:20 - BREAK

Other Research Reports

. On Inelastic Analysis of Steel Structures Subjected to Nonproporational Loading

N. T. Tseng and G. C. Lee, State University of New York at Buffalo

. Web Buckling Under Cyclic Loading

E. P. Popov and K. Hjelmstad, University of California, Berkeley

. Eccentric Load Test of Angle Column Simulated with MSC/NASTRAN Finite Element Program

G. Haaijer, P. S. Carskaddan and M. A. Grubb, U. S. Steel Corporation

PROGRAM

. The ECCS Method for Checking the Stability of Aluminum Alloy Columns

F. Mazzolani, University of Naples
F. Frey, Ecole Polytechnique Federale, Lausanne
C. Cescotto, University of Liege

11:20 a.m. - END SESSION

11:30 a.m. - SSRC ANNUAL BUSINESS MEETING

12:00 noon - ADJOURN

SSRC GUIDE REPORTS

SSRC GUIDE (4TH EDITION)

Chairman: B. G. Johnston, Consultant
Editor: T. V. Galambos, University of Minnesota

Overview of Current Stability Problems

T. V. Galambos, University of Minnesota

There are four developments which have an effect on the research related to structural stability problems:

1) Publication of Technical Memorandum No. 5;
2) Availability of sophisticated computation techniques;
3) Emergence of probability-based design codes;
4) Availability of powerful small micro-and mini-computers in the design office.

Technical Memorandum No. 5, in its statement that all the significant effects which influence the maximum strength of an element, a member or a structure must be considered, poses some important questions which need to be answered. Efficient computation techniques are available to handle all kinds of geometric and material non-linearities, as well as initial imperfections and residual stresses. The question is not "how", but "what" to compute.

The following special needs are enumerated:

1) What are the types and magnitudes of initial geometric imperfections which affect the maximum strength?

2) When can initial imperfections be ignored, and when must they be considered?

3) What initial imperfections should be considered in design codes?

4) What are the statistical data on residual stresses in all kinds of shapes, and how can they be reliably predicted?

5) When and how to consider interaction (residual stress and initial initial imperfection; local and overall instability; large deflection and small deflection; first-order and second-order analysis; etc.)

6) How to deal with the stability of structures with damaged elements?

7) When and how to consider dynamic instability?

8) What is the end restraint in nominally pinned-end beams, beam-columns and columns?

SSRC GUIDE REPORTS cont'd

9) What is effect of load history?

10) What are the statistical data on material properties?

One could go on to enumerate many more questions, but basically it is now necessary to establish rational schemes for the guidance of the researcher and the designer.

1) Establish assumption hierarchies.

2) Unify analytical model development schemes.

3) Catalogue suitable methods of analysis, including computer codes.

4) Up-date and expand theoretical models.

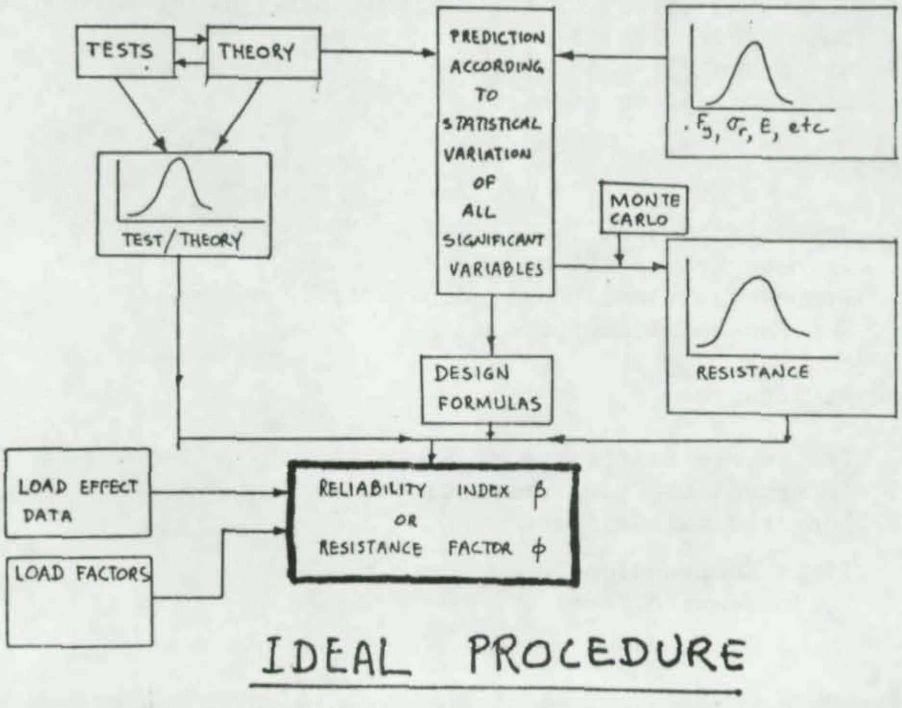
5) Perform "land-mark" experiments to verify theoretical models.

6) Identify test-solutions against which new solution schemes can be calibrated.

The challenge for structural stability research is to produce the results, both in terms of research (theory, analysis, experiments) and in terms of usable methods, which are needed by industry and the design profession.

The various task groups of the SSRC are, indeed, involved in work which is very much germane to the questions raised above, and the members of these groups are challenged to produce draft chapters for the fourth edition of the SSRC "Guide" to reflect the best answers to the enumerated problems.

The emergence of probability-based design specifications poses special demands to look at all stability problems in very fundamental ways, as outlined in the ideal procedure given the following flowchart.



IDEAL PROCEDURE

SSRC GUIDE REPORTS - cont'd

Finite Element Analysis of Stability Problems

R. H. Gallagher, University of Arizona

In this talk we discuss the chapter of the Fourth Edition of the SSRC Guide, which is concerned with finite element analysis of elastic instability problems. The finite element method is first defined. Some specialized forms of finite element analysis, such as the finite strip approach that are especially useful for elastic instability problems, are next described. The theoretical basis of the method, as it refers to bifurcation problems, is then reviewed. Finite element formulations of classical beam, plate and shell elastic instability problems are discussed and numerical results are presented to illustrate the scale of effort needed to achieve a desired level of accuracy. The analysis of practical structures requires the treatment of many other phenomena, including initial imperfections, postbuckling response, limit point behavior, and follower forces. The finite element analysis aspects of these phenomena are outlined.

Box Girders

David H. H. Tung, The Cooper Union

The following is a proposed outline of the new chapter on Box Girders, which is under consideration to be included in the 4th edition of the "SSRC Guide":

I. Introduction

II. Flanges

  a. Shear Lag Effect on the Ultimate Strength of Box Girders
  b. Unstiffened Flanges
  c. Stiffened Flanges
  d. Longitudinal Stiffeners

III. Webs

  a. Unstiffened Webs
  b. Transversely Stiffened Webs
  c. Transversely and Longitudinal Stiffened Webs
  d. Web Panels at Supports

IV. Web Stiffeners

  a. Transverse Stiffeners at Supports
  b. Intermediate Transverse Stiffeners
  c. Longitudinal Stiffeners

    1. In Compression Zone
    2. In Tension Zone

SSRC GUIDE REPORTS - cont'd

V. Review of Several Commonly-Used Design Codes

a. General Discussion
b. British
c. ECCS
d. German
e. U.S.A.

VI. Curved Girders
(To be developed by the Task Group on Horizontally Curved Girders)

VII. Research Needs

a. Fabrication Imperfections
b. Residual Stresses
c. Web and Web Stiffeners (Interaction of Flange and Web)
d. Interaction of Diaphrams and Other Girder Components
e. Haunches at Supports
f. Further Experimental Research on Shear Lag Effects on Ultimate Strength of Girders

VIII. References

The topic of "Unstiffened Compression Flanges" is used as a typical example to demonstrate the need for such a chapter, in view of the recent developments both here and abroad.

This reference topic has been investigated by several authors in recent years. Among these, an extensive parametric study was carried out to examine the effects of aspect-ratios, initial imperfections, residual stresses, as well as restraints along the unloaded edges (1). The analysis was based on elastic large deflection equations first proposed by von Karman, and later modified by Marguerre to take into consideration the effects of initial out-of-plane deflections. Plasticity was accounted for by using Ilyushin's single-layer yield function coupled with the Prandtl-Reuss flow rule, and assuming elastic-perfectly plastic material behavior. The distribution of residual stresses due to welding was assumed to consist of two narrow edge strips of tensile yield stress, in equilibrium with a central portion in uniform compression. The resulting finite difference equations were solved using dynamic relaxation.

The aspect-ratio effect was first studied. The results indicated that both the maximum strength and pre-peak stiffness of longer plates are greater than those of a square plate. Additional investigations showed that, within an arbitrary range of aspect-ratio between 0.67 and 1.0, a minimum value of peak stress occurs at an intermediate aspect-ratio when the initial out-of-flatness is in the practical range; this critical aspect-ratio, however, varies from study to study. On the other hand, no such minimum peak strengths occur for plates with larger initial out-of-flatness.

Subsequent studies on square plates suggested the following two conclusions:

1. Both the initial out-of-flatness and residual stresses tend to reduce the maximum strength as well as the pre-peak stiffness. The initial out-of-flatness is the dominant factor in slender plates, while high levels of residual stress have a pronounced weakening effect on stiffness in the case of stocky plates.

2. While the restraints along the unloaded edges have no significant effects in the case of stocky plates, the strength of unrestrained slender plates, particularly when coupled with large initial out-of-flatness, is substantially lower in comparison.

The results of the investigations were presented in the form of design curves for maximum strength. These curves, together with others, are shown in Fig. 1 (2).

On the basis of these research data, Fig. 1, Wolchuk (2 and 3) proposed a design strength curve which consists of a second-degree parabola and a straight line as shown in Fig. 2.

References

1. Frieze, P.A., Dowling, P.J. and Hobbs, R.E., "Ultimate Load Behaviour of Plates in Compression", Steel Plated Structures, Crosby Lockwood Staples, London, 1976, pp.24-50.

2. "Design Specifications for Steel Box Girders", Report No. DOT-FH-11-9259, Wolchuk and Maybaurl, June, 1979.

3. Wolchuk, R., "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.

4. Dwight, J.B. and Little, G.H., "Stiffened Steel Compression Panels -- A Design Approach," University of Cambridge, Dept. of Engineering, Technical Report CUED (C-Struct), TR. 38, 1974.

SSRC GUIDE REPORTS cont'd

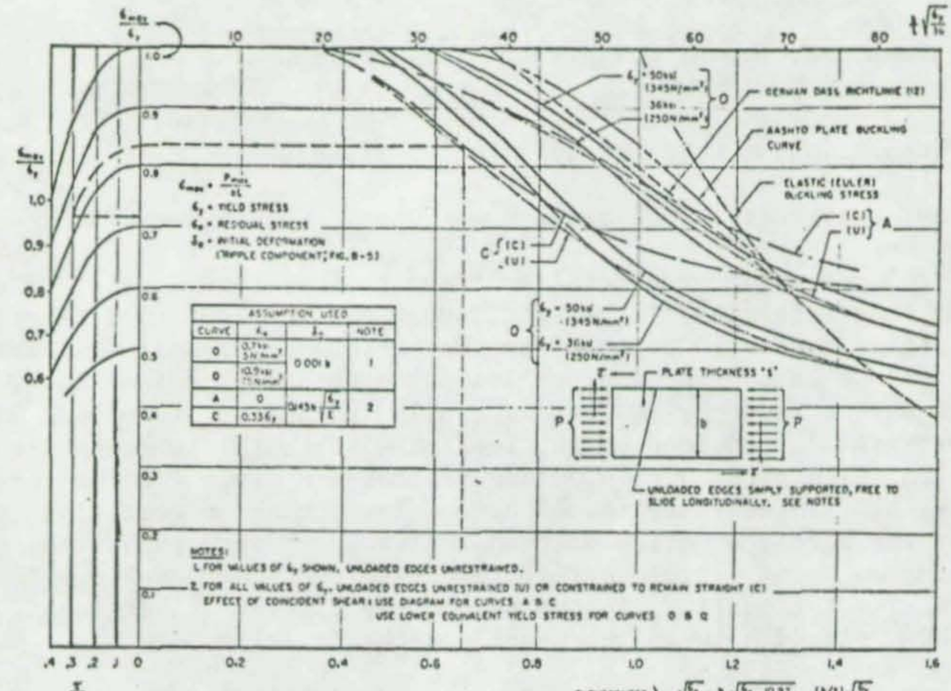


FIG. 1. PLATE STRENGTH CURVES FOR UNSTIFFENED PANELS
CURVES O & A FOR 'UNWELDED PLATES'; CURVES Q & C FOR "HEAVILY WELDED PLATES"
O & Q BY DWIGHT, LITTLE & ROGERS (4); A & C BY FRIEZE DOWLING & HOBBS (5)

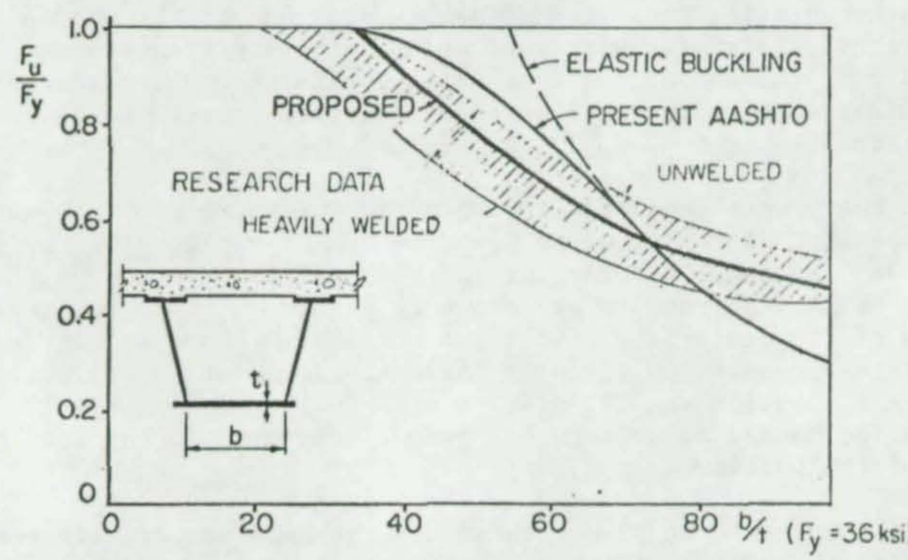


Fig. 2. Strength of Unstiffened Flange Panels

TASK GROUP REPORTS

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

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

Effect of Insulation on Cold-Formed Z-Purlin Strength

T. M. Murray, University of Oklahoma

Typical pre-engineered metal building roof systems consist of a metal panel over fiberglass insulation supported by cold-formed Z-purlins. The metal panel is usually fastened to the purlins using self-drilling fasteners. Since bending of the purlins is about non-principal axes, the purlins have a tendency to move laterally and twist. Torsional and lateral restraint is provided by the roof panel through the connection to the purlin. The panel also provides restraint against lateral-torsional buckling. To evaluate the effect of insulation on restraint supplied by the panel, a series of comparative tests were conducted and are reported here. In one set of the tests the roof panel was fastened directly to the purlin. In the second set 3 in. of fiberglass insulation was placed between the panel and the Z-purlin.

The test set-up consisted of two Z-purlins spaced 5 ft. 0 in. on center, spanning 20 ft. 0 in. and supported on short rafter sections. The nominal 8 in. deep purlins were oriented with both top flanges pointing outward. Shear plates welded to the rafter sections were used to support the purlin ends. Simulated live load was applied using solid concrete blocks. Initial load increments were 33 lbs. per foot; the load increments were decreased near ultimate load.

Instrumentation consisted of scales attached to the web and flanges of the purlin. The scales were read using surveying transits and levels positioned near the end of the test set-up. The accumulated data allowed the determination of horizontal movement, vertical movement and rotation of the purlin.

Six tests were conducted in two series of three each: D-series with the deck fastened directly to the Z-purlin and DI-series with the insulation between the panel and the purlin. Typical load versus vertical centerline deflection results are shown in Figure 1. Load versus lateral deflection of the centerline of the two purlins is shown in Figure 2. A summary of test results is given in Table 1. The average failure load without the insulation was 211 plf and with the insulation was 195 plf. The insulation caused an average 8.2 percent decrease in the load carrying capacity of the purlins.

To the writer's knowledge these are the only comparative tests demonstrating the effect of fiberglass insulation on Z-purlin strength. The data is of course limited to a very specific test set-up and must be considered as tentative.

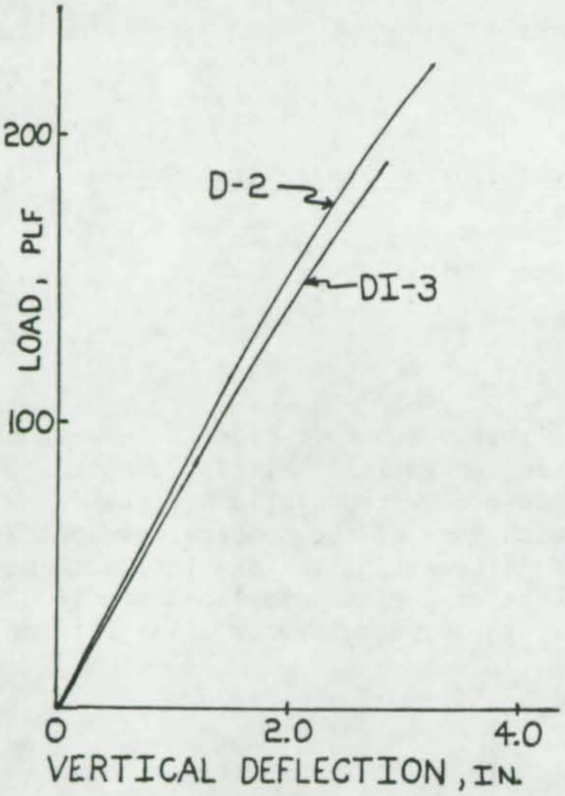


Figure 1. Load vs. Vertical Deflection

Table 1. Test Results

Test No.	W_u (plf)	Def'l (in.)
D-1	215	2.93
D-2	221	2.89
D-3	198	2.87
DI-1	206	3.05
DI-2	190	2.24
DI-3	190	2.52

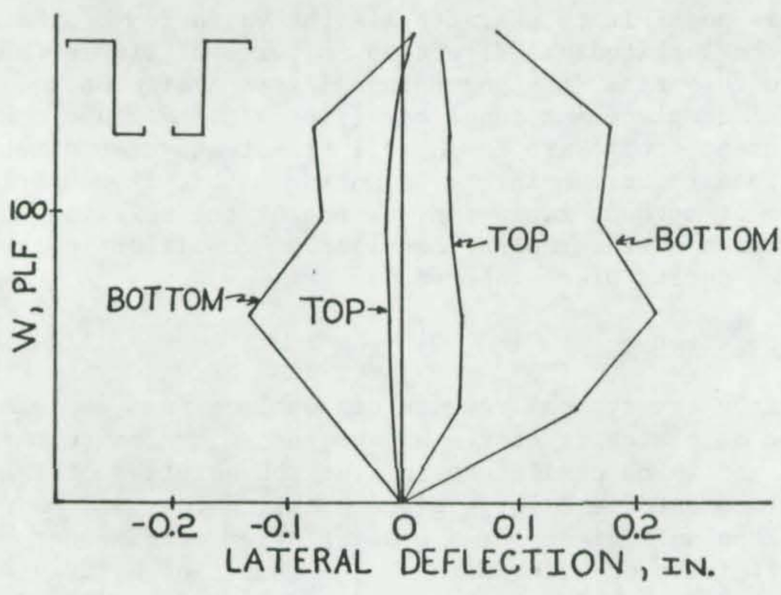


Figure 2. Load vs. Lateral Deflection

TG-13 cont'd

Finite Strip Analysis of Postbuckling Behaviour of Plate Structures-Some Recent Results

S. Sridharan and T. V. Galambos, Washington University

General

Local buckling of a plate structure involves out of plane deformation of the members with the junctions remaining essentially straight. Local torsional and forms of 'overall' buckling involve translations of the junctions in the cross-sectional plane with some of the members developing inplane deformation. In these cases, it is essential to take into account in the analysis the destabilizing influence of inplane displacements in the transverse direction. In particular, the expression for axial strain must be taken in the form (Fig. 1):

$$\varepsilon_x = \frac{\partial u}{\partial x} + \frac{1}{2} \left\{ \left(\frac{\partial w}{\partial x}\right)^2 + \left(\frac{\partial v}{\partial x}\right)^2 \right\}$$

The finite strip method appears to be the logical choice for the analysis of prismatic plate structures undergoing local buckling [1-4]. In this problem, it is possible to characterize the variation of the displacements in the longitudinal direction in terms of simple functions which can be shown to satisfy the governing differential equations. Suitable polynomial displacement functions (linear u and v and cubic for 'w' in the present study) are employed in the transverse direction. An order of magnitude reduction in the computing effort in comparison to the finite element method, rapid convergence of the solution and the exact satisfaction of the inplane equilibrium condition in the x-direction are the merits of the approach.

The present investigation

In this report a few typical results on postlocal and post-local-torsional buckling of plate structures are presented. A perturbation technique is employed which consists mainly of the solution of the second order displacement field for a given buckling load and mode. The results are often sufficient for the description of the post-buckling behaviour in the vicinity of bifurcation. The number of halfwaves is assumed to be even in order to render the bifurcation symmetric. The structure is assumed to be compressed by rigid end plattens. Two types of loading are considered, one of prescribed end displacement and the other of prescribed load eccentricity. Details of the analysis are fully described in reference 5.

Examples

(i) Local buckling under the two modes of loading

Examples were set up to study the behaviour of plate structures under the two modes of loading mentioned above, but which produced the same linear stress distribution in the longitudinal direction prior to buckling. Fig. 2 illustrates the contrast in the behaviour of a channel section strut when subjected to uniform compression and to a compression load through its centroid. The strut has a greater stiffness in the former mode of loading and as seen from Fig. 2, there exists a wide difference in the distribution in the longitudinal stresses in the postbuckling range.

(ii) Post-local-torsional buckling behaviour

Several examples of lipped channel sections are considered. The buckling mode is illustrated in Fig. 3. Fig. 4 plots the variation of average stress, the stresses at the stiffener tips (A and B) at the planes of symmetry and that at the node against the inplane displacement of the stiffener. It is seen that there takes place a fairly rapid build up of stresses at the tip of the lip in comparison to the average stress. Thus when local-torsional buckling occurs, the failure will be initiated by compressive yield at the lip.

(iii) Stiffened plate in pure bending

Fig. 5 shows the example of a plate carrying a single stiffener subjected to pure bending. From the moment curvature relationship it is seen that the plate has a significant elastic postbuckling stiffness; however the compressive stress at the tip of the stiffener builds up at a rapid rate and would lead to plastic yielding in practical cases thus setting a limit to the moment carrying capacity of the structure.

TG-13 cont'd

References:

1. Sridharan, S. "Elastic Postbuckling Behaviour and Crinkly Collapse of Plate Structures", Ph.D. Thesis, University of Southampton, 1978.

2. Graves-Smith, T. R. and Sridharan, S. "A Finite-Strip Method for the Post-locally-buckled Analysis of Plate Structures". International Journal of Mechanical Sciences, Vol. 20, pp. 833-843, 1978.

3. Sridharan, S. and Graves-Smith, T. R. "Postbuckling Analyses with Finite Strips", To be published in the Journal of the Engineering Mechanics Division, ASCE, August 1981.

4. Sridharan, S. "Post-local-buckling Analysis of Plate Structures" Under review for publication in the Journal of Structural Division, ASCE.

5. Sridharan, S. "Behaviour of Plate Structures after local-torsional and overall Buckling", Department Report No. 59, Civil Engineering, Washington University in St. Louis.

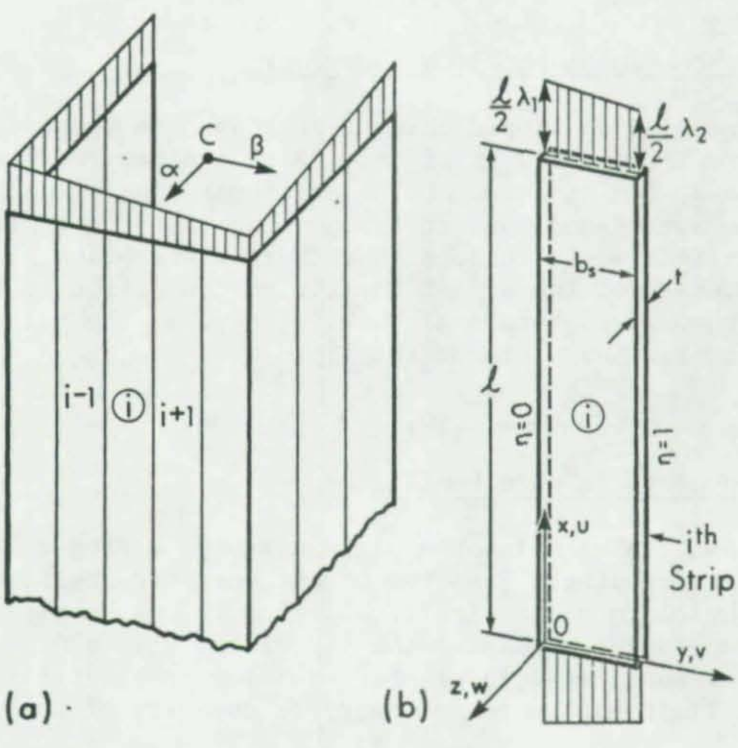


Fig. 1 Finite strip configuration, axes of coordinates and positive directions of displacements

TG-13 cont'd

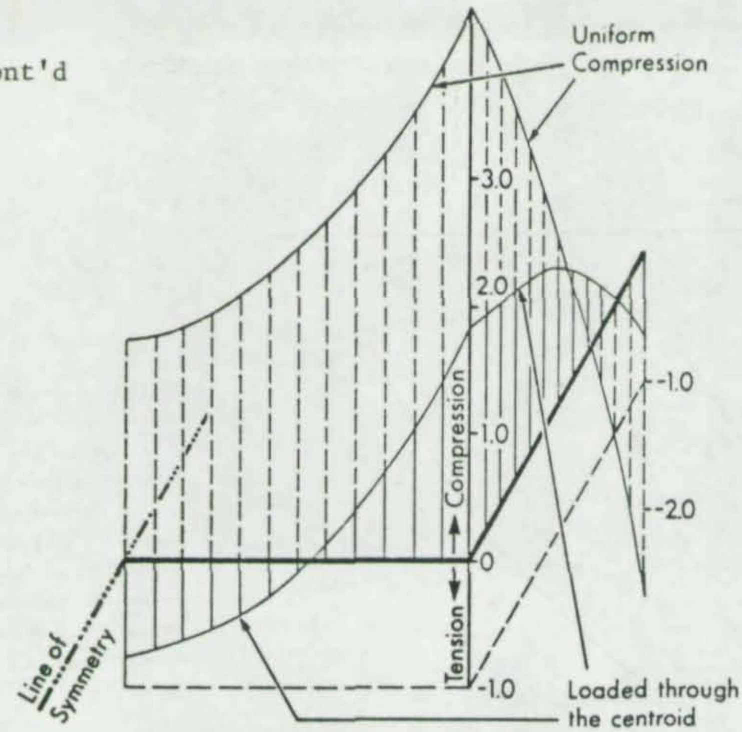


Fig. 2 Change in stress distribution from the unbuckled configuration to the buckled configuration corresponding to $\delta/t = 1.0$, for a strut with $b/a = 0.5$, $a/t = 100$ at the middle section of the column. (The stresses are given in a dimensionless form, $\sigma/E \times 10^3$, a = width of flange; b = width of web; δ = deflection at the centre of flange)

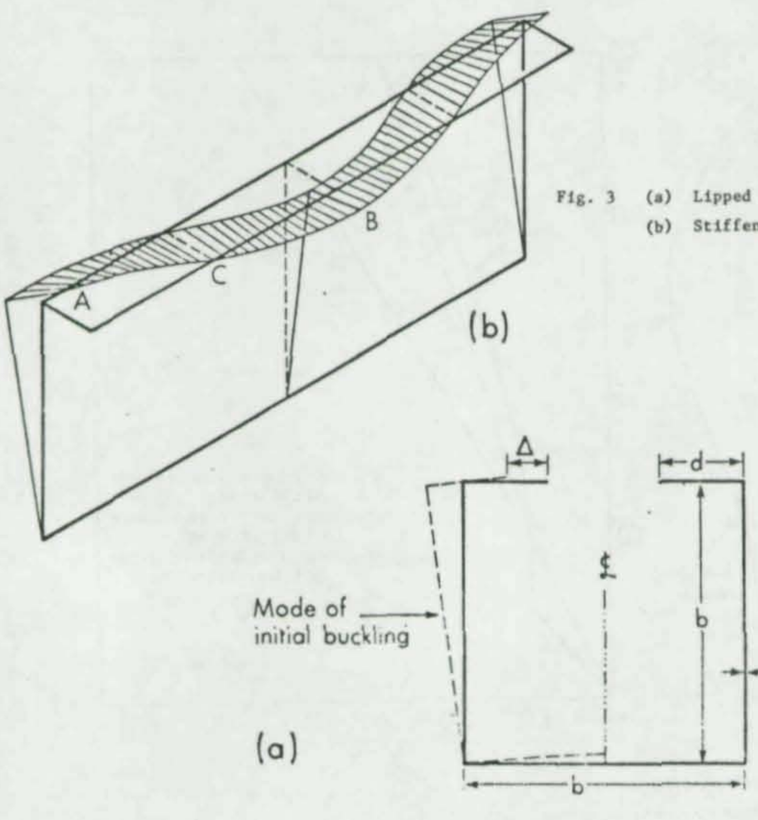


Fig. 3 (a) Lipped channel investigated
(b) Stiffener buckling mode

TG-13 cont'd

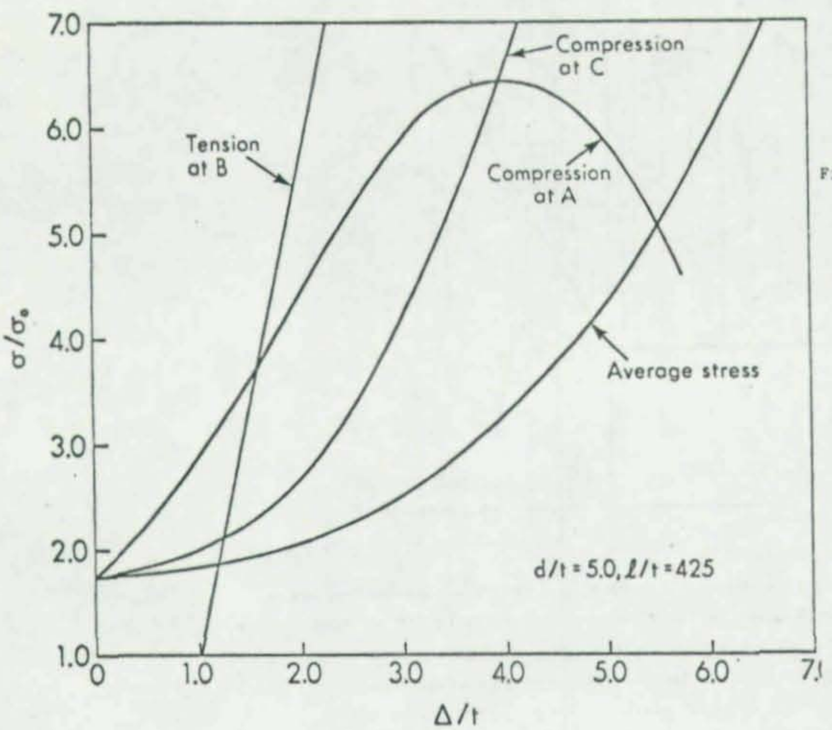


Fig. 4 Average stress carried by the lipped channel (Fig. 3a) and the stresses at the tips of the stiffeners at A, B and C (Fig. 3b) plotted against the inplane displacement of the stiffener (ℓ = halfwave length of buckling)

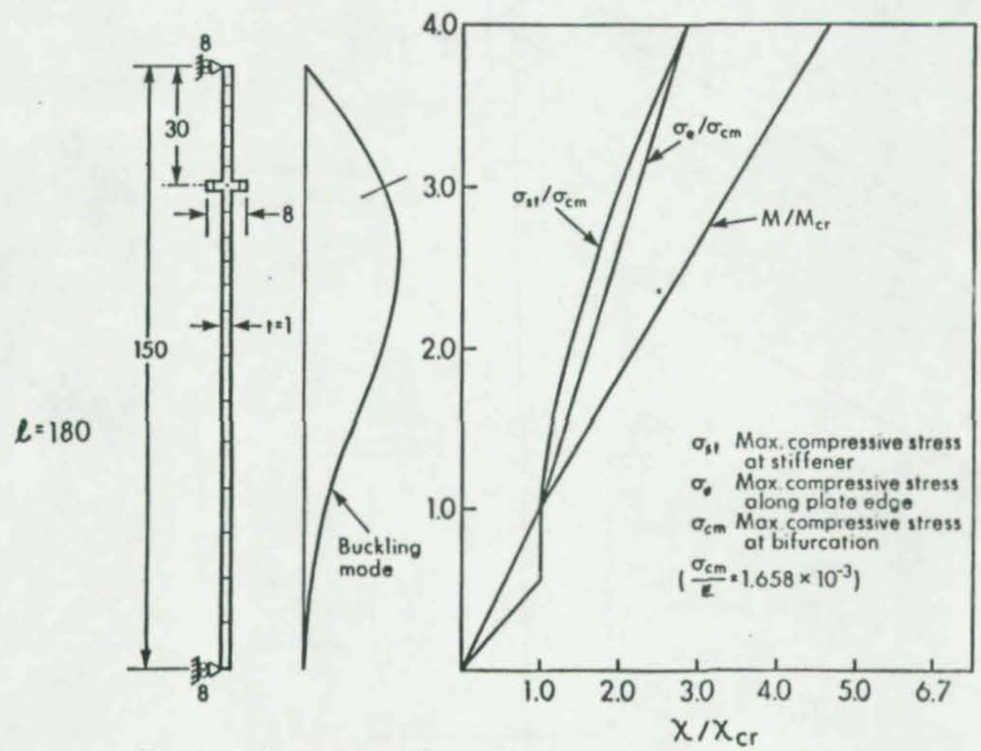


Fig. 5 Bending moment, maximum stiffener stress and the maximum plate edge stresses plotted against curvature χ for a plate carrying a single stiffener. (M_{cr} and χ_{cr} are the moment and curvature at bifurcation; 'ℓ' is the halfwave length of buckling)

SSRC SUPPORTED RESEARCH PROJECT REPORTS

Effect of End Restraints on the Stability of Geometrically Imperfect Columns as Parts of a Plane Frame

Z. Razzaq, University of Notre Dame

An inelastic study of nonsway behavior of imperfect steel columns as parts of a plane frame is presented. Beams framing-in at the column ends are assumed to provide linear or elastic-plastic partial rotational end-fixity. Results are obtained for the case of identical restraints at both ends of the column and are based on a tangent stiffness finite-difference analysis of the differential equation of column equilibrium. The effect of partial end-fixity on the column static instability about the minor axis is investigated, taking into account possible initial crookedness and residual stresses. Three main types of columns are studied: nearly straight columns with residual stresses; columns with initial crookedness and no residual stresses; and columns with both initial crookedness and residual stresses.

Fig. 1(a) shows schematically an initially crooked column OT of length L with identical rotational restraints at O and T and subjected to a gradually increasing static load P. The vertical axis through O and T is designated by z, and the lateral initial crookedness u_i, and the lateral centroidal deflection u due to P are in the xz plane, where x is the major principal axis of the cross-section. The rotational restraints are provided by identical beams OB and TC, as shown in Fig. 1(b), rigidly connected to the column ends, and simply-supported at the far ends B and C. Fig. 1(c) shows an idealization of the residual stress distribution in a hot-rolled wide-flange shape adopted in this study. In this figure, x and y are the principal axes, the web has tensile stresses σ_{rt} which are constant, and the stresses in the flanges are assumed to vary linearly from σ_{rt} at the flange center to a compressive stress σ_{rc} at the four flange tips. The residual stresses are also assumed to be constant across the thickness of the plates. The material of the column is A36 steel with an idealized elastic-plastic stress-strain ($\sigma - \varepsilon$) curve as shown in Fig. 1 (d) and assumed to be identical in both tension and compression. The initial crookedness is taken in the form of a half-sine wave.

Two types of end restraint conditions are considered, as shown in Figs. 2(a) and 2(b), in which $|M_o|$ is the absolute value of the end spring moment and θ the column end slope u' (0) at z = 0 due to the applied load P. It is noted that althought the beams of the type shown in Fig. 1(b) possess a nonlinear behavior near the knee of the moment-rotation curve shown in Fig. 2(b), an elastic-perfectly- plastic approximation is used here, with M_{op} as the beam plastic moment.

RESEARCH PROJECTS - cont'd

Fig. 3 shows three load-deflection curves for W8x31 columns with both initial crookedness $[u_i = 0.001 L \sin(\pi z/L)]$ and residual stresses ($\sigma_{rc} = 0.3\ \sigma_Y$) corresponding to three different end restraint stiffness (k) values. The vertical axis of this figure gives the applied dimensionless load defined as $\bar{P} = P/P_Y$, where P_Y is the column squash load. The effect of varying the degree of end-fixity can be seen clearly from this figure. Fig. 2(a) defines the type of restraints used for these columns. The minor-axis slenderness ratio was held constant at 71.29.

Some conclusions drawn from this research are as follows:

1. The degree of end-fixity plays a significant role in the behavior of initially crooked columns with no residual stresses. The presence of initial crookedness resulted in the development of end spring moments which were an order of magnitude larger than those for the nearly straight columns with residual stresses. The restraint provided by these larger end moments helped in increasing the column capacity.

2. For columns with elastic-plastic restraints, a relatively small or no gain in the load-carrying capacity is observed beyond the plastification of the end springs.

3. Of the three types of columns studied, those with both initial crookedness and residual stresses experienced the most pronounced effect of the presence of partial end restraints in terms of an increase in the load-carrying capacity.

4. In general, as the degree of partial end-fixity increases, the effect of crookedness alone on the column strength becomes gradually less detrimental than the effect of residual stresses alone. Generally, the presence of residual stresses alone is more detrimental than the presence of initial crookedness alone.

5. The presence of residual stresses in the initially crooked columns has the most pronounced effect on the column strength when the ends are nearly pinned or lightly restrained; the effect tends to diminish with an increase in the end restraint stiffness.

6. In the presence of somewhat large end restraints, a decrease in the slenderness ratio of an initially crooked column may sometimes result in a near complete nullification of the effect of residual stresses, that is, the column strength will be the same regardless of whether the residual stresses are present or not.

The above conclusions are based on the theoretical results for a total of thirty-three columns with various restraints, slenderness ratios, etc. Futther research is being conducted on columns with unequal restraints and higher residual stresses. The seed money support to conduct this research was provided by the Structural Stability Research Council.

RESEARCH PROJECTS - cont'd

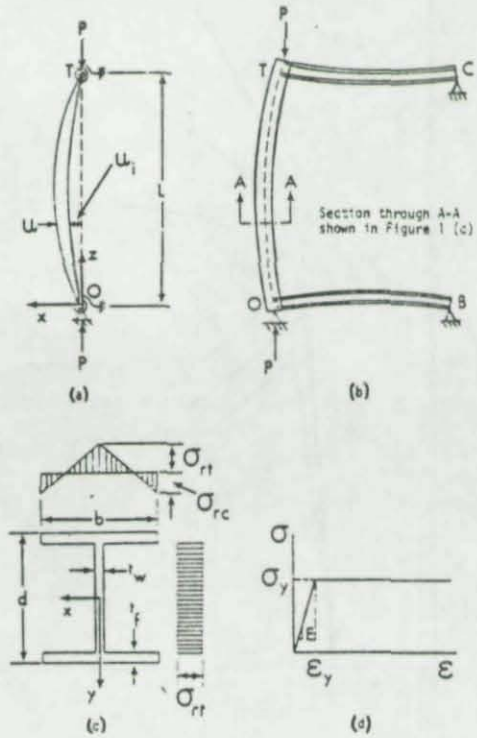


Figure 1. The problem

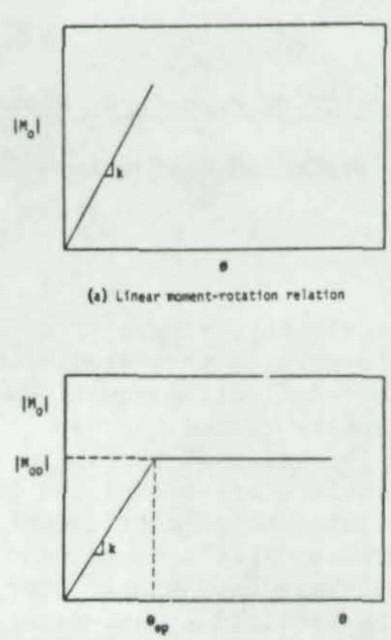


Figure 2. Moment-rotation characteristics

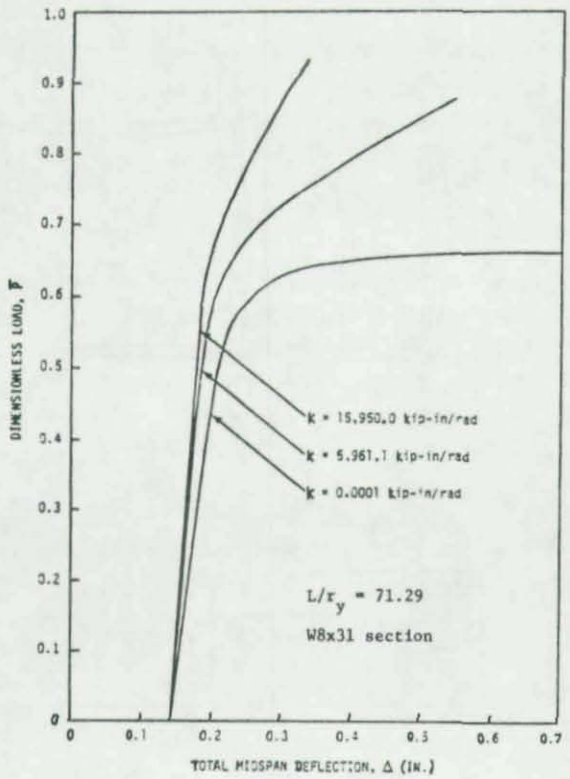


Figure 3. Load-deflection curves for columns with both initial crookedness and residual stresses

Criteria, Analysis, and Design of Braced and Unbraced Frames

M. Biswas and R. Earwood, Texas A&M University

The work of Lay on single bay, single story frames has been reviewed. Except for notations, Lay's work is very similar to that of Goldberg, i.e., they are both based on slope-deflection equations of equilibrium. From Lay's work, however, a quantity termed "threshold stiffness" can be isolated and identified. The value of threshold stiffness is calculated for several single bay, single story frames and it is expressed as functions of parameters related to geometric configurations, frame lateral stiffness and member stiffness. Lay's expression is then modified to find similar information for multiple bay, single story frames, and single bay, multiple story frames. The preliminary numerical calculations are performed using programmable calculators. Once the method for a frame type is validated, a program is then written for a computer and more extensive results are obtained. This type of distributed computing appears to optimize manpower effort as well as computer costs. Numerical results in the form of table and graphs are presented.

RESEARCH PROJECTS - cont'd

Influence of Imperfections on the Maximum Strength of Restrained Beam Columns

S. Vinnakota, University of Wisconsin-Milwaukee

A numerical method, using finite difference technic, to study the planar behavior of non-sway, steel, beam-columns has been presented. The influence of residual stresses, initial geometrical imperfections and end-restraints are included in the study, so as to confirm to the requirements of TM-5 of SSRC for the maximum strength determination of members. The restraints considered (Figs. 1 and 2) have non-linear moment-rotation characteristics. The loading on the member may consist of longitudinal forces at ends, end-moments, transverse concentrated and uniformly distributed loads. Several numerical examples are presented. The influence of geometrical imperfections and small, non-linear end-restraints on the maximum strength of W8x35 columns bent about their minor axis are shown in Fig. 3.

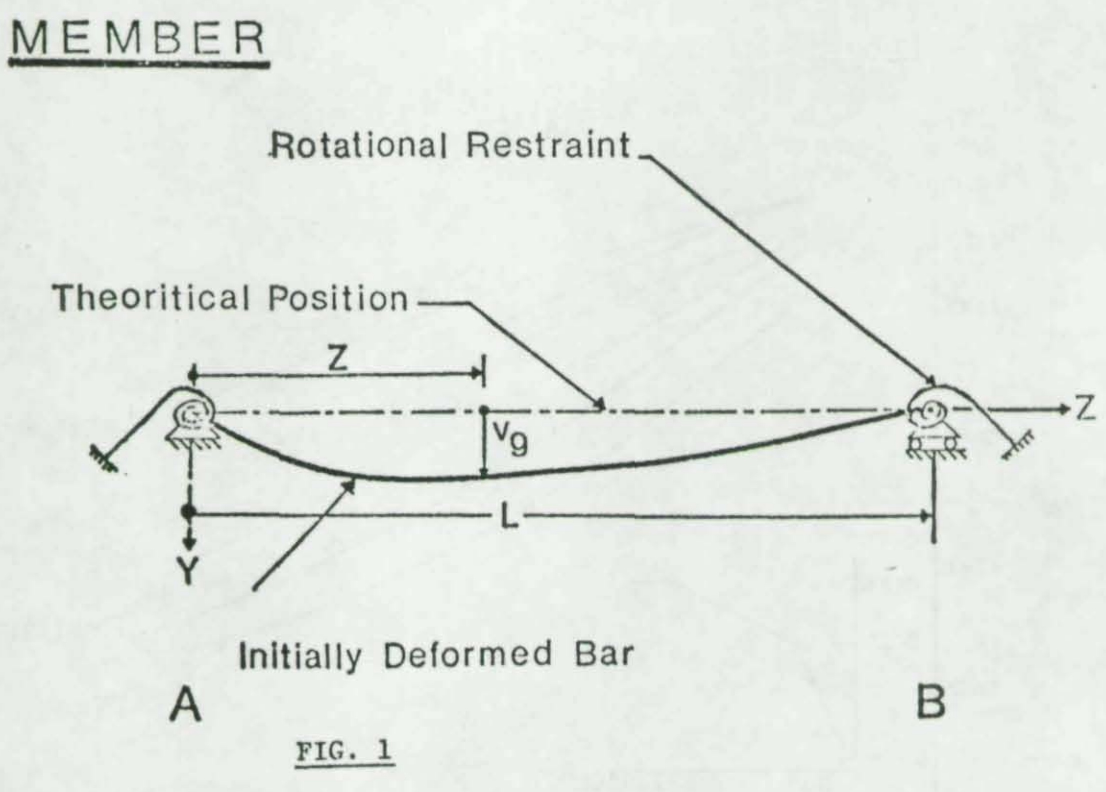


FIG. 1

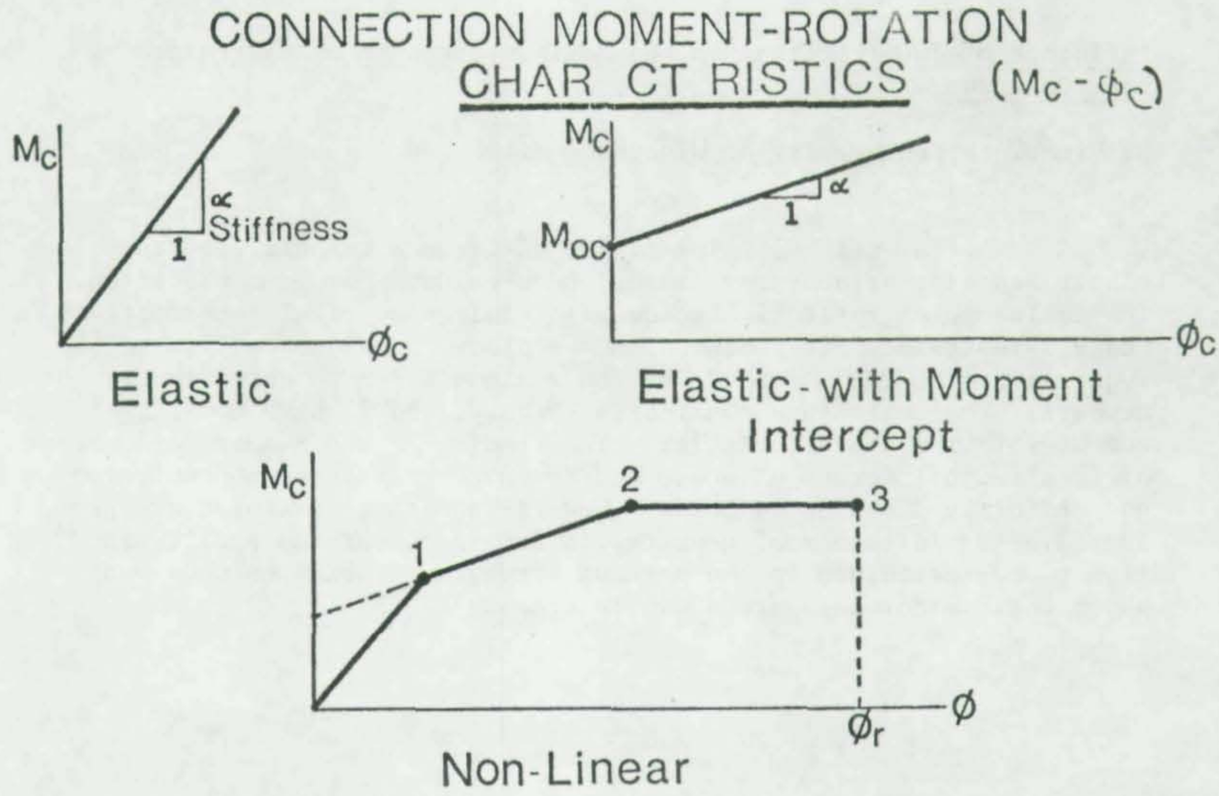

CONNECTION MOMENT-ROTATION
CHAR CT RISTICS
(M_c - ϕ_c)
M_c
α
Stiffness
1
ϕ_c
Elastic
M_c
α
1
M_{oc}
ϕ_c
Elastic - with Moment Intercept
M_c
1
2
3
ϕ
ϕ_r
Non-Linear

FIG. 2

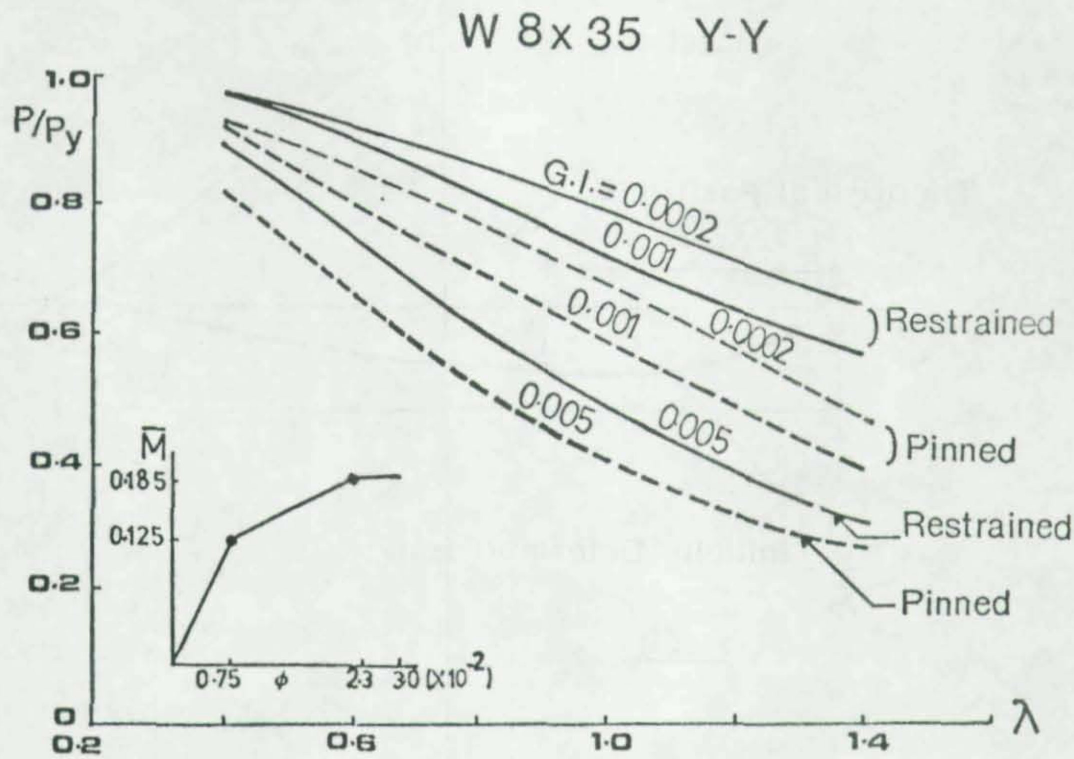

W 8x35 Y-Y
1.0
P/Py
0.8
0.6
0.4
0.2
0
0.2
0.6
1.0
1.4
λ
G.I.=0.0002
0.001
0.001
0.0002
0.005
0.005
Restrained
Pinned
Restrained
Pinned
M̄
0.185
0.125
0.75
φ
2.3
3.0 (x10^-2)

FIG. 3

The Effect of the Material Damage on the Buckling of Structures

D. Krajcinovic, University of Illinois at Chicago Circle

ABSTRACT

The deformation process of a polycrystalline structural material reflects a host of different processes developing on the microscopic scale. The most prominent deformation mechanisms can be conveniently classified into two different categories:

- plastic (viscous) deformation reflecting the rearrangement of dislocations, slip, grain-boundary sliding, etc.,

- nucleation and evolution of microdefects, cracks, voids and inclusions.

The second class becomes especially important in the third phase of the creep and in conditions commonly defined as brittle (low strains, low temperatures, high strain rates, chemically embrittled materials, etc.).

In order to describe the state of the damage accumulation (i.e., the level to which the material deteriorated) locally it is necessary to introduce a new internal (hidden) variable. In the past this task has been successfully accomplished using the Kachanov's damage variable representing the void density in the cross section. The major problem associated with this (or any other) approach consists of the establishment of a rational damage law defining the functional interdependence between all or some of the state and internal variables.

The adopted procedure is illustrated on an example of a simple three-bar truss in which the two diagonal members are compressed (and damage free) while the horizontal member is in tension. A fairly straightforward analysis demonstrates the significant influence of the accumulated damage on the onset of instability. It was shown that the failure can occur due to the:

- brittle rupture of the tensile member,
- local buckling of the compressed members, and
- increasing deformation due to the gradual deterioration of the tensile member.

The report considers both the creep and instantaneous instability and points out the practical importance of this type of problems.

TASK GROUP REPORTS

TASK GROUP 1 - CENTRALLY LOADED COLUMNS

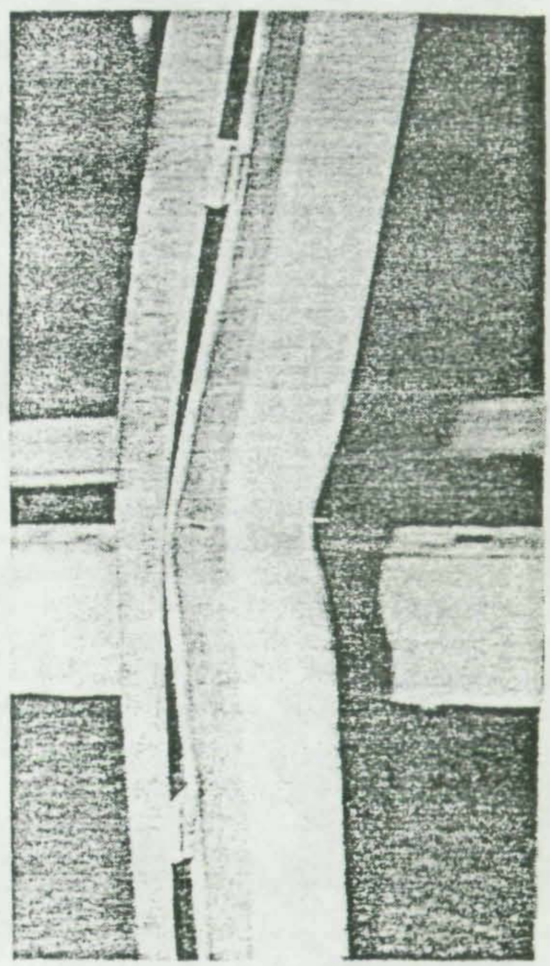
Chairman, R. Bjorhovde, The University of Alberta

Local Buckling in Cyclically Loaded Built-up Struts

E. P. Popov, University of California, Berkeley

At the 1979 SSRC Meeting, attention was called to a dramatic decrease in the column carrying capacities of columns subjected to severe cyclic loadings. An approach for estimating the column capacities during a few consecutive cycles was also indicated (1) . The described approach has now been refined and is available in a report (2). Another item of importance, which was only briefly mentioned elsewhere (3) , requires further comment. This is the matter of local buckling of the individual members in built-up struts between stitches. This effect became very significant in experiments with double-angle struts. The struts with a tendency to buckle at right angle to their cross-sectional axes of symmetry were especially poor. In conformity with AISC requirements, the slenderness ratios of the individual angles between the fillers did not exceed the governing ratios of the built-up members. An illustration of a typical failure is shown in the photograph. It is apparent that the cyclic plastic working of a strut in the hinge region softens the material due to the Bauschinger effect contributing to the deterioration of the member. Therefore, stitching of built-up critical compression members, in general, for service under severe load reversals as currently specified in standard codes appears to be unconservative. Further details may be found in Reference 2.

This work was supported by NSF and AISI.



References:

(1) Popov, E. P., "Inelastic Column Buckling Under Cyclic Loading," Proceedings, SSRC, 1979, pp. 38-40

(2) Black, R. G., Wenger, W. A., and Popov, E. P., "Inelastic Buckling of Steel Struts Under Cyclic Load Reversals," UCB/EERC-80/40, University of California, Berkeley, October 1980.

(3) Popov, E. P., "Inelastic Behavior of Steel Braces Under Cyclic Loading" Proceedings, 2nd U. S. National Conference on Earthquake Engineering, EERI, Stanford, CA, Aug. 1979, pp. 923-932.

TASK GROUP 3 - COLUMNS WITH BIAXIAL BENDING

Chairman, J. Springfield, Carruthers and Wallace Limited

Comparison of Test Results with Design Equations for Biaxially Loaded Steel Beam-Columns

S. U. Pillai, Royal Military College of Canada

Current North American specifications (1,2,3) recommend interaction equations for the proportioning of biaxially bent beam-columns. Two typical sets of such equations, applicable to certain class of sections, are given below:

Linear Equations

$$\frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \leq 1.0 \qquad \text{SSRC Eq.8.22}$$

$$\frac{P}{P_y} + \frac{0.85\, M_x}{M_{px}} + \frac{0.60\, M_y}{M_{py}} \leq 1.0 \qquad \text{SSRC Eq.8.23}$$

$$\frac{P}{P_u} + \frac{C_{mx}\, M_x}{M_{ux}\,(1 - P/P_{ex})} + \frac{C_{my}\, M_y}{M_{uy}\,(1 - P/P_{ey})} \leq 1.0 \qquad \text{SSRC Eq.8.29}$$

Exponential Equations

$$\left(\frac{M_x}{M_{pcx}}\right)^{\zeta} + \left(\frac{M_y}{M_{pcy}}\right)^{\zeta} \leq 1.0 \qquad \text{SSRC Eq.8.24}$$

$$\left(\frac{C_{mx}\, M_x}{M_{ucx}}\right)^{\eta} + \left(\frac{C_{my}\, M_y}{M_{ucy}}\right)^{\eta} \leq 1.0 \qquad \text{SSRC Eq.8.27}$$

See Ref. 3 for definition of various terms.

The accuracy of the capacities predicted by the above equations is dependent on (i) the precision with which the axial load capacity, P_u, and the bending strengths, M_{ux} and M_{uy} are computed, and (ii) the applicability of the equivalent moment factor, c_m, to biaxial bending.

Summary of a comparison (4) of the above equations with results of recent European tests (5) on 81 beam-columns under unsymmetrical and reversed curvature biaxial bending is presented herein. Several alternative procedures were considered for computing the terms P_u, M_{ux} and C_m.

Results of Comparison

A summary of the results is given in Table 1. In general, the exponential form of equations result in a mean value for the ratio 'test load/predicted load' closest to unity, with the least variation. Naturally, in a high proportion of individual cases the predictions are on the unsafe side. Thus, the exponential equations represent "mean curves" - as indeed they appear to have been developed - rather than conservative design equations on the safe side. In contrast, the linear equations give slightly more conservative predictions, the proportion of unsafe predictions being within the range of experimental scatter.

Influence of the Methods of Computing C_m, M_{ux} and P_u.

In Ref. 5 both the AISC/CSA equation and Massonnet's equation (3) were used for computing C_m, and the resulting difference in the predicted capacity was found to be negligible (less than 1%). The present study shows that both CSA (1) and AISC (2) equations for M_{ux} lead to similar predictions, the difference in capacity being only about 1%. However, the method of computing the axial load capacity P_u has a more pronounced influence on the biaxial strength predicted by the interaction equations. For these tests, use of the CRC Column Strength Curve gives the least conservative predictions. The Multiple Column Curves and the Imperfect Column Formula (SSRC Eq.3.19 with $ec/r^2 = 0.25$, $\delta_o = L/1000$ and $\sigma_{max} = F_y$) give respectively 5% and 16% more conservative predictions.

Conclusions

The C_m factor concept leads to satisfactory results for biaxially loaded columns. There is little difference between the capacities predicted by using either of the two currently available formulas for computing C_m. Both CSA and AISC formulas for M_{ux} result in similar predictions. However, the strength predicted depends to a larger extent on the method used for computing P_u. Amongst the three procedures considered, the best result is obtained with the CRC Column Strength Curve.

The linear interaction equations give safe and conservative predictions of strength. The exponential equations represent mean strength curves and not conservative design curves. Hence, these equations should be used with an 'appropriate' capacity reduction factor ϕ .

References

1. Steel Structures for Buildings - Limit States Design, CSA Standard S16.1-M78, Canadian Standards Association, 1978.

2. Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, American Institute of Steel Construction, Nov. 1978.

3. Johnston, B.G., Ed., SSRC Guide to Stability Design Criteria for Metal Structures, 3rd Ed., John Wiley & Sons, 1976.

4. Pillai, S.U., *Comparison of Test Results with Design Equations for Biaxially Loaded Steel Beam-Columns*, Report to SSRC TG-3, Civil Engineering Research Report No. 80-2, Royal Military College of Canada, 1980.

5. Anslijn, R., and Massonnet, C.E., New Tests on Steel I Beam-Columns in Mild Steel Subjected to Thrust and Biaxial Bending; Festschrift Otto Jungbluth - 60 Jahre, published by Dr. Kellar, Technische Hochschule Darmstadt, Lehrstuhl fur Stahlbau, 1978 pp. 103-123.

TABLE 1. RATIO $P_{test}/P_{predicted}$

METHODS		LINEAR EQUATIONS			EXPONENTIAL EQUATIONS			
	P_u =	CRC	MCC, CSA	MCC, CSA	CRC	MCC, CSA	MCC, CSA	Imp. Col.
	M_{ux} *=	AISC	CSA	AISC	AISC	CSA	AISC	AISC
Mean ratio		1.114	1.147	1.159	1.001	1.040	1.050	1.161
Coefft. Var. %		9.80	10.21	10.48	8.63	9.42	9.62	11.00
% Results with ratio < 1.0		17.28	12.35	11.11	51.85	33.33	30.86	4.94

NOTES - SECTION: HEA200 Number of tests: 81

L/r_x: 40 to 96 C_m : *CSA/AISC Formula*

TASK GROUP 23 - EFFECT OF END RESTRAINT ON INITIALLY CROOKED COLUMNS

Chairman, W. F. Chen, Purdue University

Effect of Small End Restraint on Strength of H-Columns

H. Sugimoto and W. F. Chen, Purdue University

Abstract

A parametric study of the strength and behavior of an axially loaded wide-flange steel column is investigated from the viewpoint of beam-column analysis, taking into consideration the effects of residual stresses, geometric imperfections and small end restraints. A computer program has been developed to obtain the numerical results, from which effective length decreasing factor and maximum load increasing factor are proposed.

Summary and Conclusions

An analytical study of the behavior and strength of a symmetrically loaded wide-flange steel columns with small end restraint (web cleat) is made by means of the approximate deflection method and the bi-linear fitting for test data of moment-rotation relationships of beam-to-column connections.

It is concluded from this study that

1. The assumed deflection function which consists of the combination of sinusoidal and polynomial functions is valid and efficient for the analysis of column with small end restraints.

2. The amounts of decrease in deflection due to modest end restraint at or near maximum load for pinned end columns are 45% to 89% in the range of L/r=40 to 160.

3. The maximum load carrying capacity of modest end restraint columns increases for each initial imperfection 0.001L, 0.002L and 0.004L, when compared with each corresponding pinned end cases. For the case of Type 1 end restraint, the amounts of maximum increase are 17%, 12% and 9% for each initial imperfections, 0.001L, 0.002L and 0.004L, respectively. For Type 2 end restraint, this value becomes 45%, 37% and 23%.

4. The maximum load carrying capacity of modest end restraint columns decreases with an increasing initial imperfection. For strong axis bending, the reduction is 13% and 27% in maximum loads corresponding to 0.002L and 0.004L initial imperfections, respectively when compared with 0.001L initial imperfection case. For weak axis bending, these values become 28% an 37%.

TG-23 cont'd

5. The reduction in effective length factor and the increase in maximum load carrying capacity of restrained columns have a linear relation to the *restraint factor* (*rotational stiffness factor/plastic moment of column*). The value of effective length factor for Type 2 end restraint is 0.72.

6. For modest end restraint columns with 0.001L initial imperfection, CRC curve is more representative in the range of slenderness ratio, λ, up to 1.25 than other column curves.

Table 1:

Effective Length Factor ,K

(W 12x65,Strong,Type 1)

P_{max}/P_y	δ_o= .001 L	δ_o=.004 L
	K	K
.9	.88	.86
.8	.91	.89
.7	.92	.9
.6 .	.91	.90
.5	.91	.92
.4	.91	.94

Table 2:

Effective Length Factor ,K

(W 10x29,Weak, Type 2)

P_{max}/P_y	δ_o=.001 L
	K
.9	.7
.8	.72
.7	.71
.6	.72
.5	.73
.4	.73

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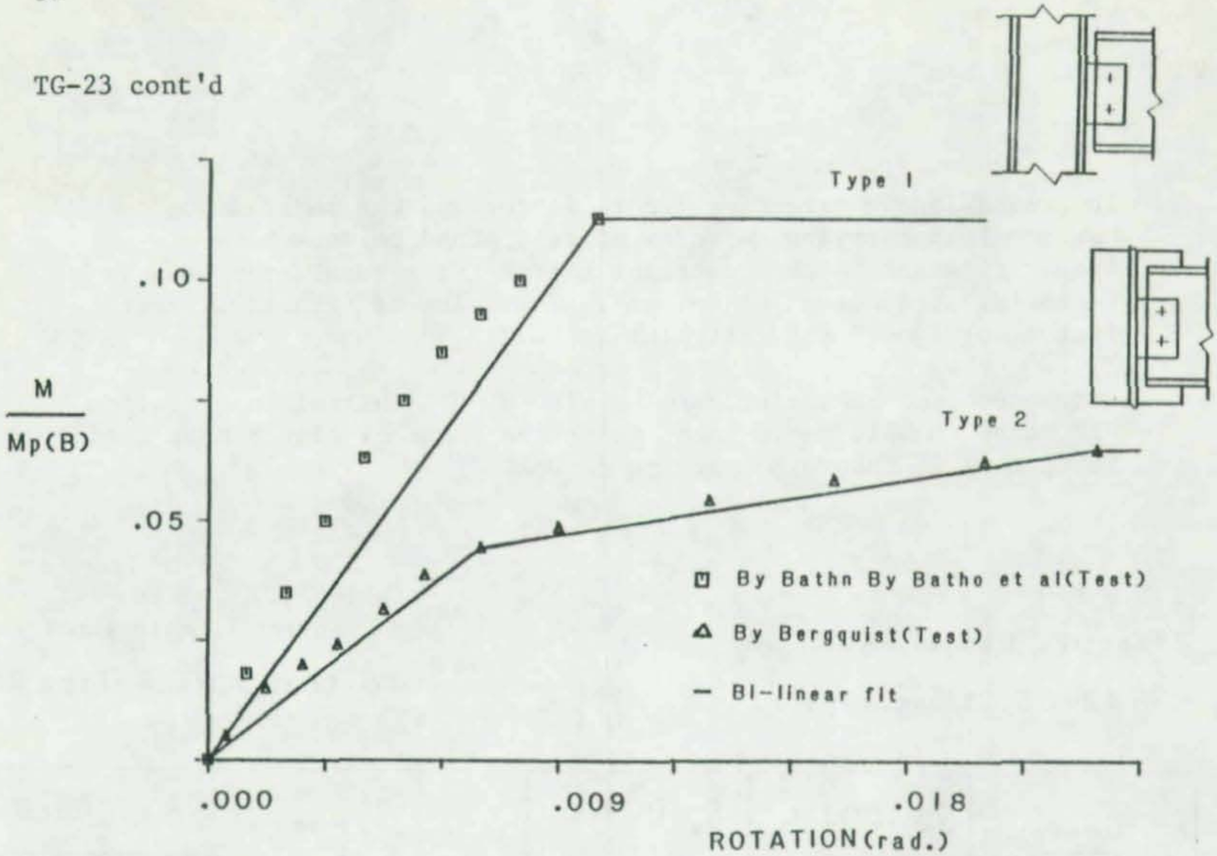

Figure 1: Moment Rotation Data

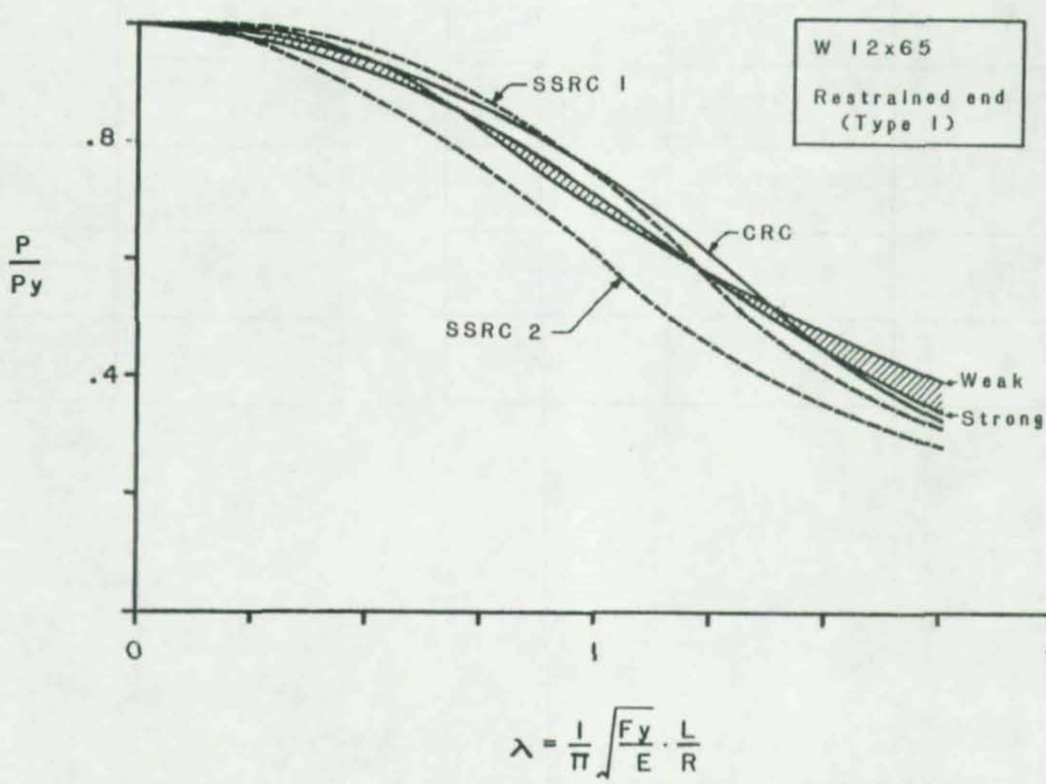

Figure 2: Column Curves

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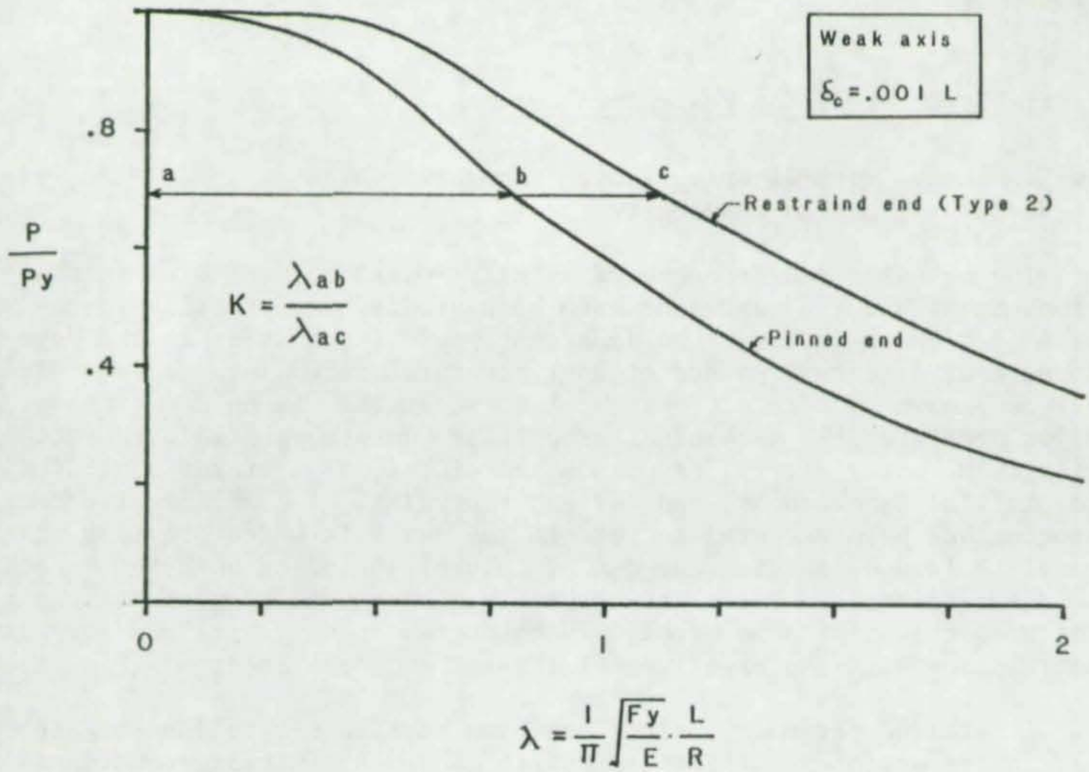


Figure 3: Calculated Column Curves

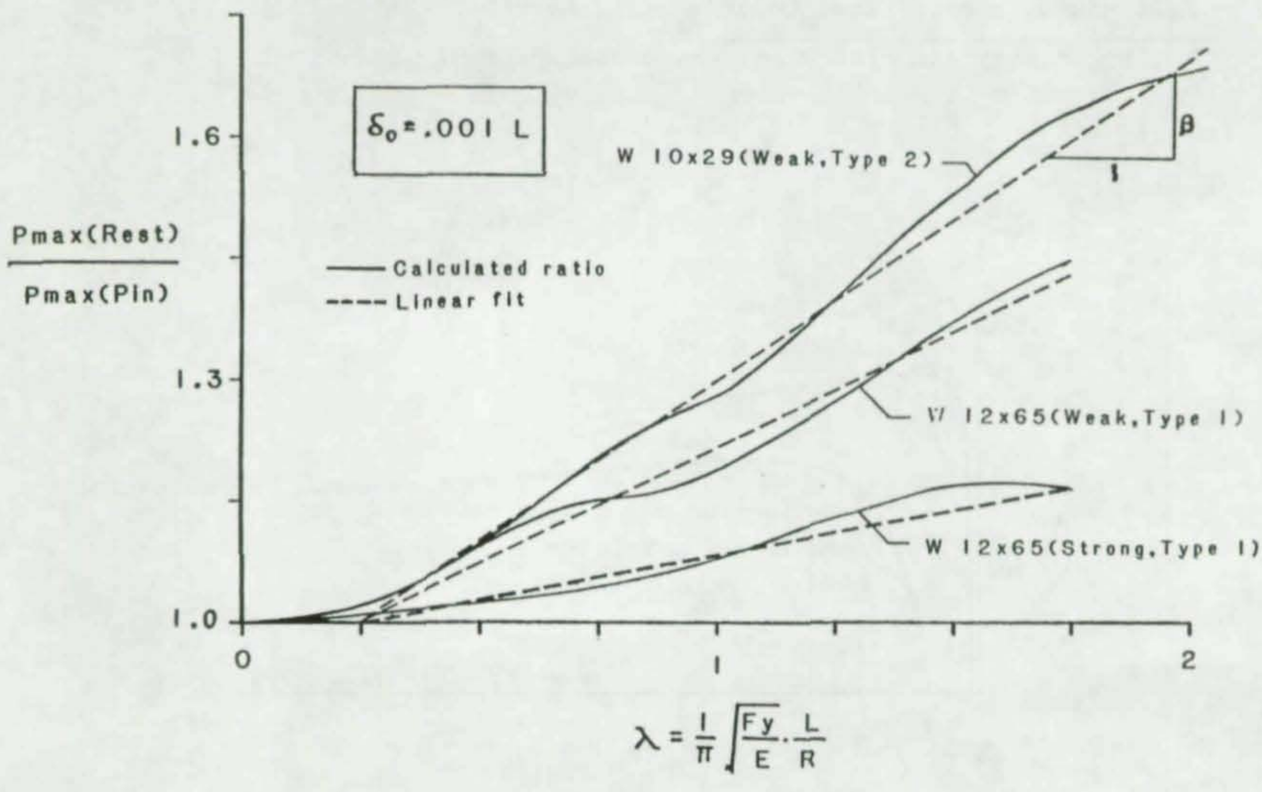


Figure 4: Ratios of Calculated Max. Loads

TG-23 cont'd

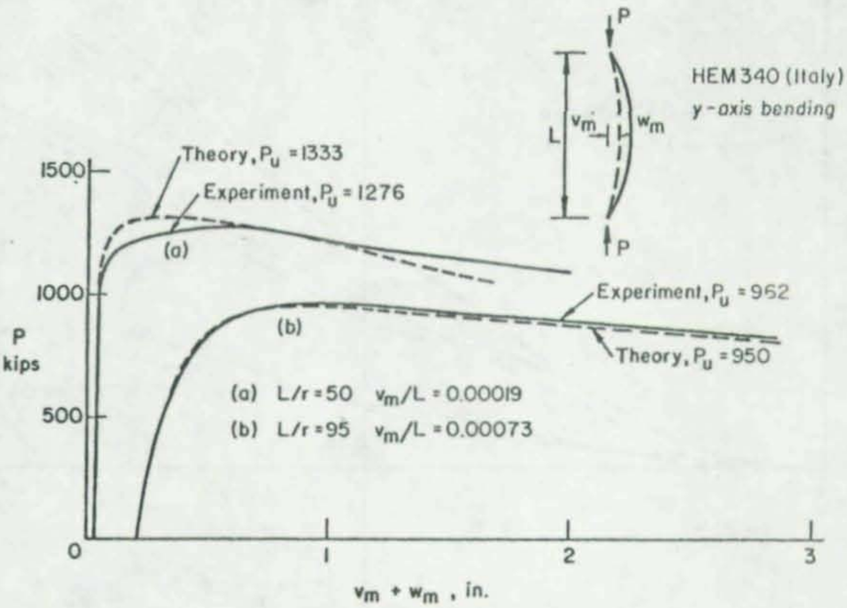
Initially Crooked, End Restrained Columns

Zu-Yan Shen, Tong-Ji University, Shanghai, China
Le-Wu Lu, Lehigh University

The behavior and strength of axially loaded columns with initial crookedness and end restraint have been studied analytically in the case of flexure failure. An important part of the study is the development of a general method of analysis which takes into account almost all the known factors affecting column strength. Among them, the major ones are: (1) mechanical properties and stress-strain characteristics of material, (2) magnitude and distribution of residual stresses, (3) initial crookedness, and (4) end restraint. A comprehensive computer program has been prepared to perform the analysis which provides the complete load-deflection curve of a column, including both the ascending and descending branches. The program has been used to generate analytical predictions of some previously conducted column tests and very close correlation with the experimental results has been observed (Fig. 1).*

A detailed parametric study has been carried out to investigate: (1) the reduction of ultimate strength caused by initial crookedness as a function of the non-dimensional slenderness ratio λ, (2) the increase in strength due to end restraint as a function of λ, (3) the strength of twelve selected columns with different yield and residual stresses, and different axis of bending, and (4) the effect of end restraint on the strength of these columns. The results of the study will be presented in the forthcoming Fritz Laboratory report "Analysis of End Restrained Crooked Steel Columns" (F. L. Report No. 471.2).

* The program can also handle beam-column problems



TASK GROUP 4 - FRAME STABILITY AND COLUMNS AS FRAME MEMBERS

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

Buckling of Three-Story, Single-Span, Rigid Frames

R. L. Ketter, State University of New York at Buffalo

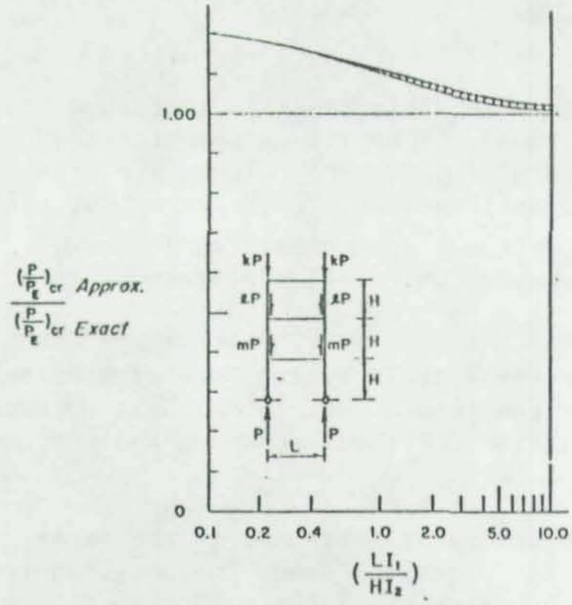
The rigid frame structures with which this paper is concerned are both fixed and pin-ended at their bases. They are of span length "L", and have equal heights between floors of "H". The columns are presumed to have constant unit bending stiffnesses "EI_1" throughout the entire three-story height. This stiffness is not related in any prescribed fashion to that of the beams, which all are presumed to be equal and have values of "EI_2".

The loadings to which the structures are subjected are symmetrical. Moreover, they are applied only at the beam-column junctions. However, all possible combinations and variations of loadings among and between the various floors are allowed.

Three distinct and separate solutions are obtained in the paper. The first of these, the so-called 2nd Order or "exact" method, requires the determination of the lowest eigenvalue of either a 13 x 13 determinant (for the pin-ended structure) or a 15 x 15 determinant (for the fixed-ended structure). These buckling solutions are obtained as a function of "LI_1/HI_2" for the full range of presumed loadings. It is assumed that "LI_1/HI_2" can vary from 0.1 to 10.0. A second set of solutions is obtained presuming the AISC (SSRC) sway-buckling Nomograph. (Actually, the equation governing the Nomograph itself, was used to obtain buckling values.) Finally, an Approximate Solution based upon the "P-Δ" concept, described by the author at the SSRC Annual Meeting that was held in Pittsburgh in 1978, is given.

Ratios of the AISC to the exact solutions and ratios of the Approximate to the exact solutions are given in both tabular and graphical form. (The ranges of variation of these ratios are shown graphically on the attached sheets.) In general, the Approximate Solution <u>overestimates</u> the buckling strength of the structure by 1% to 20%. The AISC buckling solution, on the other hand, for the range of variables presumed, <u>underestimates</u> the true buckling load by a factor of between 1% and 300%.

TG-4 cont'd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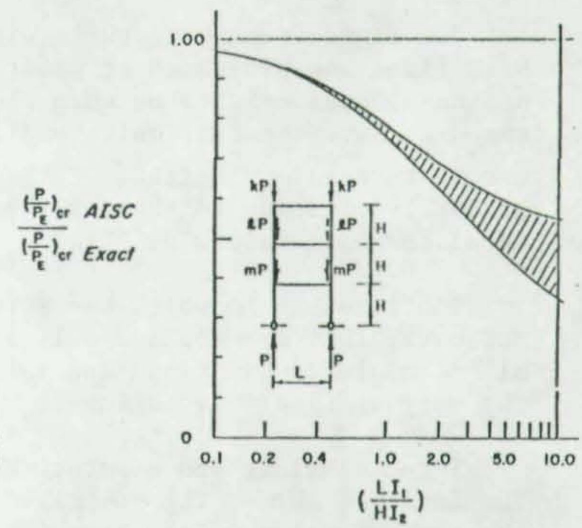
$(\frac{LI_1}{HI_2})$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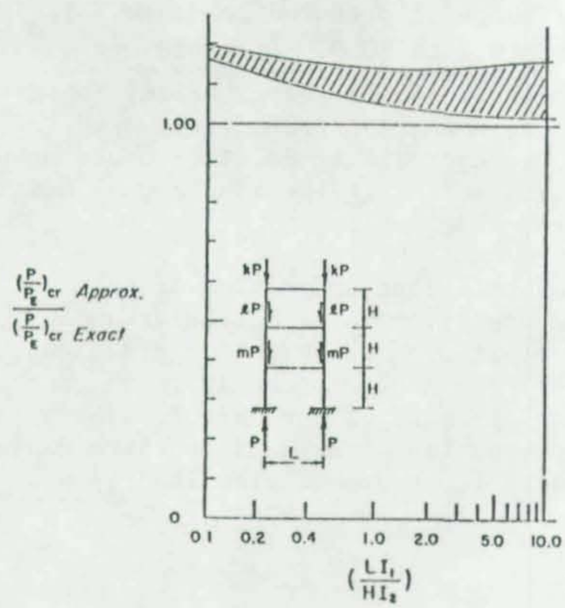
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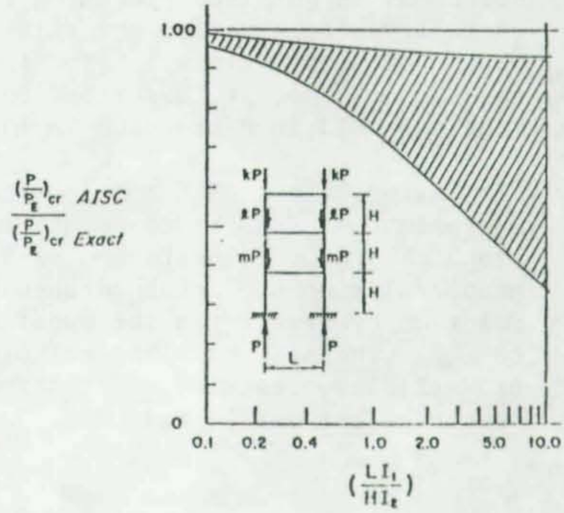
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TG-4 cont'd

Stability of Tall Structures Restrained by Braces with Non-Linear Stiffness

L. F. Estenssoro, Wiss, Janney, Elstner and Associates, Inc.
A. J. Gowens, Goodell-Grivas, Inc.

Classical methods of predicting buckling for perfectly aligned structures result in a bifurcated load displacement curve. Consideration of geometric nonlinearities lead to a load displacement curve which approaches the classical buckling load asymptotically. When elements within the structure yield, the structure's displacement increases and instability may occur at a load below the classical buckling load asymptote.

A theory is developed which uses a Fourier series to represent the deflected shape of the structure. A model of a high rise building is given which consists of columns, beams, internal x-bracing, diaphragms and external springs.

The energy method is used to determine the conditions for equilibrium and stability. Expressions are derived which express the stiffness of the structural elements, their internal work, their change in work for a small displacement, and the external work produced by the loads in terms of the deflection coefficients of the Fourier series. The size of the matrices is determined by the number of terms in the Fourier series and is thus much smaller than the size of matrices for other commonly used stiffness methods of analysis.

The number of terms in the Fourier series may be increased by calculating additional terms for the stiffness matrix. All previously calculated elements are unchanged and reused. No wasted calculations are encountered. Elements may be deleted or added to the structure by subtracting or adding their stiffness to the global stiffness matrix of the structure.

This formulation is quite suitable for use in the calculation of stability of structures with nonlinear bracing elements. Since the stiffness matrix is small, it can be inverted many times as many elements yield, without a great deal of computational effort.

When elements yield, the softened structure is restrained by a fictitious force which accounts for the initially greater stiffness of the structure. Yielding of elements reduces the structure's buckling load and causes the structure to have greater displacements for any increase in loads. As displacements increase, usually additional members of the structure will yield. The structure reaches instability when the yielding causes the buckling load to be reduced to a magnitude less than the load which is acting on the structure. The method also allows the analysis of the structure's load displacement characteristics after instability occurs.

Examples 1 and 2 show the application of the theory to a column supported by one or two evenly spaced springs, respectively. These examples serve to illustrate several aspects of buckling of structures with nonlinear stiffness.

Example 3 is of a ten-story building. The theory illustrates the succession of plastic hinge formation and its effect on the instability load of the structure. The instability load is found to be less than the buckling load of the elastic structure.

TASK GROUP 7 - TAPERED MEMBERS

Chairman, A. Amirikian, Amirikian Engineering Co.

Inelastic Torsional Properties of Tapered Steel Channels

G. C. Lee and K. C. Chang, State University of New York at Buffalo

Channel shapes with linear variation in depth have been used in single-story steel frames, proportioned based on elastic analysis. Some of these frames are presently being examined for their seismic and/or blast load resisting capacity. This calls for some kind of inelastic analysis of the structure. One question is the variation of the longitudinal shear center axis when a tapered channel is loaded into the inelastic range.

A procedure has been developed to determine the inelastic torsional properties and stresses of tapered steel channels subjected to torsion and/or bending. Some important assumptions used are:

1) elastic-perfectly plastic material behavior without residual stresses

2) Von Mises yield criterion

3) single beam theory

4) no elastic unloading in the inelastic range, and

5) no yielding progression through the thickness of the plate elements of the cross section.

Special emphasis in this paper is given to the variations of the shear center axis along the tapered channel, which is modeled as a series of prismatic segments. Numerical examples are carried out by the finite element approach by using updated effective stiffness matrices consistent with loading increments.

TASK GROUP 8 - DYNAMIC STABILITY OF COMPRESSION ELEMENTS

Chairman, D. Krajcinovic, University of Illinois
at Chicago Circle

Dynamic Snap-Through of Shallow Structures

S. M. Holzer and R. H. Plaut, Virginia Polytechnic Institute
and State University

The snap-through instability of shallow structures under dynamic loads is considered. Possible modes of failure (symmetric, asymmetric) and types of snapping (direct, indirect) are discussed. Various stability criteria, such as the Budiansky-Roth criterion and Hsu's sufficient condition, are compared. Loads are classified by their spatial distribution (e.g., concentrated loads, uniformly distributed loads) and their temporal variation (e.g., impulse loads, triangular pulse loads, step loads of infinite duration).

Several types of shallow structures are treated in the literature, such as arches, trusses, and shells. Both continuum and discrete mathematical models are utilized, and the material behavior is assumed to be elastic, viscoelastic, or elastic-plastic. Common techniques used for numerical integration over time are Newmark's beta-method, Houbolt's procedure, and the central difference method.

The influence of damping on the critical load is examined. Attention is given to the effect of imperfections, such as initial displacements, and the effect of boundary conditions. Recent work involving multiple, independent, dynamic loads is also reviewed.

Stability of Circular Cylindrical Structures Subject to Fluid Flow

S. S. Chen, Argonne National Laboratory

Abstract

Many structural components consist of long, slender circular cylinder, such as offshore structures, pipelines, heat exchanger tubes, and nuclear fuel rods. These components are subjected to fluid flow. The fluid flow represents a source of energy that can cause dynamic instability. The objective of this paper is to present the stability of circular cylinders subjected to parallel or cross flow.

Parallel flow may be classified as internal flow and external flow. Instability of cylinders induced by parallel flow has been the subject of numerous investigations. Several intriguing phenomena have been studied in detail. Some general conclusions can be made regarding the stability.

TG-8 cont'd

1. In general, flow-induced instability occurs at relatively high flow velocity. 2. At high flow velocities, cylinders may be subjected to buckling and/or dynamic instability depending on system parameters. 3. Several characteristics associated with nonconservative systems also exist in parallel flow, such as the destabilizing effect of damping and structural stiffness, and a sharp change of system behavior with a small change of a parameter.

Several types of dynamic instability for circular cylinders across a flow have been discussed in literature: galloping, flutter, and coupled galloping-flutter instability. The general characteristics of the different types of instability will be presented and stability criteria will be given.

Some of the general design guidelines which can be applied in design analysis to avoid detrimental instability will be presented.

Dynamic Response and Stability of Rigid Frames Subjected to Time-Dependent Axial Forces

C. H. Tay, National University of Singapore
C. K. Wang, University of Wisconsin-Madison

The dynamic response of rigid frames that are subjected to time-dependent axial forces and the boundary frequencies of the periodic axial forces between stable and instable regions, are obtained by treating the axial forces as piece-wise time dependent.

First, the effect of the presence of constant axial forces within each step on the natural frequency is solved by the distributed mass method, for which the dynamic stiffness matrix is obtained from the exact solution of the differential equation. Then a computer program is written to produce continuous plotting of the time-dependent response. It has been found that there is significant reduction in the natural frequency of a rigid frame when there are axial forces in the constituent members.

The boundary frequencies of the periodic axial forces for the region of dynamic instability are obtained by both the Bolotin's direct method and the characteristic equation method. In the first approach the boundary frequencies are the natural frequencies at which the mass of the structure is reduced to one-fourth of its real magnitude and the axial forces take the values of

$P_o \pm \frac{1}{2}P_t$, where P_o is the constant part and P_t is the time-variable part of the periodical axial forces. In the second approach, the boundary frequencies are obtained by a trial and error procedure so that the characteristic values are equal to positive and negative unity in the solution of a second-order ordinary differential equation. It has been found that the width of the instability region widens linearly with increase in P_t for any constant value of P_o from a point when P_t is zero. This width also increases with increase in P_o for any constant value of P_t.

TASK GROUP 18 - UNSTIFFENED TUBULAR MEMBERS

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

Post Buckling Behavior of Tubular Beam-Columns

D. R. Sherman, University of Wisconsin-Milwaukee

Tubular Beam-Column Test Program

An experimental program of over a hundred beam-column tests was undertaken to determine the conditions at the ultimate load and subsequent behavior. Some of the specific questions addressed in the program include the ultimate load and corresponding strain or rotation; the rate of load decay; energy capacity in monotonic or cyclic tests; shape of the hysteresis curve; and degredation of the cyclic curve. These characteristics were determined for various loading conditions, D/t ratios and lengths.

Two major categories of portal and strut members were tested. Portals are members with constant axial force and variable lateral displacement while struts are fixed end members with constant lateral load and variable axial load or displacement. The three D/t ratios included in the program were 37, 49 and 77. Tubes in all D/t ratios were tested with short lengths corresponding to L/D of 20 for the struts and a length to give the same moment gradient in the portals. For D/t of 49 only, an additional set of tests were conducted on longer tubes with L/D of 50. The test specimens were ERW tubing which had rounded stress-strain curves with little or no strain hardening. Yield strengths by 0.2% offset varied from 42 to 53 ksi.

It was determined that both portal and strut strengths followed the same interaction relation which can be approximated by

$$\frac{1}{1 - P/P_E} \frac{Q}{Q_P} = \cos \frac{\pi}{2} \frac{P}{P_{cr}}$$

where P is the ultimate axial load for the beam-column

P_{cr} is the critical column load for an axially loaded member

P_E is the larger of P_{cr} or $\pi^2 EI/L^2$

Q is the ultimate lateral load for the beam-column

Q_p is the lateral load for a mechanism failure if there is no axial load

TG-18 cont'd

For short members, this relation is conservative except for members toward the beam end of the interaction relation with D/t of 77. In this case, inelastic local buckling reduced the member strength. For long members, local buckling may be a factor in the middle range of the interaction curve, especially for higher strength materials.

Strains are an important factor in determining the influence of local buckling. In members with primarily axial loads, failures occur at strains on the order of the yield strain. On the other hand, in flexural members strains more than ten that large are required before mechanism loads are reached. Since inelastic local buckling is a function of strain level, it should be expected to reduce the moment capacity at a given D/t before it will reduce the axial strength.

From a study of the load-distortion data, algorithms were developed which have the general forms shown in Figures 1-4. The normalized curves for struts are independent of the amount of axial load. In the monotonic curve of Figure 1, the length of the plateau decreases with D/t and member length while the absolute values of the decay slopes increase with these variables. The cyclic strut behavior is contained in the envelope of Figure 2 which shifts to the right if the member is stretched in tension yielding. The ordinate at the break in the slopes of the extension portion of the envelope varies from .5 of the yield load for low D/t to .25 for the thinnest tubes. The cyclic tests also indicated that the ratio of peak compression in a cycle to the original peak can be approximated by R_2^n where R_2 is the ratio for the second peak and n is the cycle number.

In the monotonic portal behavior of Figure 3, the parameters of the curve are functions of D/t, length and the amount of axial load. The proportional limit is relatively constant at .85 for low axial loads but decreases rapidly as critical axial column conditions are approached. The critical inelastic rotation also depends on the degree that the member approaches the critical column condition. For low axial loads, D/t is the key parameter but for high axial loads or long members, the magnitude of axial has the greatest influence on θ_p.

The moment decay can be described by a logarithmic reduction factor

$$R = \exp(-C\theta_d/\theta_p)$$

applied to the ordinates of the bilinear relation. The cyclic algorithm in Figure 4 also uses the same reduction factor, with θ_d being the accumulation of absolute rotation after peak moments are reached.

The results of this study have been used in the formation of algorithms for member elements of the DYNAS program used in the inelastic dynamic analysis of offshore towers.

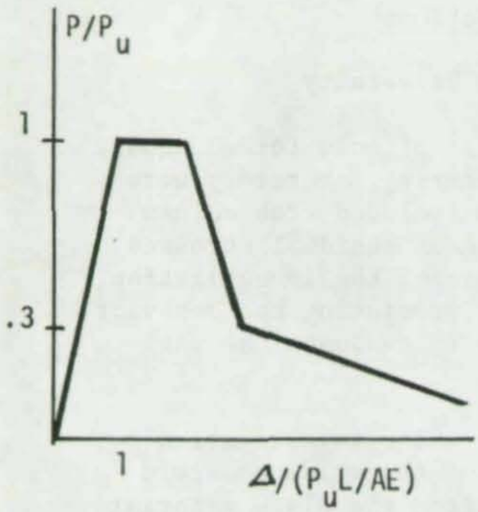


FIG. 1 - MONOTONIC STRUT

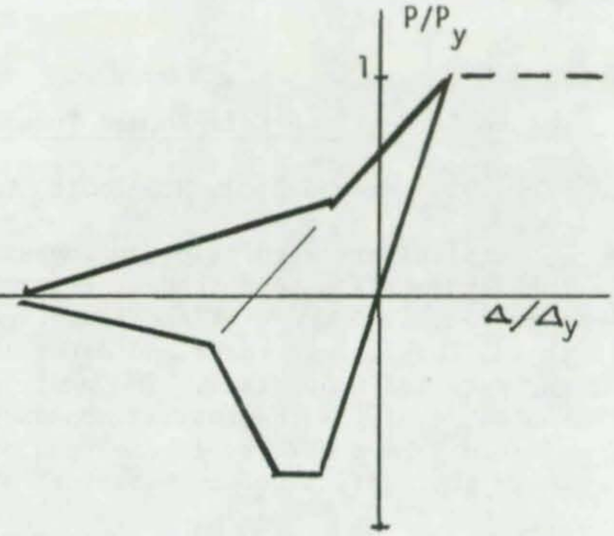


FIG. 2 -CYCLIC STRUT

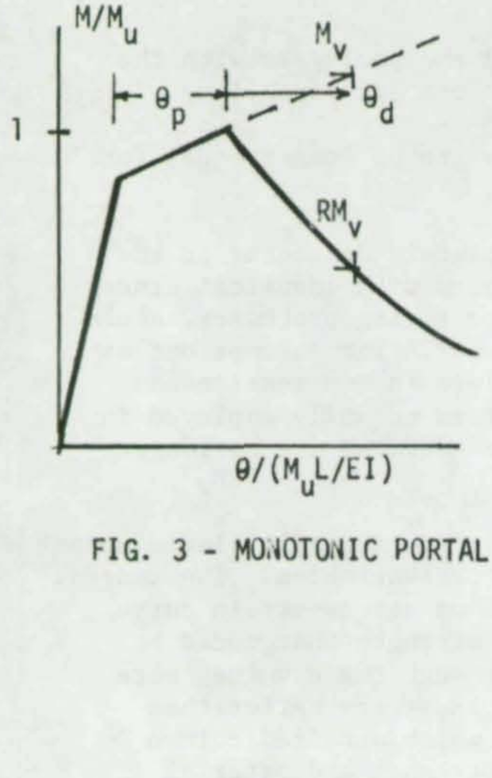


FIG. 3 - MONOTONIC PORTAL

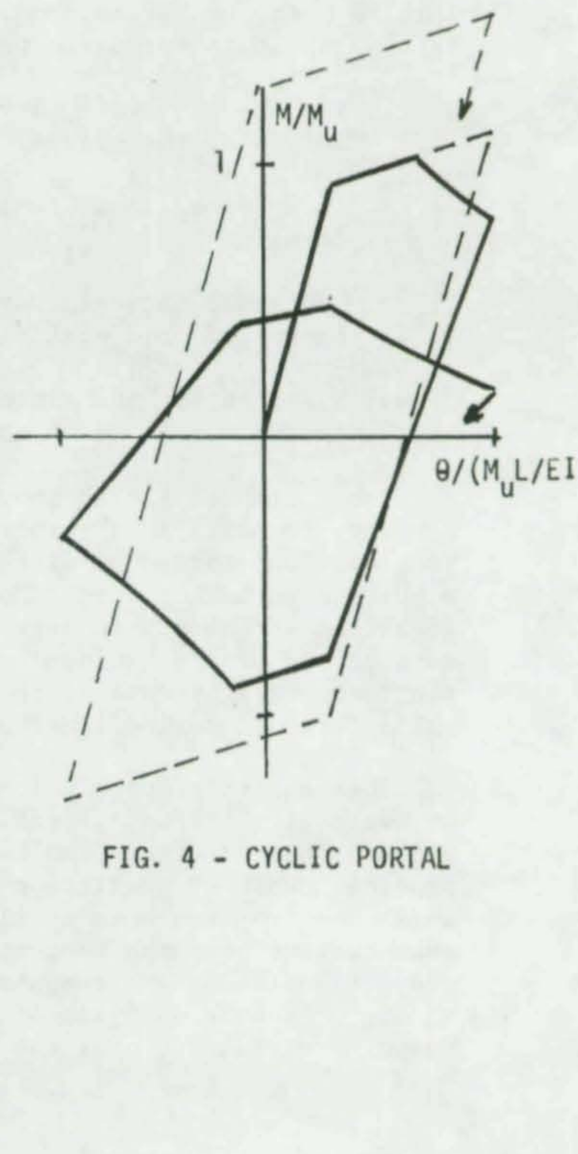


FIG. 4 - CYCLIC PORTAL

Column Strength of Cold-Formed Tubular Sections

R. G. Slutter and R. J. McDermott, Lehigh University

Preliminary results of an investigation of cold formed square tubular sections conducted at Fritz Engineering Laboratory were reported. The scope of the investigation included stub column tests, long column tests and determination of residual stresses and material properties. The basic purpose of the investigation was to develop a theoretical approach for predicting the behavior of cold formed members in compression and to evaluate the variables that affect column behavior.

The production of the tubular members under investigation involved welding, local stress relieving of the welds and cold forming. The member as produced differs from the plate material from which it is formed in the following aspects:

1. The stress-strain curve no longer exhibits a sharp yield but is of the gradual yielding type.

2. Three dimensional residual stresses are produced in cold forming.

3. The yield strength varies around the perimeter with the amount of cold work required to form each area.

4. Yield in tension and compression are no longer equal for the flat regions of the member.

The variables listed above are completely dependent on the forming process. As a result, two members with identical cross sections, but formed by different manufacturing processes, could result in rather different column behavior. The assumptions of idealized stress-strain curve, equal yield in compression and tension and simple residual stress pattern normally employed in the theoretical treatment of hot rolled sections are no longer valid for cold formed sections.

The results of stub column tests provide the only simple means of evaluating the net affect of all of the variables. The tangent modulus load obtained from the stub column stress-strain curve provides the best prediction of column strength that could be developed from the data available. Although these values were conservative compared to test results, they were better than predictions made by a computer program which obtained column strength from an analysis of residual stresses and material property variations over the cross section.

TASK GROUP 22 - STIFFENED CYLINDRICAL MEMBERS

Chairman, C. D. Miller, Chicago Bridge & Iron Company

Preliminary Results of Tension and Collapse Tests on Fabricated Steel Cylinders

C. D. Miller, Chicago Bridge & Iron Company

In 1980 a test program was funded by 11 companies associated with the offshore industry to determine the buckling strength of unstiffened and ring stiffened cylinders under combinations of uniform external pressure and axial tension loads. This program was initiated because no test data was known to exist for fabricated cylinders under this combination of loads. Existing data was from small diameter tubes with maximum D/t values of 27. The objective of the tests was to provide data for verifying or modifying the current American Petroleum Institute (API) design rules for fixed offshore platforms[1]. A selection of test specimens was based upon the analysis of existing test data given in Ref. 2.

There were six groups of test speciments (1 group unstiffened and 5 groups with ring stiffeners) with the same geometries and materials. The groups of specimens were designed to fail at different stress levels, when loaded by radial pressure only, ranging from $0.3\sigma_y$ to $0.9\sigma_y$, where σ_y is the yield stress of the material. The yield strength and stress-strain properties of the material before fabrication were determined for each plate in both the longitudinal and hoop directions of the test specimens. Initial out-of-roundness measurements were made of each specimen. Only three of the specimens were within API Spec 2B[4] tolerances. The effects of out-of-roundness are being studied. A total of 22 specimens were tested with D/t ratios ranging from 32 to 96. The test specimens were fabricated from ASTM A36 and A633 Gr C materials with minimum specified yield strengths of 36 ksi (248 N/mn^2) and 50 ksi (345 N/mm^2) respectively. The axial tension load was applied by jacks to a frame and the cylinder together with the jacking frame was placed inside a pressure vessel for application of external pressure. The tests were performed at Southwest Research Institute and the results are reported in Ref. 3.

The theoretical interaction curve for elastic buckling is a straight line. This line is a continuation of the line which passes through the points σ_{re} and σ_{he}. These points correspond to the theoretical buckling stresses in the hoop direction for cylinders under radial pressure only ($\sigma_x = 0$) and hydrostatic pressure ($\sigma_x = 0.5\sigma\theta$). Von Mises' distortion energy theory has been found to be in closest agreement with test results when buckling is not a consideration.

TG-22 cont'd

The test data which was included in the study made in Ref. 2 was all from small diameter tubes. These tests showed that tensile loads decreased the buckling stresses in the hoop direction. For each group of cylinders with a given value of F_{rc} where F_{rc} is the buckling stress for a fabricated cylinder under radial pressure only, it was found that the interaction curve described by the test data was a straight line (see Eq. 1) until the line intersected the interaction curve given for yield failures. The test data then followed the yield failure curve. To obtain a lower bound on test data, Von Mises' equation was modified by a factor K as shown in Eq. 2. The theoretical value for K is 1.0 compared with a value of 1.5 based upon tests.

(a) Elastic Buckling

$$\frac{\sigma_\theta}{F_{rc}} \leq \left(1 - 0.25\frac{\sigma_x}{\sigma_y}\right) \tag{1}$$

(b) Inelastic Buckling

$$\left(\frac{\sigma_x}{\sigma_y}\right)^2 - K\frac{\sigma_x}{\sigma_y} \cdot \frac{\sigma_\theta}{\sigma_y} + \left(\frac{\sigma_\theta}{\sigma_y}\right)^2 \leq 1.0 \tag{2}$$

The interaction equation given in the current API rules and by Eq. 3 is a further modification of Eq. 2, with σ_θ/F_{hc} substituted for σ_θ/σ_y and K = 0.6. This equation was selected because it was found to be conservative with respect to available test data.

(c) API RP 2A

$$\left(\frac{\sigma_x}{\sigma_y}\right)^2 + 2\mu\left(\frac{\sigma_x}{\sigma_y}\right)\left|\frac{\sigma_\theta}{F_{hc}}\right| + \left(\frac{\sigma_\theta}{F_{hc}}\right)^2 \leq 1.0 \tag{3}$$

where $\mu = 0.3$

σ_x, σ_θ = maximum applied stress in longitudinal and hoop directions and
F_{hc} = predicted buckling stress for fabricated cylinder under hydrostatic pressure. ($F_{hc} \approx F_{rc}$ for the geometries tested)

The interaction curves given by Eqs. 1-3 are compared with the theoretical curves in Fig. 1. The results of the test data are compared with Eqs. 1-3 in Fig. 2. Equation C2.5.4-1 of Ref. 1 was used in determination of F_{hc} when the allowable out-of-roundness given by Ref. 4 was exceeded. The test data is in fairly good agreement with Eqs. 1 and 2 except for test points near the curve given by Eq. 2, such as tests 3D and 4D which were below the predicted values. Equation 3 is a lower bound on all tests except 3A, 6A, and 6B. The effect of imperfections is apparently greater than predicted for test 3A. Group 6 was identical to group 1 except for materials. The yield stress values were 59 ksi and 39 ksi, respectively.

TG-22 cont'd

The results of these tests will be used as a basis for an additional set of tests to further evaluate the effects of initial imperfections, residual stresses and material properties on the buckling strength. These tests will be funded by API and performed at SWRI in 1981.

Acknowledgements

The present tests were sponsored by Amoco Production Co., Brown and Root, Chicago Bridge & Iron Co., Cities Service Oil Co., Exxon Production Research Co., Gulf Research and Development Co., McDermott, Mobil Research and Development Corp., Shell Oil Co., Southwest Research Institute, and Union Oil Co.

References

1. American Petroleum Institute, "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," API RP 2A, Twelfth Edition, 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. Marlow, R.S., "Collapse Performance of Fabricated Cylinders Under Combined Axial Tension and External Pressure,"SWRI Project No.03-6139, Final Report, Jan. 1981.

4. American Petroleum Institute, "Specification for Fabricated Structural Pipe," API Spec 2B, Third Edition, November 1977.

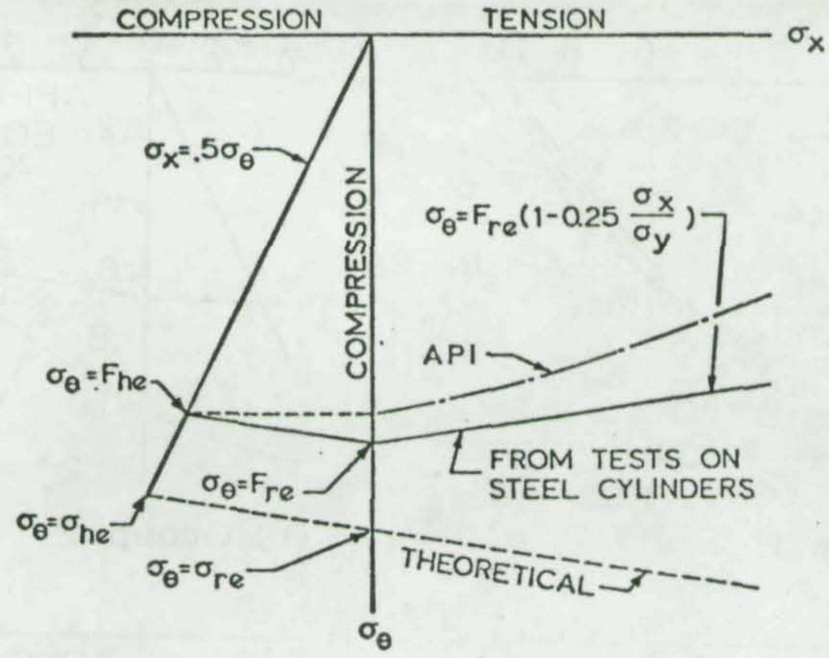
TABLE 1 - SUMMARY OF TESTS ON FABRICATED UNSTIFFENED AND RING STIFFENED CYLINDERS UNDER COMBINATIONS OF AXIAL TENSION AND EXTERNAL PRESSURE TAKEN FROM REF. 3

SPEC. NO.	MAT'L ASTM NO.	R in	t, t_r in	L_B in	L in	h_r in	E_x ksi $x10^{-3}$	E_θ ksi $x10^{-3}$	σ_{yx} ksi	$\sigma_{y\theta}$ ksi	γ_c	e_{max} in	L_c in
1A	A36	8.73	.524	142.0	26.50	2.445	29.1	28.0	38.9	37.6	1.583	.018	8.818
1B	A36	8.72	.524	140.6	26.38	2.478	29.1	28.0	38.9	37.6	2.494	.018	8.817
1C	A36	8.75	.518	140.6	26.50	2.476	28.4	29.2	39.1	39.2	.711	.038	12.472
1D	A36	8.75	.518	140.5	26.50	2.510	28.4	29.2	39.1	39.2	.950	.014	8.818
1E	A36	8.76	.528	141.4	26.50	2.450	29.1	29.8	38.8	38.6	2.333	.023	8.818
2A	A36	11.77	.504	143.5	54.00	2.658	30.4	29.3	37.8	38.2	1.292	.044	12.536
2B	A36	11.76	.504	143.4	54.25	2.898	30.4	29.3	37.8	38.2	1.317	.072	12.536
2C	A36	11.75	.504	143.0	54.00	2.668	30.4	30.6	37.8	38.2	1.383	.115	16.711
2D	A36	11.78	.496	143.0	54,00	2.659	30.4	30.6	35.8	35.6	1.808	.079	12.536
3A	A36	11.80	.376	144.0	69.00	2.246	28.8	29.8	42.2	41.3	1.475	.057	12.536
3B	A36	11.54	.376	144.0	69.50	2.229	28.8	29.8	42.2	41.3	.458	.020	12.535
3C	A36	11.79	.376	144.0	69.00	2.319	28.8	29.7	42.2	41.3	1.992	.056	12.535
3D	A36	11.81	.382	143.6	69.50	2.286	31.5	29.7	40.1	39.9	1.538	.036	12.536
4A	A36	11.85	.246	144.0	60.00	1.847	30.4	29.2	39.5	40.3	2.917	.075	12.535
4B	A36	11.85	.246	144.0	60.00	1.831	30.4	29.2	39.5	40.3	1.988	.054	12.535
4C	A36	11.85	.252	144.2	60.00	1.950	29.2	29.[illegible]	43.3	42.7	1.196	.039	12.535
4D	A36	11.86	.252	144.0	58.75	1.879	29.2	29.6	43.3	42.7	1.512	.045	12.535
5A	A36	8.78	.374	140.8	140.80	.000	29.1	29.0	46.4	45.5	2.050	.049	12.471
5B	A36	8.80	.374	141.4	141.40	.000	29.1	29.0	46.4	45.5	2.350	.046	12.472
5C	A36	8.80	.379	141.2	141.20	.000	30.0	30.6	46.3	46.6	2.128	.059	12.472
5D	A36	8.80	.379	141.0	141.00	.000	30.0	30.6	46.3	46.6	2.461	.055	12.472
6A	A633 GRC	8.75	.534	140.4	26.75	2.400	29.4	30.8	58.9	59.4	1.456	.025	8.818
6B	A633 GRC	8.78	.534	141.3	26.00	2.365	29.4	30.8	58.9	59.4	1.994	.019	8.819
6C	A633 GRC	8.76	.533	141.3	26.75	2.422	30.5	29.5	58.9	58.4	1.161	.034	8.818

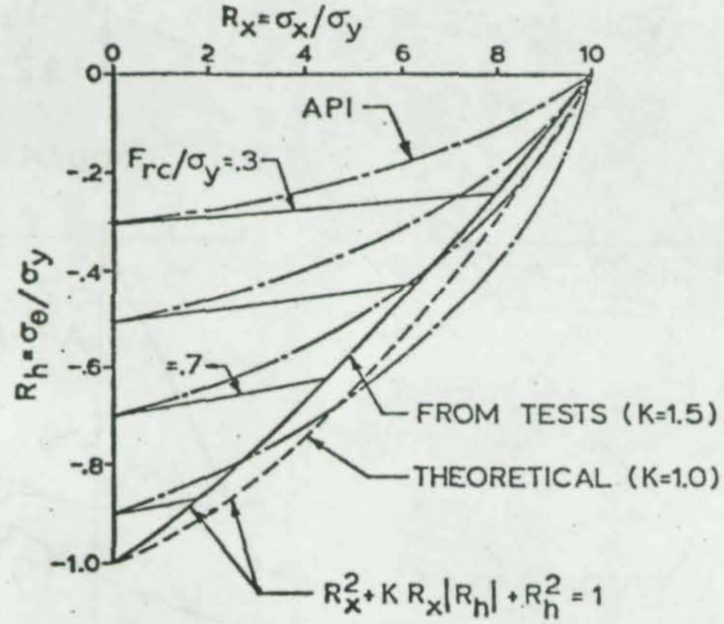
SPEC. NO.	$\frac{R}{t}$	$\frac{L}{\sqrt{Rt}}$	T kips	σ_x ksi	p psi	σ_θ ksi	$\frac{\sigma_x}{\sigma_{yx}}$	$\frac{\sigma_\theta}{\sigma_{y\theta}}$	$\frac{F_{xc}}{F_{yx}}$	$\frac{F_{rc}}{F_{y\theta}}$	$\frac{F_{hc}}{F_y}$	α_x	α_θ
1A	16.7	12.4	-16.	-0.6	2100.	36.0	-0.014	.958	.987	.865	.855	.207	.748
1B	16.6	12.3	204.	7.1	1950.	33.4	0.183	.888	1.000	.847	.837	.343	.684
1C	16.9	12.4	1100.	38.6	0.	0.0	0.988	.000	1.000	.875	.865	.359	.800
1C*L	16.9	12.4	-641.	-22.5	2480.	43.1	-0.576	1.099	1.000	.875	.865	.359	.800
1C*G						24.7	-0.576	.630	1.000	.787	.787	.525	.800
1D	16.9	12.4	416.	14.6	1900.	33.1	0.374	.843	1.000	.875	.865	.359	.800
1E	16.6	12.3	699.	24.1	1450.	24.8	0.621	.642	1.000	.859	.849	.353	.695
2A	23.3	22.2	-385.	-10.3	1150.	27.4	-0.273	.718	1.000	.650	.645	.349	.773
2B	23.3	22.3	254.	6.8	1000.	23.8	0.181	.624	1.000	.649	.644	.349	.770
2CL	23.3	22.2	574.	15.4	925.	22.0	0.408	.577	1.000	.657	.652	.349.	.765
2CG						12.9	0.408	.339	1.000	.571	.571	.349	.683
2D	23.7	22.3	989.	26.9	550.	13.3	0.753	.375	1.000	.657	.652	.323	.731
3A	31.4	32.8	-206.	-7.4	600.	19.1	-0.175	.463	.971	.506	.498	.381	.757
3B	30.7	33.4	103.	3.8	675.	21.0	0.090	.509	.974	.537	.528	.381	.800
3C	31.3	32.8	472.	16.9	575.	18.3	0.402	.443	.971	.479	.471	.381	.718
3D	30.9	32.7	805.	28.4	400.	12.6	0.709	.315	.987	.527	.519	.366	.752
4A	48.2	35.1	-99.	-5.4	275.	13.4	-0.138	.332	.916	.268	.264	.370	.658
4B	48.2	35.1	199.	10.8	290.	14.1	0.275	.350	.916	.293	.288	.370	.718
4C	47.0	34.7	422.	22.5	275.	13.1	0.520	.307	.908	.316	.311	.394	.781
4D	47.0	34.0	609.	32.5	160.	7.6	0.750	.178	.908	.312	.307	.394	.754
5A	23.5	77.7	-12.	-0.6	600.	14.4	-0.012	.317	1.000	.242	.242	.422	.714
5B	23.5	78.0	341.	16.5	560.	13.4	0.356	.296	1.000	.234	.234	.422	.693
5C	23.2	77.3	584.	27.9	525.	12.5	0.602	.267	1.000	.252	.252	.433	.708
5D	23.2	77.2	775.	37.0	400.	9.5	0.798	.204	1.000	.244	.244	.433	.686
6A	16.4	12.4	-18.	-0.6	2600.	43.9	-0.010	.739	1.000	.790	.779	.561	.759
6B	16.4	12.0	634.	21.5	2200.	37.3	0.365	.627	1.000	.784	.772	.561	.718
6C	16.4	12.4	1021.	34.8	2050.	34.7	0.591	.594	1.000	.791	.780	.551	.785

*INDICATES RETEST

NOTE: SPECIMENS 1C AND 2C FAILED BY GENERAL INSTABILITY BOTH LOCAL BUCKLING AND GENERAL INSTABILITY STRESSES ARE SHOWN.



(a) Elastic Buckling



(b) Elastic and inelastic buckling

FIG. 1 Comparison of interaction curves for cylinders under combined axial tension and hoop compression.

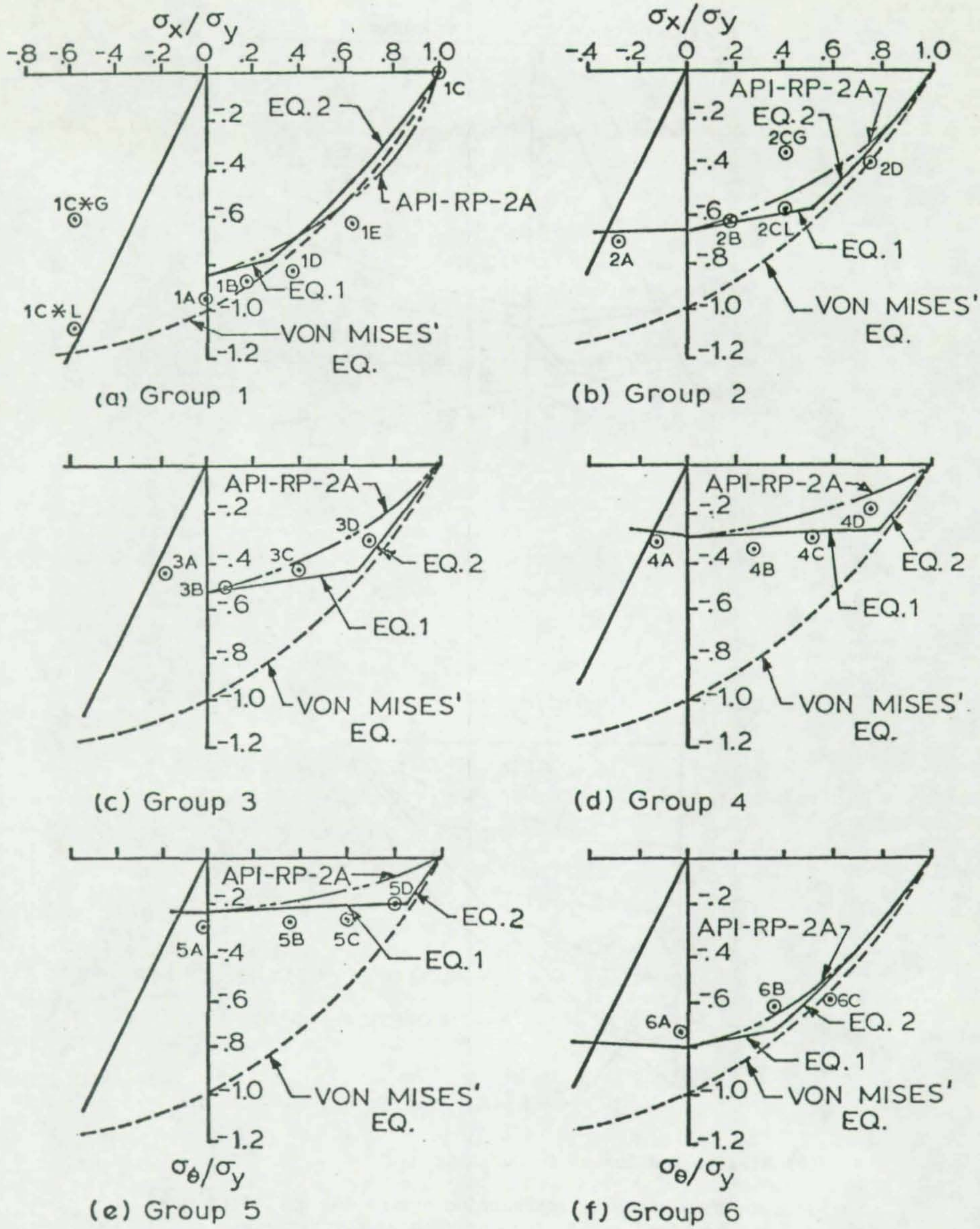


FIG 2. COMPARISON OF TEST DATA FROM FABRICATED CYLINDERS WITH INTERACTION CURVES.

OTHER RESEARCH REPORTS

On Inelastic Analysis of Steel Structures Subjected to Nonproportional Loading

N. T. Tseng and G. C. Lee, State University of New York at Buffalo

Nonlinear behavior of structures is generally the result of geometrical and/or material nonlinearlities. Plastically designed structures have been based on the assumption of proportional loading, or of prescribed variable repeated loading. When both nonlinearities must be taken into consideration, the analysis procedure is quite tedious. Numerical methods such as the finite element approach are usually used.

For structures subjected to nonproportional loading, there is no satisfactory approach to determine their nonlinear responses. The subject has been studied in recent years by many investigators, both analytically and experimentally. One of the most difficult areas is the formulation of the material constitutive law to satisfactorily and systematically describe the multiple state of stress in the inelastic range.

In this study an attempt is made to refine existing plasticity models and to develop a finite element procedure to determine the inelastic behavior of steel structures subjected to nonproportional loading with both geometrical and material nonlinearities. Simple numerical examples are given.

Web Buckling Under Cyclic Loading

E. P. Popov and K. Hjelmstad, University of California, Berkeley

The need for designing building frames that are both stiff under frequently occurring loads and ductile under rare overloads such as may occur during a severe earthquake has aroused interest in the concept of eccentrically braced steel frames. Any bracing scheme in which the axial force in a brace is transferred to a column through shear in a beam can be classified as an eccentrically braced frame. The region of the beam where this transfer of force takes place is called a link and it is within such regions that most of the inelastic action in the frame occurs. During an earthquake such links are subjected to severe cyclic load reversals.

Frames having short active links will generally have greater lateral stiffness than those having long links. Further, a short link will dissipate energy primarily through plastic straining of the web plate in shear. The high lateral stiffness of the frame that is achieved with the use of links places high ductility demand on these links. The large inelastic shear distortions which may occur in short links, coupled with the cyclic nature of earthquake type loadings can lead to problems of web instability in the links.

OTHER RESEARCH REPORTS cont'd

The problem of web buckling in plate girders having thin webs has been thoroughly investigated and procedures are available for evaluating the post-buckling capacity of these elements [1,3]. These procedures only apply where elastic web buckling precedes inelastic action and the effects of material strain hardening are usually neglected. Also, testing in this area has been confined to cases involving monotonically applied loads. While certain features of the behavior of the thin webbed beams are similar to those of the thicker ones, such as the formation of a tension field in the post-buckling range of behavior, the two problems have fundamental differences which prevent the thin web analysis from being applicable to the thicker webs, just as the Euler formula does not apply to columns which buckle inelastically.

In order to better understand the buckling phenomenon in active links subjected to cyclic loading in the inelastic range, an experimental program was designed and a number of tests on full sized links have been performed. For the purpose of testing, a typical link was isolated from the rest of the structure as shown in Fig. 1. The specimens were fully welded to thick end plates and bolted to the test fixture shown in Fig. 2. The testing system was designed to impart shear and bending to the specimen, conforming to the following modeling assumptions: (1) No axial forces develop in a link. (2) Reverse curvature bending with equal end moments will exist at ultimate load. (3) Warping due to shear distortion is restrained at both ends of the element. These assumptions are believed to be a reasonably accurate simulation of the conditions found in links of prototype structures.

While the ultimate goal of the research is to make inferences about the behavior of active links under general reversing loads, the loading program was kept simple so as not to mask the fundamental behavior of the links. All of the specimens were subjected to quasi-statically applied cycles of displacement in the plane of the web. The loading program consisted of sequentially increasing the relative displacement between the ends of a specimen. Beginning with a cycle of maximum end displacements of $\pm$ 0.5 in., the maximum displacements for the following cycles were performed in pairs increasing from $\pm$ 1.0 in., to $\pm$ 1.5 in., to $\pm$ 2.0 in., etc. until failure defined by a substantial loss of load capacity of a specimen had occurred.

The W18x40 wide flange beam section was chosen for the initial test program to model in full size, sections typically used in building construction. The specimen length was set at 28 in. to insure that the web would yield in shear prior to any significant inelastic action in the flanges. The average tensile yield strength of the material was approximately 36 ksi. Initially the material exhibited a characteristic yield plateau followed by strain hardening.

The first series of tests included in the program consisted of five of these specimens stiffened transversely in the web region to varying degrees. The results described below are from the two bounding cases in this series.

OTHER RESEARCH REPORTS cont'd

Specimen 1 was constructed without stiffening the web region. Consequently, it had a single panel with an effective size of 28 x 16.85 in. Fig. 3 shows the action-deformation relationship for this specimen. Buckling occurred early in the second cycle. Large amplitude buckling on the order of three inches caused substantial load carrying capacity deterioration in the later cycles. The failed specimen is shown in Fig. 4.

Specimen 4, with three equally spaced stiffeners, had four 7 x 16.85 in. panel zones. The deformations in this specimen were primarily caused by inelastic shearing strains throughout most of the test. Buckling was of small amplitude and did not occur until the ninth cycle of the test at 2.5 in. amplitude. The action-deformation relationships and the failed specimen are shown in Figs. 5 and 6 respectively.

The improvement in behavior achieved by stiffening the web to resist buckling is dramatic. Stiffened links can attain higher loads due to material strain hardening, they dissipate a greater amount of energy, and they can achieve much greater ductilities before failure by tearing. Tests on specimens with unequally spaced stiffeners also indicate a great sensitivity to relative panel dimensions, leading to the conclusion that, even in the presence of bending, the panel zones should be made equal.

Tests are currently being performed on a much larger class of specimens in an effort to assess the effect of other factors important in the behavior of active links in eccentrically braced frames. Results of these tests will be available at a later date.

References:

(1) Basler, K., "Strength of Plate Girders Under Combined Bending and Shear", Proc. ASCE, 87 (st7), 1961.

(2) Roeder, C.W., Popov, E.P., "Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loading", EERC Report 77-18, University of California, Berkeley, Aug. 1977.

(3) Evans, H.R., Porter, D.M., Rockey, K.C., "The Collapse Behavior of Plate Girders Subjected to Shear and Bending", IBASE Proceedings, P-18/78, 1978.

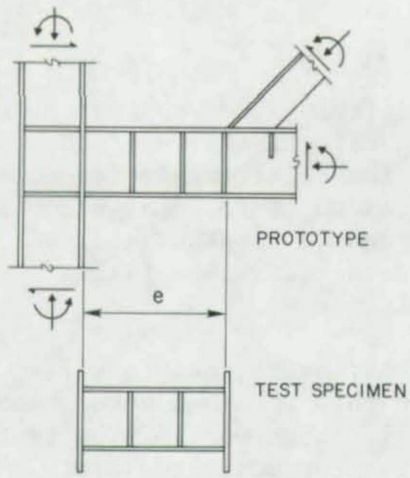


Fig. 1 Modeling of a Link

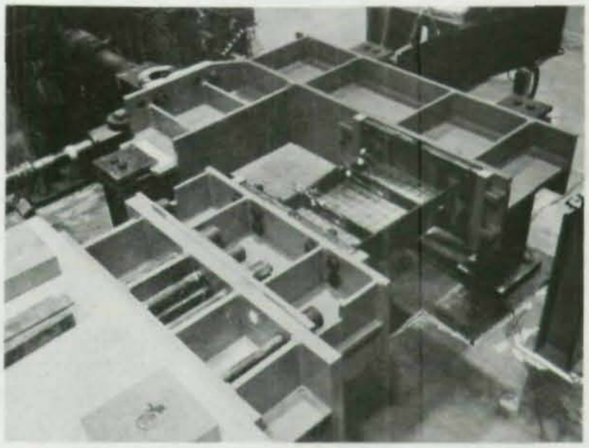


Fig. 2 Test Setup

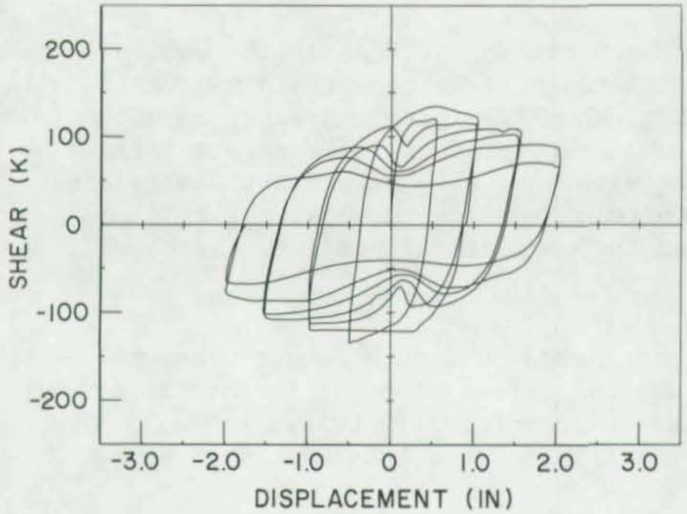


Fig. 3 Specimen 1 Force-Displacement History

Fig. 4 Failed Specimen 1

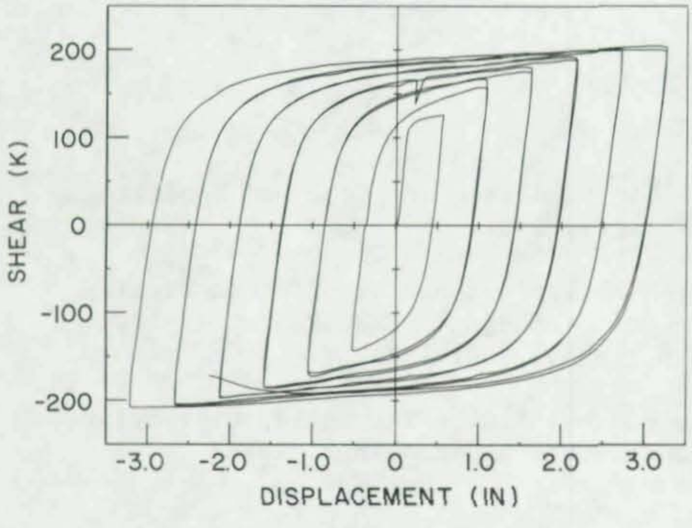


Fig. 5 Specimen 4 Force-Displacement History

Fig. 6 Failed Specimen 4

OTHER RESEARCH REPORTS cont'd

Eccentric Load Test of Angle Column Simulated with MSC/NASTRAN Finite Element Program

G. Haaijer, P. S. Carskaddan and M. A. Grubb, U. S. Steel Corporation

Single-angle columns are frequently connected at only one leg so that the axial force is accompanied by bending about the strong axis. Current specifications do not adequately cover the design of such members. For example, the AISI specification[1] for the design of cold-formed steel structural members states that the strength of singly symmetric shapes subject to both axial compression and bending applied out of the plane of symmetry (that is, the present case) must be determined by test.

An exploratory study was conducted to investigate the feasibility of using program MSC/NASTRAN[2] to simulate an elastic test of a 105-inch-long 5x5x5/16 angle loaded at the leg midwidth. Because of symmetry, only half the angle was modeled. QUAD4 rectangular modified isoparametric elements were used, 4 across the width of each leg and 20 along half the length. The boundary conditions for the eccentrically loaded specimen are illustrated in Figure 1. The loaded leg was free to rotate in its plane about the point of load application at midwidth of the leg. However, the end of the loaded leg was restrained from rotating out of its plane. The end of the unloaded leg was left completely unrestrained.

For eccentric axial load--the load applied at the leg midwith--two MSC/NASTRAN solutions were obtained: an eigenvalue and a large-deflection nonlinear solution. The eigenvalue is an approximate maximum load based on small-deflection formulation of the stiffness matrix. The large-deflection solution is theoretically correct. The resulting load-deflection curve is shown in Figure 2. A maximum compressive stress of 50 ksi in the midlength cross section was reached for an applied load of 51.8 kips.

The elastic eigenvalue load for an eccentrically loaded angle column can be used to establish the effective-length factor, K, for an equivalent concentrically loaded column that would have the same axial-load capacity. In the present example, K = 1.09.

For a steel with a yield point of 50 ksi column curve 2 of the SSRC Guide[6] gives a column strength of 51.7 kips. The above analysis was based on the assumption that the loaded leg was clamped against a stiff gusset plate that would prevent out-of-plane rotation. If the loaded edge were free to rotate out of its plane, MSC/NASTRAN calculates an eigenvalue load of 57.75 kips. The corresponding effective-length factor is K = 1.15. The column strength in accordance with curve 2 is 48.1 kips. Designers might want to use this lower conservative value.

OTHER RESEARCH REPORTS cont'd

Although the suggested approach appears promising, further studies are needed before the method can be proposed for general use. Experimental verification would be particularly valuable.

Bibliography

1. "Cold-Formed Steel Design Manual," American Iron and Steel Institute, 1977.

2. MSC/NASTRAN, the MacNeal-Schwendler Corporation, Los Angeles, California.

3. A. Chajes and G. Winter, "Torsional-Flexural Buckling of Thin-Walled Members," ASCE Structural Journal, August 1965.

4. S. P. Timoshenko and J. M. Gere, Theory of Elastic Stability, McGraw-Hill Book Company, 1961.

5. F. Bleich, Buckling Strength of Metal Structures, McGraw-Hill Book Company, 1952.

6. B. G. Johnston, editor, Guide to Stability Criteria for Metal Structures, third edition, Structural Stability Research Council, John Wiley and Sons, 1976.

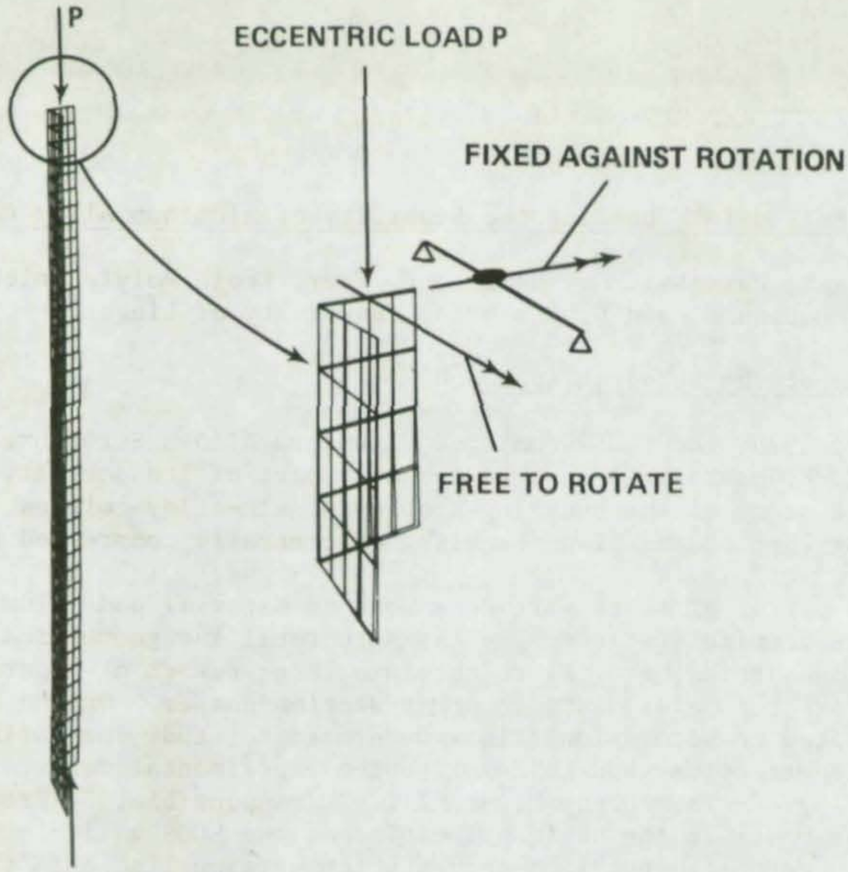


Fig. 1 - Boundary conditions for eccentrically loaded angle

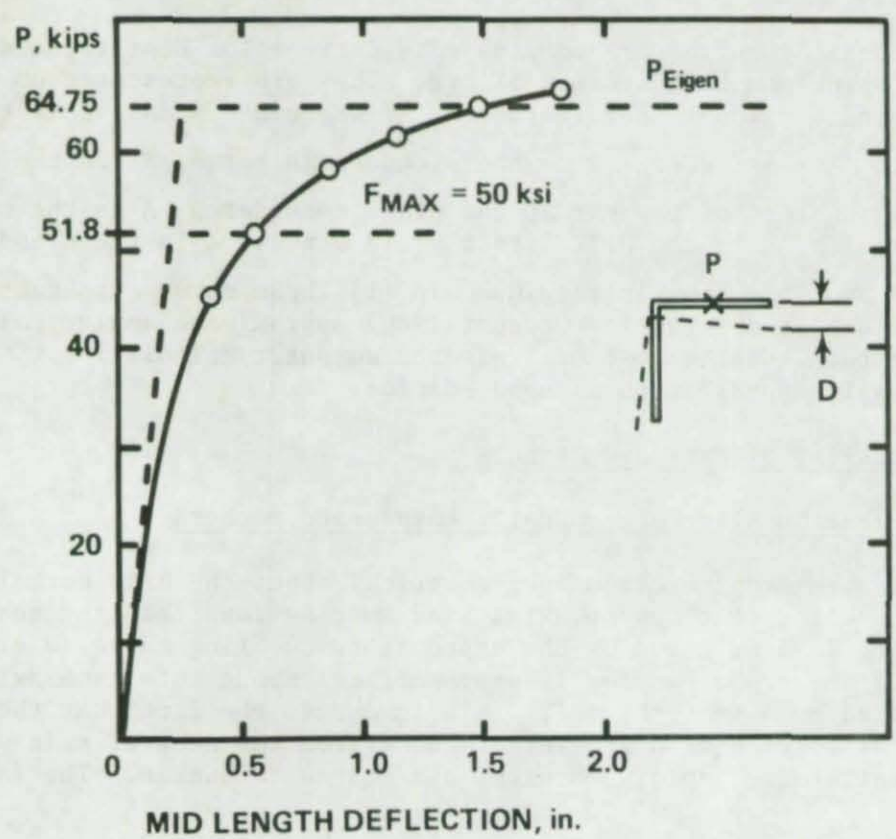


Fig. 2 - Lateral deflection from eccentric load

OTHER RESEARCH REPORTS cont'd

The ECCS Method for Checking the Stability of Aluminum Alloy Columns

F. Mazzolani, University of Naples, F. Frey, Ecole Polytechnique Federale, Lausanne, and S. Cescotto, University of Liege

Philosophy of the European Approach

Since 1970, the ECCS-Committee "Aluminum Alloys Structures" (Chairman, Prof. F. Mazzolani) devoted the main part of its activity to the systematic study of the buckling problem of alu-alloy columns, the basic case being that of the plane buckling of centrally compressed bars.

Wide series of tests were done both on material and columns in order: a) to characterize statistically the structural and geometrical imperfections of industrial bars; b) to obtain a large number of experimental buckling loads for several distinct cross section shapes. On the other hand, sophisticated computer simulations were used to study the influence of the various imperfections and to complete the experimental data. This work, performed by the Task Group "Instability" (responsible, F. Frey), followed as near as possible the basic principles of the ECCS valid for steel bars and led to establish buckling curves which were published in the first edition of the European Recommendations for Aluminum Alloy Structures [1]. These buckling curves form the base of the ECCS method for checking the stability of aluminum alloy columns.

The Buckling Curves

Two buckling curves are considered: Curve a for heat treated alloys and Curve b for non heat treated alloys. They are represented on Figure 1 in the nondimensional coordinates $(\overline{N}, \overline{\lambda})$ where $\overline{N} = N_C/A\sigma_{0.2}$ is the buckling force ratio, $\overline{\lambda} = \lambda/(\pi\sqrt{E/\sigma_{0.2}})$ is the slenderness ratio, N_C is the characteristic buckling load of the bar in the plane considered, A is the cross section area, $\sigma_{0.2}$ is the 0.2% offset yield stress, λ is the slenderness of the bar and E is Young's modulus. In [1] these curves are tabulated for easier use, and recently (October 1980) approximate analytical equations have been adopted to facilitate the computer calculations. These equations will appear in the second edition.

Overall Buckling of Extruded Members

3.1 Plane buckling of centrally compressed members

If the cross section is symmetrical about the axis normal to the plane of buckling, the design axial load must be less than the characteristic buckling load N_C given by the appropriate buckling curve (a or b). However, if the cross section is unsymmetrical about this same axis, N_C is multiplied by a factor $k < 1$. This is due to the fact that the points of the cross section at a greater distance from the neutral axis will become plastic more rapidly when the bar starts to buckle. The factor k is a function of the unsymmetry coefficient ψ defined by $\psi = (y_{max} - y_{min})/h$ (see Figure 2), and of the slenderness ratio $\overline{\lambda}$. A provisional value of k is given in the comments of [1]. It is based on a parametric numerical simulation and an experimental program is presently developed to check the validity of the formulation.

3.2 Members in bending (lateral buckling)

For beams of bisymmetrical cross section loaded in the plane of the web, the moment M_D which causes the lateral buckling failure is computed by $M_D = M_P/\sqrt{1 + \overline{\lambda}^4}$, where $\overline{\lambda} = \sqrt{M_P/M_{cr,D}}$ is a slenderness parameter, M_P is the plastic moment for major axis bending and $M_{cr,D}$ is the theoretical elastic critical bending moment for lateral buckling. The plastic moment is to be computed as for steel (product of the plastic modulus Z by the yield limit $\sigma_{0.2}$)

3.3 Members in bending and compression

Beam columns are assumed to have a bisymmetrical cross section. Sway is supposed to be prevented.

In the case of bending in one plane only, the stability is ensured if the following relation is verified

$$\frac{N}{N_C} + \frac{m}{m - 1}\frac{M^*}{M_D} \leq 1 \qquad (1)$$

N_C is the smallest buckling load according to §3.1 above; $m = N/N_E$; N_E = Euler buckling load in the plane of bending; M* is an equivalent bending moment.

In the case of biaxial bending, let x-x be the strong axis and y-y the weak axis of the cross section. Then the following inequality ensures *stability*

$$\frac{N}{N_C} + \frac{m_x}{m_x - 1}\frac{M^*_x}{M_{xD}} + \frac{m_y}{m_y - 1}\frac{M^*_y}{W_y\sigma_{0.2}} \leq 1 \qquad (2)$$

where W_y is the section modulus corresponding to the weak axis y-y.

Overall Buckling of Welded Members

4.1 Plane buckling of centrally compressed members

For members with longitudinal welds, the buckling load computed as in §3.1 above is multiplied by a factor $n < 1$ (see Figure 3). This reduction takes account of the existence of residual stresses and, in the case of heat treated alloys, of a zone of reduced strength in the vicinity of the welds. For members with transversal welds, the buckling load computed as in §3.1 above is multiplied by a factor $\beta < 1$ (metallurgical efficiency factor) corresponding to the lowering of the mechanical properties of the parent metal due to welding and depending on the combination parent metal-filler metal.

4.2 Members in bending (lateral buckling)

For members with longitudinal welds, the requirements of §3.2 remain valid, except that all computations must be conducted with the "reduced cross sectional properties" defined by:

- reduced area: $A_r = A - (1 - \beta) \sum_i A_{rsz,\, i}$
- reduced moment of inertia: $I_r = I - (1 - \beta) \sum_i (A_{rsz,\, i} \cdot y_i^2)$

where $A_{rsz,\, i}$ is the part i of the cross sectional area with reduced strength properties and y_i is the corresponding distance to the bending axis. For members with transversal welds, the requirements of §3.2 remain valid except that $\sigma_{0.2}$ has to be replaced by $\beta \cdot \sigma_{0.2}$.

4.3 Members in bending and compression

Formulas (1) and (2) become respectively

$$\frac{N}{N^*_C} + \frac{m}{m - 1} \frac{M^*}{M^*_D} \leq 1 \tag{3}$$

$$\frac{N}{N^*_C} + \frac{m_x}{m_x - 1} \frac{M^*_x}{M^*_{xD}} + \frac{m_y}{m_y - 1} \frac{M^*_y}{W^*_y \sigma^*_{0.2}} \leq 1 \tag{4}$$

where N^*_C and M^*_{xD} are computed according to §4.1 and §4.2 respectively. For members with longitudinal welds, $A^* = A_r$ (reduced area), $W^* = I_r/y_{max}$ (reduced section modulus) and $\sigma^*_{0.2} = \sigma_{0.2}$ (yield stress of the unaffected metal). For members with transversal welds, $A^* = A$, $W^* = W$ and $\sigma^*_{0.2} = \beta\sigma_{0.2}$.

Future Development and Improvements

The rules presented here will be improved in the future by considering the case of unsymmetric cross sections. On the other hand, provisions for local buckling and torsional buckling already exist in [1] but have not been exposed here by lack of space. These provisions, however, are still incomplete and require improvements. This will be the trend of the future work on instability of the ECCS-Committee "Aluminum Alloys Structures".

OTHER RESEARCH REPORTS cont'd

Reference

1. ECCS-CECM-EKS Committee T2, "Aluminum Alloy Structures", European Recommendation for Aluminum Alloy Structures, First Edition, 1978 (May be ordered from: Mr. A. van der Heyden, Centre d'Information des Metaux Non Ferreaux, Bd. de Berlaimont, 12 B-1000 BRUXELLES, *BELGIUM*)

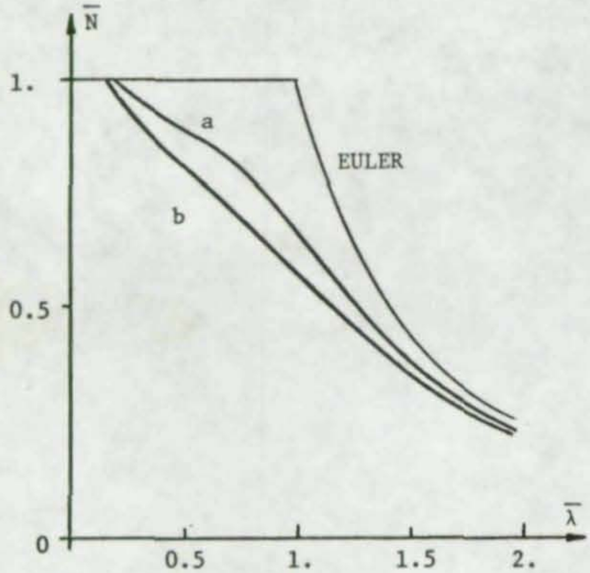


Figure 1

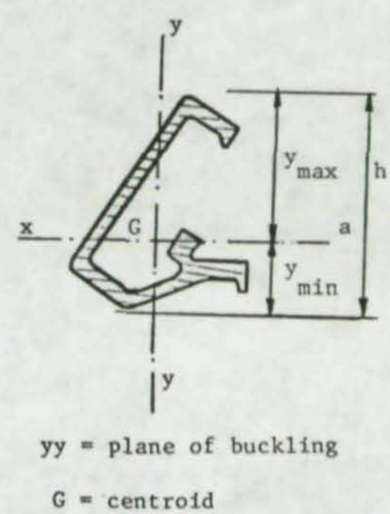


Figure 2

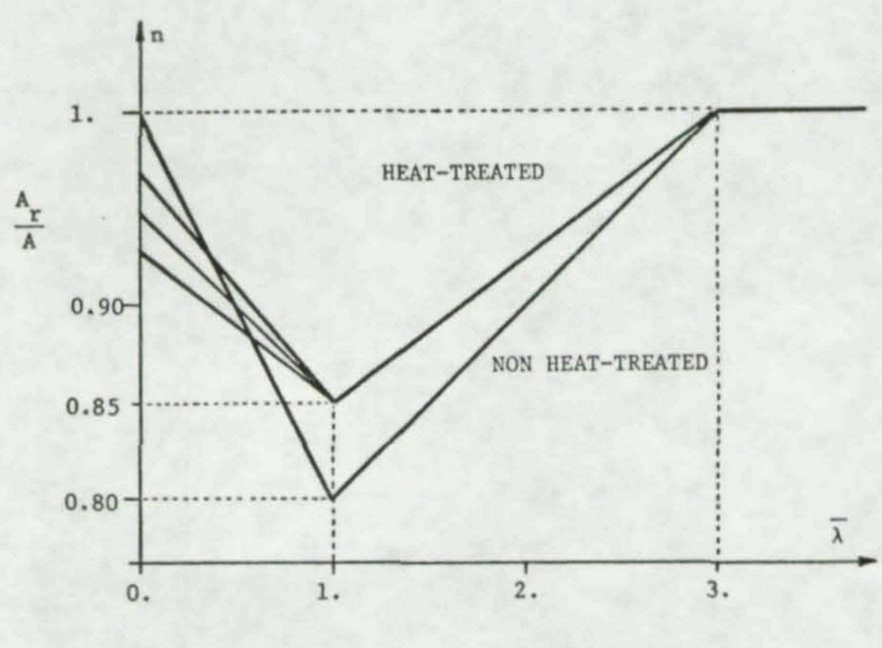


Figure 3

F. R. Khan, M. M. El Nimeiri, R. S. Nair, J. G. Stockbridge, G. Winter

PANEL DISCUSSION

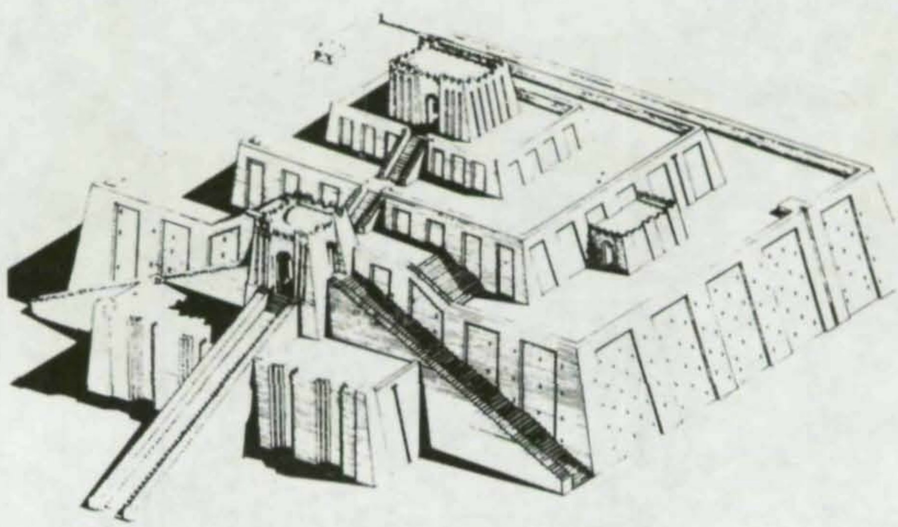
STABILITY OF TALL BUILDINGS

MODERATOR: George Winter, Cornell University

PANELISTS: R. Shankar Nair, Alfred Benesch & Company
Fazlur R. Khan, Skidmore Owings & Merrill
Jerry G. Stockbridge, Wiss, Janney, Elstner and Associates

GEORGE WINTER - Introductory Remarks

The development of highrise buildings as we know them today began about 100 years ago right here in Chicago. It is, therefore, most appropriate that our panel discussion will deal with one aspect, stability, of this type of construction. However, man has been building tall, high structures for thousands of years. In this introduction, I will give you two examples of the earliest such structures, one in the Old World and one in the New World, and then a very brief review of the development of the modern "skyscraper".



You all know of the biblical Tower of Babel. It was what we now call a Babylonian ziggurat. This slide shows a reconstruction of the best preserved of these ziggurats, of about 2100 B.C. I am guessing the height of this one to be about 150 ft. The reference in Gen. 11.4 is most revealing: "Come, let us build a tower with its top in heaven, and let us make a name for ourselves." The motivation of making a name for oneself has persisted for 4000 years, from the Tower of Babel to the World Trade Center.

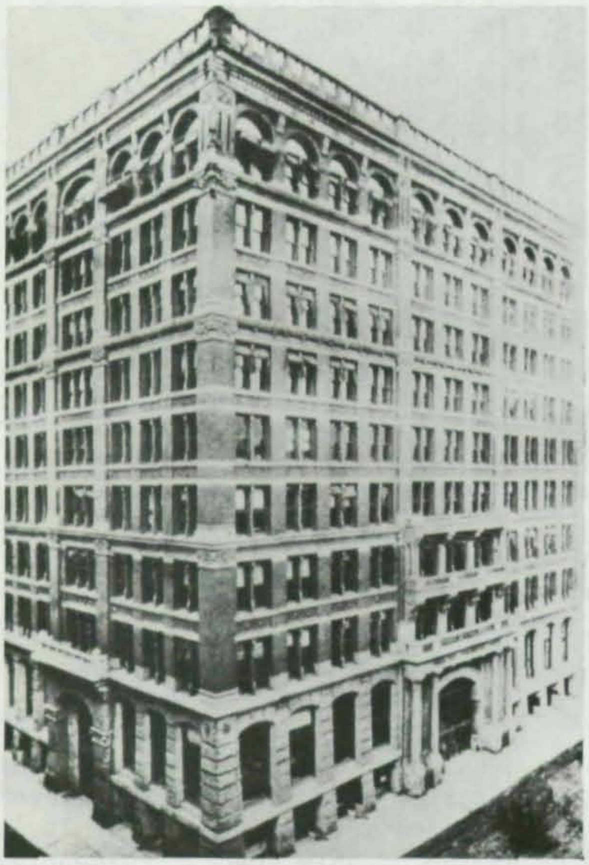
PANEL DISCUSSION (WINTER)

The tallest ancient structure in the Western hemisphere is of the same nature as the Tower of Babel. It is Pyramid IV in the great ruin city of the Mayas, Tekal in Guatemala. This shows a similar though smaller Mayan pyramid. Pyramid IV is about 220 ft. high, roughly the height of an 18 - 20 story building. It served both as temple and as astronomical observatory, and was built around 700 A.D., i.e. around 1300 years ago.

We will now jump some 1200 years and go directly to the decisive Chicago developments. One can probably say that the skyscraper development in Chicago started in 1870 with the 9 - 10 story Marshall Field Building. It was, of course a masonry structure, designed by Daniel Burnham. After Chicago was largely destroyed by fire in 1871, a "Chicago School" of architects started to develop the essentials of highrise construction.

PANEL DISCUSSION (WINTER)

The development first continued in masonry, which reached its limit in the Monadnock Building. It was 16 stories high, and at groundlevel its bearing walls, both exterior and interior were 7 ft. thick. It is easy to imagine how much this encroached on useable space. Evidently, the development could not continue in masonry construction. The first use of cast and wrought iron in columns and beams came in the 11-story Home Life Insurance Building. The leading architectural firm of Burnham and Root then quickly adopted the complete steel skeleton, first in the 15-story Reliance Building, in 1890. This strikes us as

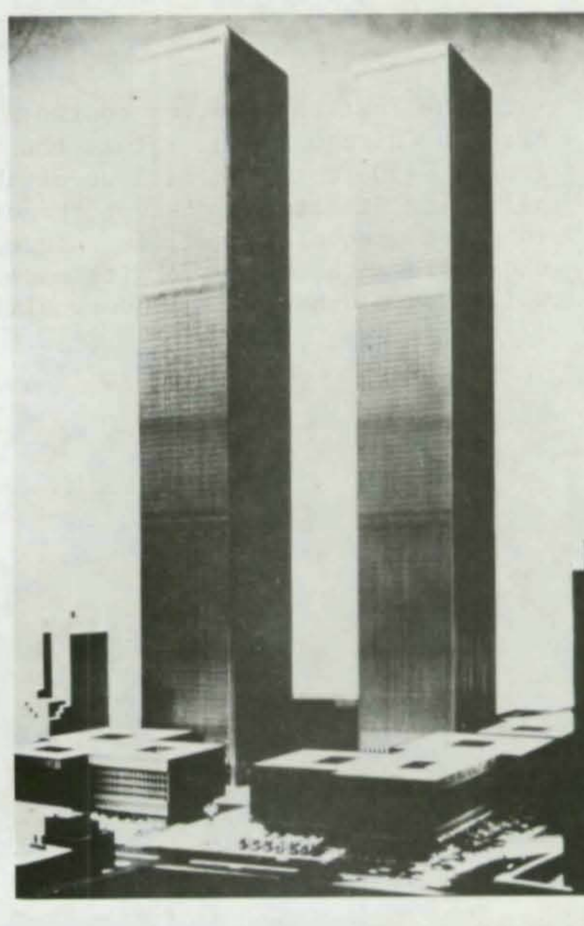


definitely modern with its thin columns and floors, and large glass areas. With this structure the modern highrise building had been launched. It was made possible by three technical innovations: the steel skeleton; the electric elevator; and, if you don't mind, the flush toilet. The combination of these three produced a revolution in urban development.

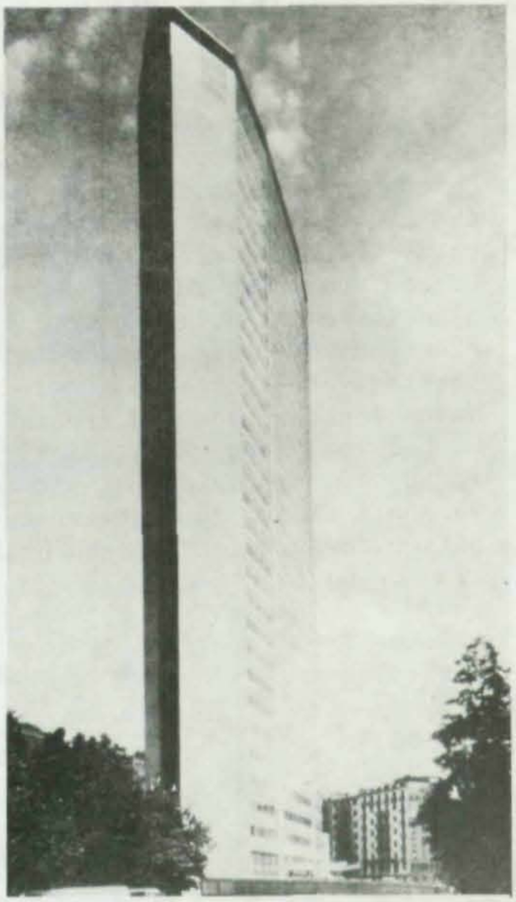
PANEL DISCUSSION (WINTER)

We will now jump directly to four outstanding examples of the most recent developments, two of them again in Chicago. My favorite among these is the Hancock Tower, some 90 stories, I think. It is elegant in concept, a braced, tapering tube, and innovative also in function, since it combines, from the bottom up, commercial space, parking, office space, a swimming pool and sky plaza, some 40 floors of apartments, and on top TV studios, restaurants and observatories. It is alive 24 hours a day, in contrast to the pure office tower which is dead half the time or more.

The twin towers of the World Trade Center in New York are such office buildings which are dead half or more of the time. Personally, I rather dislike these somewhat brutal towers. They have spoiled once and for all the unique skyline of New York, whether you see it from the water or the air. To me they say clearly: "Let us build high and let us make a name for ourselves as the Babylonians in the Bible."



PANEL DISCUSSION (WINTER)



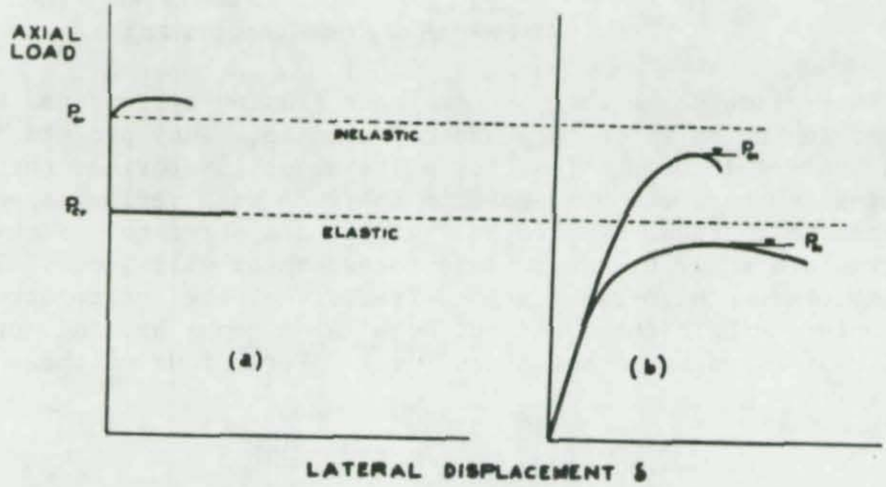
Much in contrast, in courage and delicacy of approach, is the Pirelli Tower in Milan, Italy, built in 1961. Although it too is huge in comparison with its surroundings, the delicacy of its curved and pointed shape presents esthetic pleasure rather than brute force.

The last example, of course, is here in Chicago again. It is the Sears Tower, 1450 feet, the tallest of them all. Its structural concept, bundled tubes of unequal length, is unique and mellows the effect of its mass, which would otherwise be overwhelming.

PANEL DISCUSSION (WINTER)

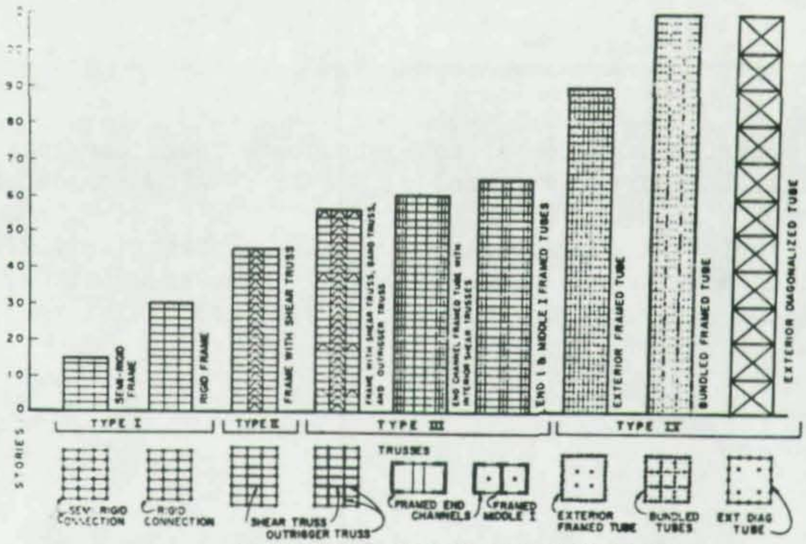
So much for examples of tall structures over the last 4000 years. Now let me very briefly introduce the topic of our panel discussion.

The topic, of course, is: Stability of Highrise Buildings. Stability can mean things to many people. However, what we, or at least I, mean in the present connection is illustrated by this slide. If you



have a system, anything from a single column to a highrise structure, and it is "ideal", i.e. straight, perfect, axially loaded etc., it becomes unstable at a certain load by sudden transverse deflection, at the so-called bifurcation load. If the same system is real, i.e. geometrically imperfect and under intentionally or accidentally eccentric loading, it behaves as shown. I.e. with increasing load, lateral deflections increase, at an increasing rate, until a peak load is reached. This is the stability limit P_{st}, shown for both elastic and inelastic conditions. For high-rise buildings, the details depend, of course, on the framing system and other features.

PANEL DISCUSSION (WINTER)



COMPARISON OF STRUCTURAL SYSTEMS

This slide shows the most frequent framing systems and the number of stories up to which they are appropriate. They proceed from framing with semi-rigid connections for up to about 15 stories, through rigid framing (up to about 30) and from there on with various special bracing systems to increase transverse rigidity and strength. Such bracing systems are shear trusses, rigid cores, shear walls, etc. The highest of the present high-rises are so framed that the entire structure represents a single tube, without or with diagonal bracing, or even a bundle of tubes as in the Sears Tower. Here, four of these framing

Fig. 5.7 Circular tubular framework

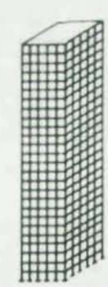


Fig. 5.8 Rectangular tubular framework

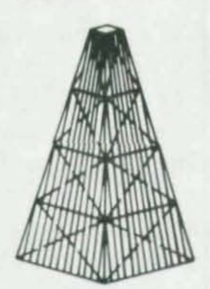


Fig. 5.9 Trussed tubular frame with sloping exterior columns

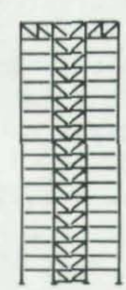


Fig. 5.10 Center braced core capped with truss attached to exterior tie-down columns

systems, three tubes and one with braced core, are shown in a better detail.

PANEL DISCUSSION (WINTER)

This slide illustrates, somewhat exaggeratedly, how sensitive such systems are in regard to stability, depending on design features. This asymmetrical structure is braced by a shear-rigid core. The stability depends to a very high degree on the location of that core.

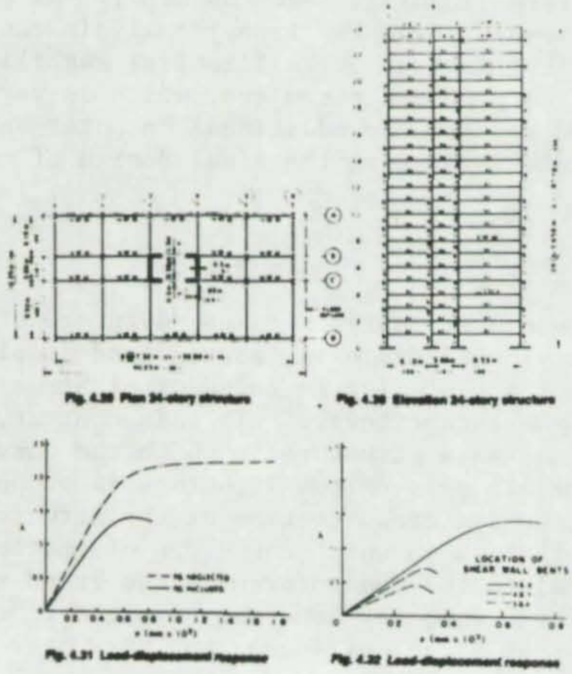


Fig. 4.29 Plan 24-story structure

Fig. 4.30 Elevation 24-story structure

Fig. 4.31 Load-displacement response

Fig. 4.32 Load-displacement response

The optimum location of the core is, of course, at the center of the long dimension. If it is located one bay off center, the stability limit falls to about one-half of the concentric one, and when it is two bays off center, it falls to about one-third. The strength of the individual members, of course, is the same in all three cases. However, when the braced core is located asymmetrically, twist is induced which radically reduces the overall frame stability.

All this refers only to the stability of the framing itself, under static conditions. There are many other important questions, such as the interaction of framing with external cladding and with internal partitions, dynamic instability under resonance conditions, and ways of influencing it by damping devices, and a host of other matters. All I wanted to do here was to sketch the historical development of highrises, and of stability questions connected with them.

PANEL DISCUSSION

R. SHANKAR NAIR - A Simple Method of Overall Stability Analysis For Multistory Buildings

Introduction

Accurate linear analysis with a digital computer is, nowadays, a standard part of the structural design process for multistory buildings. Several computational techniques and computer programs are available to the designer for this purpose. In general, the results of the linear analysis will include the lateral displacements caused by lateral loadings (such as wind). As explained in this paper, these results from the linear analysis can be used to obtain a very good estimate of overall lateral stability effects in the building. The proposed technique, which is very simple and straightforward and requires no additional computer analysis, is sufficiently accurate for use in the final design of most multistory buildings.

Development of Procedure

The proposed method of analysis takes advantage of the fact that most multistory buildings have lateral load-displacement characteristics that are similar to those of either a "flexural cantilever" or a "shear cantilever." In this context, a flexural cantilever is defined as a structure in which the curvature of the longitudinal (vertical) axis of the structure is proportional to the bending moment on the cross section of the structure. A shear cantilever is a structure in which the slope of the longitudinal axis is proportional to the shear force on the cross section of the structure. In either case, the constant of proportionality (flexural stiffness or shear stiffness) may vary over the height of the structure.

Flexural Cantilever

Buildings with braced frames or shearwalls and very tall buildings with unbraced frames (in which lateral displacement is caused primarily by column shortening and column elongation) usually have lateral load-deformation characteristics that approach those of a flexural cantilever.

For a flexural cantilever of height H and constant stiffness EI, the uniformly distributed vertical load, p_c per unit height, that will cause lateral buckling is given by the equation:

$$p_c = 7.84\ EI/H^3 \qquad \ldots(1)$$

PANEL DISCUSSION (NAIR)

If the stiffness of the cantilever varies with height in accordance with the equation, $EI=(a/H)EI_o$, where EI_o is the stiffness at the base and a is the distance from the top, the critical load is given by:

$$p_c = 5.78\ EI_o/H^3 \qquad ...(2)$$

If the stiffness varies with the equation, $EI=(a/H)^2EI_o$, the critical load is:

$$p_c = 3.67\ EI_o/H^3 \qquad ...(3)$$

These equations for critical load (Eq. 1, 2 and 3, above) can be found in basic texts on elastic stability.

If a uniformly distributed lateral load of f per unit height is applied on cantilevers with each of the three stiffness configurations described above, the lateral displacement, Δ, at the top of the three structures is given by the following:

$$\text{For constant EI: } \Delta = 0.125\ fH^4/EI \qquad ...(4)$$

$$\text{For } EI=(a/H)EI_o\text{: } \Delta = 0.167\ fH^4/EI_o \qquad ...(5)$$

$$\text{For } EI=(a/H)^2EI_o\text{: } \Delta = 0.250\ fH^4/EI_o \qquad ...(6)$$

By combining Eq. 1, 2 and 3 with Eq. 4, 5 and 6, respectively, EI can be eliminated and p_c can be expressed in terms of f/Δ as follows:

$$\text{For constant EI: } p_c = 0.98\ fH/\Delta \qquad ...(7)$$

$$\text{For } EI=(a/H)EI_o\text{: } p_c = 0.96\ fH/\Delta \qquad ...(8)$$

$$\text{For } EI=(a/H)^2EI_o\text{: } p_c = 0.92\ fH/\Delta \qquad ...(9)$$

Equations 7, 8 and 9 cover the range from constant stiffness to a more extreme stiffness variation than is normal in real multistory buildings. It is obvious that the relationship between p_c and f/Δ is not very sensitive to stiffness variations over the height of the structure. Regardless of the distribution of stiffness, the following equation is sufficiently accurate for purposes of design:

$$p_c = 0.95\ fH/\Delta \qquad ...(10)$$

Thus, if the lateral displacement caused by lateral loading is known, the critical load for lateral buckling can be accurately and easily estimated using Eq. 10. (See Fig. 1 for definition of symbols and summary of procedure.)

Analysis and Design Procedure

The following procedure is suggested for the overall stability analysis and design of buildings that approach the behavior of flexural cantilevers.

1. Analyze the structure for wind loading (or other lateral loading). Denote the lateral deflection of the top of the building as Δ. If the wind load is uniform, denote the wind load per unit height as f. If it is not uniform, define f as the uniform lateral load that would produce the same base moment as the wind loading used in the analysis. Let H be the total height of the building.

2. Compute the critical load per unit height, p_c, from Equation 10, above.

3. Compute the magnification factor , as follows:

$$\mu = \frac{1}{1 - \frac{\gamma p}{\phi p_c}} \qquad \ldots(11)$$

   where p is the actual average vertical load per unit height on the building, γ is the load factor, and ϕ is the strength reduction factor. (Note: p must include the load on <u>all</u> vertical members, including those that are not part of the lateral load-resisting system. Thus, p is the total vertical load on the entire building divided by the height H.)

4. For design of structural members, multiply all lateral load effects (i.e., all moments, shears and axial forces caused by lateral loading) by the factor μ. If gravity loads cause lateral displacement of the building, the "sidesway" component of the moments and forces due to the gravity loading should also be magnified by the factor μ .

5. Design structural members for the magnified forces and moments, with all floors assumed to be restrained against lateral displacement (or "braced against sidesway").

In the procedure outlined above, no distinction has been made between the effects of moment on the overall building and the effects of shear on the overall building. (The effects of moment include axial forces in columns, moments in independent shearwalls, and moments and axial forces in coupled shearwalls. The effects of shear include shear forces in shearwalls, axial forces in bracing diagonals, and moments and shear forces in beams and columns.) The magnification factor computed in Step 3, above, is strictly applicable only to the effects of moment on the overall structure. Application of the same factor to the shear effects represents an approximation.

Shear Cantilever

Buildings of low or moderate height with unbraced frames (in which column shortening and column elongation do not contribute significantly to lateral displacement) usually have lateral load-displacement characteristics similar to those of a shear cantilever.

If a portion of a vertical shear cantilever undergoes lateral deformation δ over height h when subjected to a shear force V, the critical vertical load for lateral buckling of that portion of the cantilever is given by:

$$P_c = Vh/\delta \qquad \ldots(12)$$

Equation (12) is also applicable, as an approximation, to framed structures (as opposed to theoretical shear cantilevers). When Eq. 12 is applied to a story of a building, h is the story height, δ is the lateral deformation of the story caused by a shear force of V in the story, and P_c is the total vertical force that would cause lateral buckling of the story. (See Fig. 2)

The accuracy of Eq. 12, when applied to a story of a framed structure, depends on the relative stiffness of beams and columns and on the way in which gravity loads are distributed among the columns in the story. The source of error in Eq. 12 is the nonlinearity of the stiffness matrix of individual columns in the story; the flexural stiffness of each column is influenced by the vertical load on the column. If most of the vertical load in the story is in columns that are not part of the lateral load-resisting frame, Eq. 12 will be very nearly exact since the flexural stiffness of these columns has no influence on the lateral stiffness or stability of the structure. If most of the vertical load is in the lateral load-resisting frame, the error in Eq. 12 can be between 0 and about 20%, depending on the relative stiffness of beams and columns. (The error is greatest for stiff beams and slender columns.)

Analysis and Design Procedure

The following procedure is suggested for lateral stability analysis and design of buildings that approach the behavior of shear cantilevers. Note that since lateral buckling of this type of structure is largely a one-story phenomenon, the magnification factor must be computed specifically for the particular story that is being considered; it may be different in different stories.

1. Analyze the structure under wind loading (or other lateral loading).

2. For the story being considered:

   h = height of story;

   V = total horizontal shear in story due to loading used in Step 1;

   δ = lateral deformation of story (from results of Step 1).

3. Compute the critical load for the story, P_c, from Equation 12.

4. Compute the magnification factor, μ, for the story:

$$\mu = \frac{1}{1 - \frac{\gamma P}{\phi P_c}} \qquad \text{...(13)}$$

   where P is the actual total vertical force in the story. γ is the load factor, and ϕ is the strength reduction factor.

5. Apply the magnification factor, μ, to the moments and shears produced in beams and columns by lateral loading. If gravity loads cause lateral displacement of floors in the building, the "sidesway" component of moments and shears due to gravity loading should also be magnified by the factor μ.

6. Design members for the magnified forces and moments, with floors assumed to be restrained against lateral displacement (or "braced against sidesway").

It may be noted that in Step 5, above, the magnification factor is applied only to the effects of shear on the building. (These effects are moments and shears in beams and columns.) Lateral stability effects will also cause some magnification of the column axial forces produced by lateral loading. This effect is usually unimportant in a shear-cantilever type of structure and can usually be neglected. Alternatively (as a conservative approximation, in most cases) the μ factor computed using Eq. 13 can be applied to all lateral load effects in the story, including column axial forces due to lateral load.

Torsional Stability

The techniques that have been developed for planar stability analysis can easily be extended to allow analysis of torsional stability. In this case, torsional displacements indicated by linear analysis under torsional loading are used to obtain estimates of critical loads for torsional buckling.

PANEL DISCUSSION (NAIR)

If a multistory building's torsional stiffness is provided by braced frames, shearwalls or tall unbraced frames (in which lateral displacements are caused primarily by column length changes)--and if these stiffening elements are not arranged in the form of a closed tube--the building will have torsion-rotation characteristics that are similar to the lateral load-displacement characteristics of a flexural cantilever. The formula for torsional buckling of such a building (analogous to Eq. 10) is as follows:

$$r^2 p_c = 0.95\ tH/\theta \qquad \text{...(14)}$$

in which t is an applied torsional load, per unit height, on the building; θ is the rotation of the top of the building, in radians, due to load t; H is the height of the building; p_c is the critical vertical load, per unit height, for torsional buckling of the building; r is the polar radius of gyration of the vertical loading, about the vertical axis of the building.

For a doubly symmetrical structure with uniform loading on rectangular floors of plan dimensions a and b:

$$r^2 = (a^2 + b^2)/12 \qquad \text{...(15)}$$

If a building's torsional stiffness is provided by unbraced frames in which lateral displacements are caused primarily by "shear wracking" deformations--or if the stiffening elements of the building are arranged in the form of a closed tube--the building will have torsion-rotation characteristics that are similar to the lateral load-displacement characteristics of a shear cantilever. The formula for torsional buckling of a particular story of such a building (analogous to Eq. 12) is as follows:

$$r^2 P_c = Th/\theta \qquad \text{...(16)}$$

in which T is an applied torsional load on the story; θ is the torsional deformation of the story, in radians, due to torque T; h is the height of the story; P_c is the critical load for torsional buckling of the story; r is the polar radius of gyration of the vertical load.

With critical loads for torsional buckling computed using Eq. 14 or 16, as appropriate, the magnification factor concept can be used for design.

Examples

Figure 3 shows six 20-story buildings on which complete and rigorous stability analyses and large-deformation lateral-load analyses have been performed. The methods of analysis and selected results from the analyses are described in Reference 1. These buildings have also been analyzed using the simple methods of stability analysis proposed in the present paper. Agreement between the simple and rigorous analyses has been found to be excellent, as illustrated below for North-South loading of Building V and VI from Fig. 3.

Building V (Braced Frames)

A linear analysis was performed on this structure under a wind load of 25 psf on the south face (which is 138 ft. wide). The resulting displacement at the top of the building was 0.729 ft. The critical load can now be determined from Eq. 10 as follows:

$$H = 240 \text{ ft.}$$

$$f = 0.025(138) = 3.45 \text{ k/ft.}$$

$$\Delta = 0.729 \text{ ft.}$$

$$p_c = 0.95(3.45)(240)/0.729 = 1079 \text{ k/ft.}$$

This critical load of 1079 kips per foot corresponds to 12948 kips per floor or 1360 psf on each floor. The actual gravity load on this building was taken as 130 psf on each floor. The corresponding magnification factor (computed without load factors or strength reduction factors) is as follows:

$$\mu = 1/(1 - 130/1360) = 1.106$$

and the magnified lateral displacement at the roof is given by:

$$\mu\Delta = 1.106(0.729) = 0.806 \text{ ft.}$$

The rigorous stability analysis of this building indicated a critical load for North-South buckling of 1369 psf on each floor (cf. 1360 psf from the simple analysis). The complete large-deformation analysis under combined gravity load and North-South wind loading indicated a roof displacement of 0.805 ft. (cf. 0.806 ft. from the simple analysis).

Building VI (Unbraced Frames)

The behavior of this building, in the North-South direction, can be expected to be closer to that of a shear cantilever than to that of a flexural cantilever. Since instability of a shear cantilever is essentially a local or one-story phenomenon, critical loads and magnification factors must be computed separately for each story. The 5th, 10th and 15th stories will be considered in the following.

PANEL DISCUSSION (NAIR)

A linear analysis was performed on the building under a wind force of 25 psf on the south face. The resulting story deformations (relative lateral displacement of floors at the top and bottom of the story) were 0.0522 ft., 0.0609 ft. and 0.0582 ft. in the 15th, 10th and 5th stories, respectively. The corresponding shear forces in these stories were 228 kips, 435 kips and 642 kips. Each story is 12 ft. high. Critical loads for lateral buckling of these stories can be determined from Eq. 12 as follows:

$$\text{15th story:}\quad P_c = 228(12)/0.0522 = 52414 \text{ kips}$$

$$\text{10th story:}\quad P_c = 435(12)/0.0609 = 85714 \text{ kips}$$

$$\text{5th story:}\quad P_c = 642(12)/0.0582 = 132371 \text{ kips}$$

The actual gravity load was taken as 130 psf on each floor. The corresponding total vertical force in the 15th, 10th and 5th stories is 7427 kips, 13616 kips and 19806 kips, respectively. The magnification factors (computed without load factors or strength reduction factors) are as follows:

$$\text{15th story:}\quad \mu = 1/(1 - 7427/52414) = 1.165$$

$$\text{10th story:}\quad \mu = 1/(1 - 13616/85714) = 1.189$$

$$\text{5th story:}\quad \mu = 1/(1 - 19806/132371) = 1.176$$

and the magnified lateral story deformations are as follows:

$$\text{15th story:}\quad \mu\delta = 1.165(0.0522) = 0.0608 \text{ ft.}$$

$$\text{10th story:}\quad \mu\delta = 1.189(0.0609) = 0.0724 \text{ ft.}$$

$$\text{5th story:}\quad \mu\delta = 1.176(0.0582) = 0.0684 \text{ ft.}$$

The complete large-deformation analysis of this building under combined gravity load and North-South wind loading indicated story deformations of 0.0607 ft., 0.0723 ft. and 0.0686 ft. in the 15th, 10th and 5th story, respectively.

Summary and Conclusions

Procedures have been developed for the inclusion of overall lateral stability effects in the design of multistory buildings. These procedures, which are very simple to use, are applicable to the framing systems of most high and medium-rise buildings. The accuracy of the proposed techniques has been demonstrated by comparing the results yielded by these procedures with the results of complete and rigorous stability and large-deformation analyses.

PANEL DISCUSSION (NAIR)

REFERENCE

1. Nair, R. S., "Overall Elastic Stability of Multistory Buildings, " Journal of the Structural Division , ASCE, Vol. 101, No. ST12, Dec. 1975.

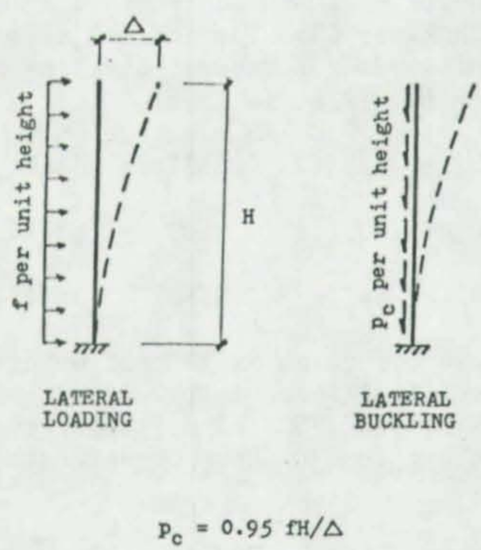


Fig. 1.-- Lateral Loading and Buckling of Flexural Cantilever

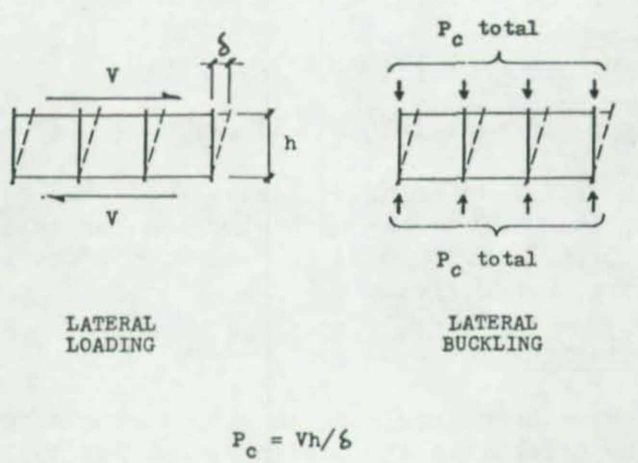
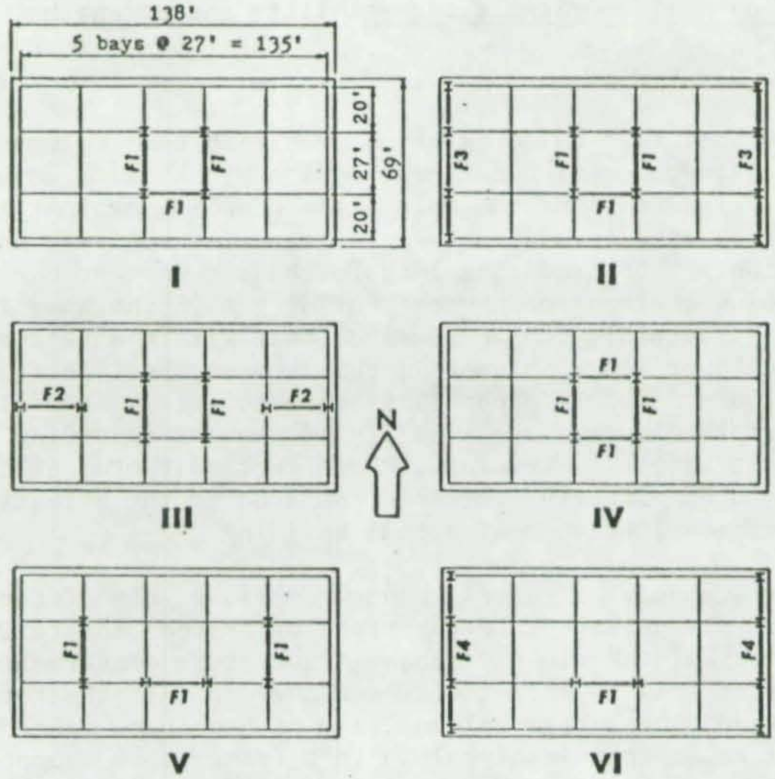
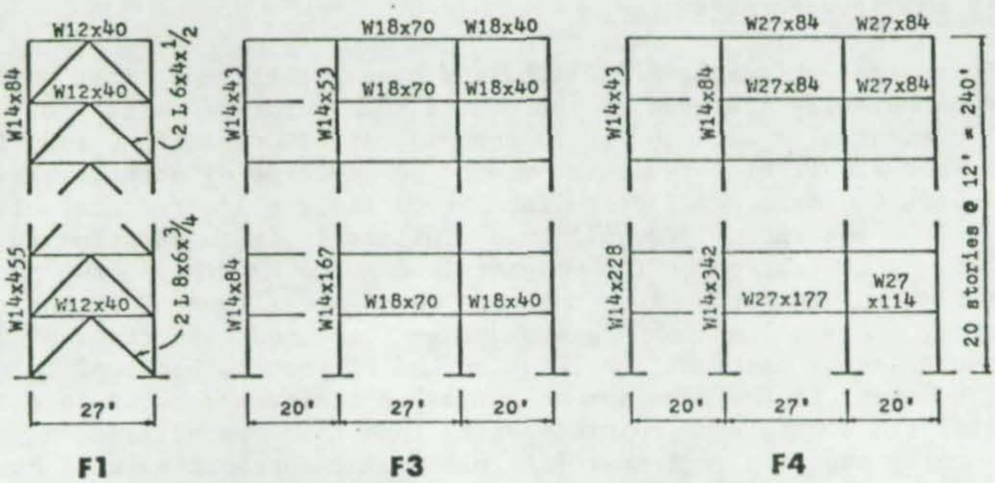


Fig. 2.-- Lateral Loading and Buckling of Story in Shear-Cantilever Type Building



PLAN OF BUILDINGS



ELEVATION OF FRAMES

Fig. 3.-- Buildings Analyzed in Reference 1.

PANEL DISCUSSION

Fazlur R. Khan and Mahjoub M. El Nimeiri - Effects of Structural Redundancy and Importance on Design Criteria for Stability and Strength

Introduction

The traditional tall building structures with beam column frames have been designed without any special consideration for probability aspects of failures of its members. In designing the column and checking against its load carrying capacity, no difference is made between a high rise building and normal low rise construction. This traditional approach assumes that the vertical members of a tall building and that of a short building have the same levels of importance in the overall structure. It also assumes that within a tall structure the importance of the lower floor columns is the same as the importance of the columns in the upper floors. From a probability point of view, it would be more appropriate to adjust the level of reliability of members according to their importance in the overall structure. Therefore, the effective factor of safety of the lower level columns can be slightly increased compared to the effective factor of safety for upper level columns of a tall building.

Similar questions should be raised when one considers the different performance probabilities of altogether different types of systems for tall buildings. The recent development of the closely spaced framed tube construction for tall buildings provides a large amount of redundancy against the total collapse when compared to a traditional beam column frame with columns spaced far apart (Figure 1). Since an individual column in a framed tube cannot by itself initiate a progressive collapse or an incipient failure of the entire structure, it would be reasonable to question why the effective factor of safety of columns in a framed tube construction should be the same as for a traditional beam column framing system.

Performance of a Single Structural Member Versus Reliability of the Total Structural System

The general principle of reliability of a structure against possible failure is normally achieved by the use of the factor of safety for a design of each element of a structure. In the present AISC specifications it is assumed that the single structural member in any type of structural system will require the same level of reliability. Since all structures ultimately relate to the concept of probability of failure it is interesting to make a qualitative illustration of the point raised above by drawing two normal distribution curves for a given compression member (Figure 2). The curve for a one story frame column for a given design force and conditions of restraints will have a normal distribution curve marked "A" on the basis of idealized tests. However, if the same member exists on the ground floor in a tall steel structural frame with wide column spacing then this member takes a very special importance by the very fact that its collapse or instability will cause possible progressive collapse and failure of the entire structure. Adding that importance factor from a qualitative way this column is expected to have its normal distribution curve shifted somewhat downwards as shown by Curve "B". To give an example for this case, the diagonal members or the vertical columns in the 100 story John Hancock Center at the ground level (Figure 3) are much more critical for the entire structure than the interior columns in the same structure. From a safety point of view they should have a higher level of reliability than other compression members in that building. Conversely, in a framed tube structure such as the World Trade Center in New York or a bundled tube structure such as the Sears Tower in Chicago, the individual column elements and their performance have a lower impact on the overall stability and reliability of the total structure. Redistribution of load will occur due to any unknown premature instability of a single column element. Therefore in such a case the normal distribution curve in Figure 2 should be moved upwards qualitatively as shown in Curve "C" to take into account the high redundancy of such a structural system.

Adjustment for Redundancy or Uniqueness through LRFD Design Method

The Load and Resistance Factored Design (LRFD) method which is being widely discussed for possible acceptance for steel structural design, is based on the need for providing a safe margin between the normal distribution curve for the resistance and the normal distribution curve for the load and its effects as shown in Figure 2. The reliability index β expressed as:

$$\beta = \frac{R_m - Q_m}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

where R_m and Q_m are the mean Resistance and Load Effects and σ_R and σ_Q are the standard derivations for the Resistance and the Load Effects. The LRFD method accommodates for the Reliability Index β by the use of the Resistance Reduction Factor ϕ and the Load Effect increment factor γ_D for dead load and γ_L for live load. As shown in Figure 4 the LRFD method provides for the adequate safety margin between the resistance and the load effect by moving the normal distribution curve for resistance downward with a multiplier ϕ and moving the normal distribution for loads upwards by multiplying appropriate factors γ. This is shown by the second set of curves in Figure 4.

The effect of redundancy or uniqueness of structural elements at present are not considered or incorporated either in the present concept of factor of safeties or in the proposed LRFD method where the reliability index β is used without regard to the location or importance of any individual structural member. The authors recommend that the effect of redundancy or uniqueness of a structural member be taken into account by adjusting the ϕ factor and not the load factors thereby philosophically accepting a shift upwards or downwards for normal distribution curves for resistance only. This approach would be consistent with the general design methods used at this time.

PANEL DISCUSSION (KHAN/EL NIMEIRI)

Suggested Design Approach

In quantitative terms the upward or downward shift of the ϕ factor can be looked at in the same way as column load reductions are allowed for tall buildings based on number of floors above any level. However in the absence of a detailed research program in this subject, the design engineer may be allowed to adjust the normal ϕ factor upwards or downwards on the basis of a probabilistic analysis showing the effect of the redundancy or the uniqueness of a certain member in relation to the total structural system.

Conclusions

The paper has been presented to stimulate discussion on the importance of redundancy or uniqueness of any structural member on the total structural system. The stability and strength of individual elements should consider the type of structural system used and the location of individual members within the overall structure. A rational approach for redundant systems will undoubtedly result in savings in construction costs while recognizing the need to provide a higher level of safety for those structural elements which may cause greater damage and loss of life in the event of an accidental structural collapse.

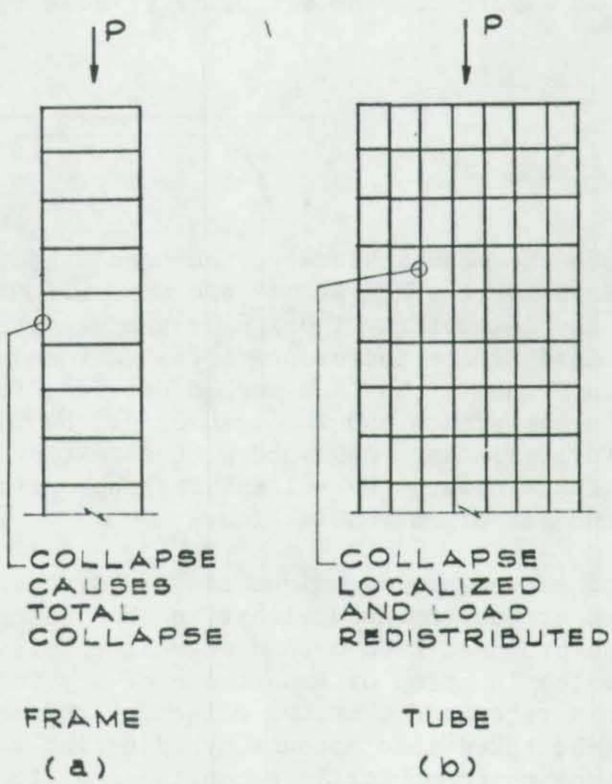


FIG. 1 EFFECT OF LOCAL COLLAPSE ON (a) FRAME (b) TUBE

PANEL DISCUSSION (KHAN/EL NIMEIRI)

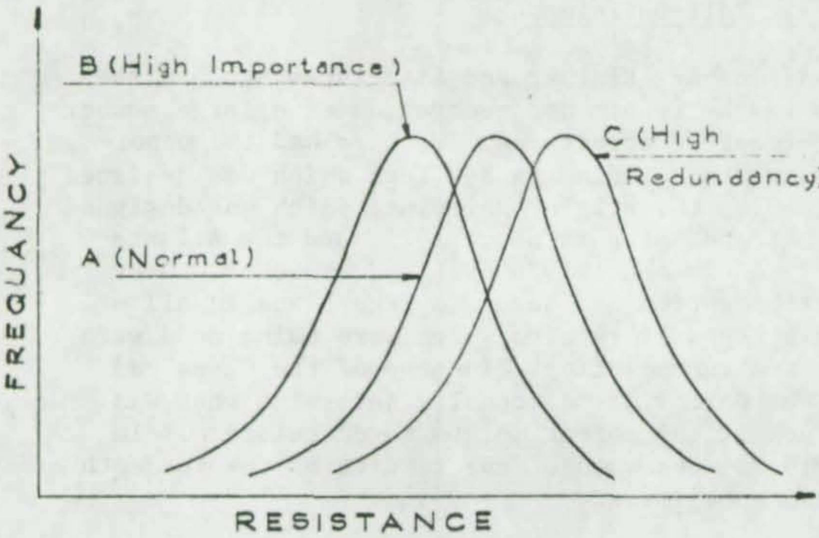


FIG 2 COMPARATIVE R FOR THREE CASES

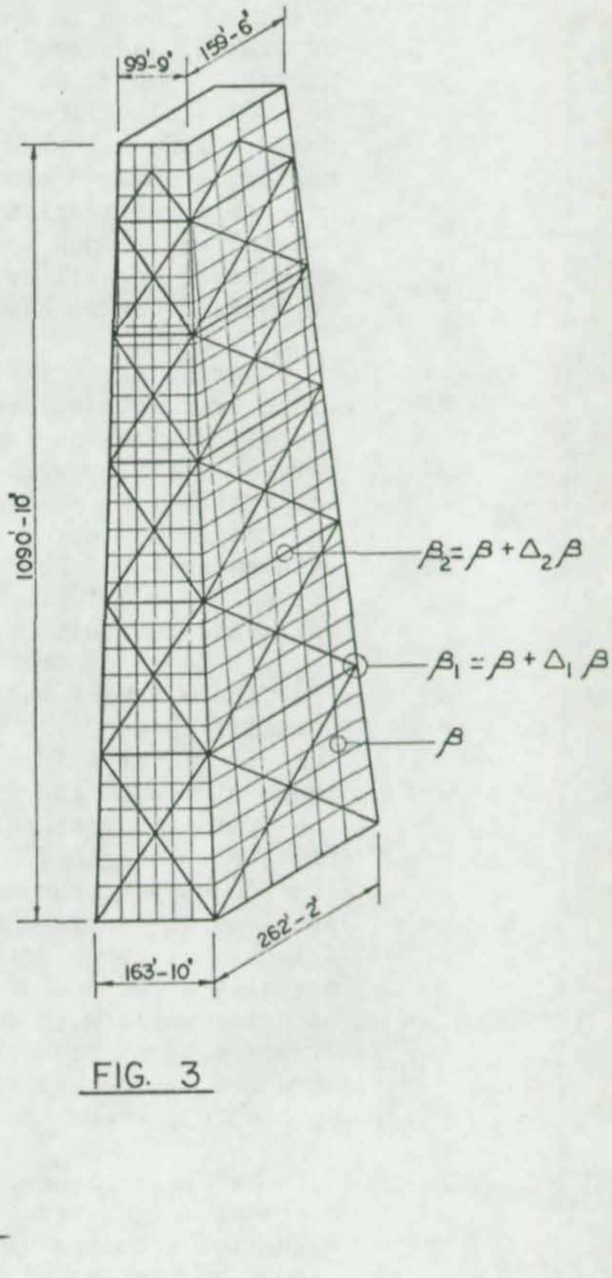
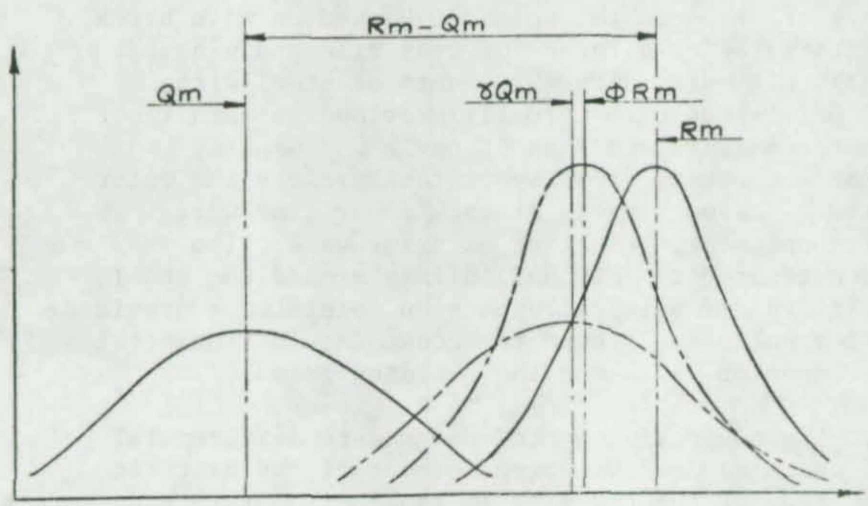


FIG. 3



$$\beta = \frac{Rm - Qm}{\sqrt{\sigma^2_R + \sigma^2_Q}}$$

$$\phi Rm \geq \gamma Qm$$

FIG. 4 LRFD DESIGN BASIS

PANEL DISCUSSION

JERRY G. STOCKBRIDGE - The Interaction Between Exterior Walls and Building Frames in Historic Tall Buildings

In recent years Wiss, Janney, Elstner and Associates has had the privilege to be involved in the repair and renovation of a large number of significant historic high-rise structures. We have had the opportunity to work on the Woolworth Building in New York which was designed by Cass Gilbert in about 1910, the Wrigley Building, which was designed by Graham, Anderson, Probst and White in about 1921, and the Atlanta City Hall, which was designed in about 1930 by Lloyd Preacher. Over the years significant deterioration had begun to take place in all of these buildings. The only types of repairs which were being done were replacing damaged stones and tuckpointing. In none of the cases had indepth investigations been performed to actually determine what was causing the stones to crack or the mortar joints to deteriorate. In this paper we will briefly discuss some of the results of the in-depth investigations we have since performed.

During the course of these investigations and renovations we discovered nothing new or revolutionary, but we have been amply impressed with the importance of designing for realistic interaction between exterior walls and building frames, which is something the early high-rise designers failed to do. A tremendous amount of time and energy was spent by the original designers on the detailing of the ornamentation of the exterior walls, but there was little or no attention at all given to the interaction between the exterior masonry walls and the building frame.

The exterior walls of these older high-rise structures were normally about 2 1/2 to 3 feet thick. The exterior walls were usually constructed of 4 inches of terra on the outside, backed up with brick, and finished on the interior with 4 inches of clay tile and plaster. The frames in most early high-rise structures were of steel with riveted connections. Outriggers were normally provided at each floor line to support the outer masonry and a shelf angle was usually provided on the face of the outrigger to specifically carry the outer 4 inches of terra cotta. The outriggers at each floor line were normally sized to carry one story height of exterior wall. The exterior walls were constructed rigidly and solidly around the steel frame members. No horizontal or vertical expansion joints were provided anywhere in the exterior walls to attempt to accommodate differential movements between the exterior walls and the building frame.

Needless to say, with no provisions to accommodate differential movement, significant cracking is developing in most of the historic high-rise structures. Some of the cracking is random in nature and casued by rusting of the steel, freeze-thaw action, etc. But by far, most of the cracking is being caused by compressive forces which have developed in the exterior walls due to the lack of accommodation of interaction between the frame and the exterior walls. The unaccommodated vertical compressive forces which have developed in the piers have occurred because of a number of factors.

PANEL DISCUSSION (STOCKBRIDGE)

First, terra cotta, like all clay materials, after firing is as small as it is ever going to be in its entire life. While in service it will have a tendency to grow. If this growth is restrained it will, of course, induce compressive stresses. Second, terra cotta, like all other materials, has a tendency to expand when it gets hot and contract when it gets cold. Therefore, on hot summer days when the exterior walls try to expand and are restrained by the steel frame, which is in a controlled environment, compressive forces again will develop. Third, because no space was provided below the shelf angles as each floor line, the angles could not deflect slightly to carry the loads intended. Stacking has taken place in the exterior walls of the building, with the lower floors carrying loads which were intended to be carried by shelf angles higher up in the building. Fourth, because the steel frame is rigidly built into the exterior walls, many of the floor loadings which were intended to be carried by the columns in the exterior walls are actually being carried by the masonry in the walls. And finally, when wind forces are applied against the side of the building, the masonry walls tend to take a lot of the forces which are intended to be taken by the steel frame because the steel frame is rigidly built into the exterior walls.

To develop repairs which properly addressed the conditions which exist, WJE has undertaken a number of strain relief testing programs on these old high-rise structures to determine the actual build-up of pressure which has taken place over the years. Strain relief tests are performed by attaching strain gages to the face of the exterior walls, taking an initial set of readings, cutting around the piece of terra cotta which contains the gages to relieve any pressure in the face of the wall, and then taking a final set of readings. A comparison of the first set of readings with a follow-up set of readings indicates the amount of strain which was released when the piece of terra cotta was cut free from the wall. Samples of the terra cotta are then removed from the wall, taken back to the laboratory, and tested to develop the Modulus of Elasticity of the material and the recorded strains in the field are converted to stresses.

If the walls in the historic structures were performing as the designers had intended, the levels of the stress at each floor line would normally be on the order of 15 lbs per square inch, assuming a 15-foot story height. The actual level of stress which we have been measuring in many of the structures runs as high as 2,000 to 3,000 psi. The actual stresses which exist in the wall may even be greater than the level of stress we are actually measuring because of the fact that it is highly unlikely that terra cotta material is completely elastic. It is much more likely that some plastic deformation has taken place over the years and that we are not getting complete recovery of the material when we cut it free.

As part of the renovation of many of these high-rise structures, we have included the installation of horizontal expansion joints into the existing building to relieve the build-up of the high compressive forces before beginning the repair of the damaged pieces. To determine the spacing required for the horizontal expansion joints, it has been our normal practice to instrument a representative pier, cut in a horizontal joint at one floor level, and measure how much the forces are relieved horizontally, both up and down from the cut.

In the case of the Atlanta City Hall, it appears that we will be able to space the expansion joints at more than one-floor intervals. On the Wrigley Building in Chicago, the joints were installed at every floor line, and at the Woolworth Building in New York, where the terra cotta was very rigidly anchored to the back-up material, it was necessary to cut in at every other joint to relieve the build-up of pressure.

In the case of the Atlanta City Hall and the Wrigley Building in Chicago, it is possible to install the joint at existing shelf angles and therefore the joints can be left open after cutting and be caulked to accommodate future movements.

On the Wrigley Building in New York, however, where there are no shelf angles at floor lines and we had to cut into the wall at every other course, it was impossible to install expansion joints to accommodate future movements. In the Woolworth Building, it was necessary to refill the joints with mortar again after they had been cut out. The mortar in the joints did not create a condition capable of accommodating future movements without the development of some stress, but the cutting out of the joints did relieve some stresses which are unlikely to ever develop again. These were the stresses induced by the moisture expansion of the brick, those associated with stacking, and the loads which were transferred in from the floor system.

It is true that some compressive forces will continue to occur in the future related to thermal effects and wind forces, but these forces should be significantly lower than those which the building had to tolerate in the past and we anticipate that the performance of the building will be substantially improved.

1981 ANNUAL BUSINESS MEETING

The Structural Stability Research Council holds an Annual Business Meeting for the purpose of reporting activities, election of officers and presentation of the following year's proposed budget for approval by the membership. The 1981 Annual Business meeting was held on April 8, 1981, in conjunction with the Annual Technical Session at the Conrad Hilton Hotel, Chicago.

The minutes of the 1981 Annual Business Meeting follow:

CALL TO ORDER

The meeting was called to order at 11:25 a.m. by the Chairman, J. S. B. Iffland. Approximately 50 persons were present.

The Chairman expressed the Council's appreciation for the National Science Foundation's support of the conference.

ELECTION OF OFFICERS

The Nominating Committee, chaired by G. Haaijer, nominated J. S. B. Iffland for a one-year extension of his term as Council Chairman, and J. Springfield for a one-year term as Council Vice Chairman.

Voting was conducted by letter ballot to the membership. The nominees were elected, effective 1 October 1981.

ELECTION OF EXECUTIVE COMMITTEE MEMBERS

The Nominating Committee nominated incumbants G. F. Fox, R. R. Graham and B. G. Johnston for three year terms on the Executive Committee.

Voting was conducted by letter ballot to the membership. The nominees were elected, effective immediately.

MEMBERS-AT-LARGE

The following persons were nominated for election to Member-at-Large:

S. Kitipornchai, University of Queensland
T. H. Lin, UCLA
J. W. Cox, Terra Inc.
Z. Y. Shen, Lehigh University

The motion that the nominees be elected as Member-at-Large was carried unanimously.

BYLAWS

J. L. Durkee, Chairman of the Bylaws Committee, reported that a new draft and ballot would be forthcoming to the membership which takes into account suggestions and comments received on the previous ballot. A Standing Committee on Technical Sessions Programs has also been added.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director on behalf of the Finance Committee Chairman, G. F. Fox. The proposed budget for fiscal 1981-82 was also presented and approved.

Budget 1981-82 Summary:

Expected balance, 1 Oct 1981	$28,408
Income	24,400
Expenditures	44,500
Expected balance, 30 Sep 1982	$ 8,308

Concern was expressed over the reduction of "seed money" due to inflation. Increased revenues will be sought.

DIRECTOR'S REPORT

L. S. Beedle, Council Director, summarized the Executive Committee activities and concerns, and highlighted the task group activities. Future task group activity will be centered around the preparation of the 1st chapter drafts of the 4th Edition of the "Guide". TG-11 will be engaged in the preparation of the 3rd International Colloquium. This colloquium will follow a similar "travelling" format used by the 2nd Colloquium and is tentatively scheduled to begin in Paris in the fall of 1982.

The Director thanked J. L. Durkee, outgoing Council Vice Chairman, for the splendid job he has done while in office. Mr. Durkee has been appointed Council Treasurer (without salary) to assist in the handling of the Council's funds.

The Director also thanked J. S. B. Iffland, SSRC Chairman, Z. Y.Shen, SSRC Technical Secretary, and L. G. Federinic, SSRC Administrative Secretary, for their dedication and the fine work they continue to perform.

NEXT ANNUAL TECHNICAL SESSION AND MEETING

The Chairman announced that the next Annual Technical Session & Meeting will be held at The Rault Center (formerly Holiday Inn Superdome) in New Orleans. The dates will be 30-31 March 1982. The theme will be "Stability of Offshore Structures".

ADJOURNMENT

The meeting was adjourned at 12:00 noon.

1981 ANNUAL TECHNICAL SESSION & MEETING ATTENDANCE

Participant	Affiliation
Abrahams, M. J.	Parsons, Brinckerhoff, Quade, & Douglas, New York
Austin, W. J.	Rice University, Houston, TX
Beedle, L. S.	Lehigh University, Bethlehem, PA
Benito, R.	Washington University (Student), St. Louis, MO
Bernstein, M. D.	Foster Wheeler Energy Corp., Livingston, NJ
Birkemoe, P. C.	University of Toronto, Toronto, Ontario
Biswas, M.	Texas A & M University, College Station, TX
Bjorhovde, R.	University of Alberta, Edmonton, Alberta
Broderick, J. R.	Lockwood, Andrews & Newnam, Inc., Houston, TX
Capanoglu, C.	Earl and Wright Consulting Engineers, San Francisco, CA
Cescotto, S.	University of Liege, Belgium
Chan, H. K. P.	Dominion Bridge Co., Ltd., Ottawa, Ontario
Chang, J. G.	University of Notre Dame (Student), Notre Dame, IN
Chen, A. C. T.	Exxon Production Research Co., Houston, TX
Chen, S. S.	Argonne National Laboratory, Argonne, IL
Chen, W. F.	Purdue University, West Lafayette, IN
Del Valle, E.	National University of Mexico, Ciucad Universitaria
Disque, R. O.	American Institute of Steel Construction, Chicago, IL
Durkee, J. L.	Consulting Structural Engineer, Bethlehem, PA
Ellifritt, D. S.	Metal Building Manufacturers Assoc., Cleveland, OH
Errera, S. J.	Bethlehem Steel Corp., Bethlehem, PA
Estenssoro, L. F.	Wiss, Janney, Elstner & Assoc., Northbrook, IL
Federinic, L. G.	Lehigh University, Bethlehem, PA
Fleischer, W. H.	Bethlehem Steel Corp., Bethlehem, PA
Foss, G.	Det Norske Veritas, Oslo, Norway
Fox, G. F.	Howard, Needles, Tammen & Bergendoff, New York
Galambos, T. V.	Washington University, St. Louis, MO
Gallagher, R. H.	University of Arizona, Tuscon, AZ
Gouwens, A. J.	Goodell-Grivas, Southfield, MI
Graham, R. R.	U. S. Steel Corp. Pittsburgh, PA
Grove, R.	Chicago Bridge & Iron Co., Plainfield, IL
Gunzelman, S. X.	Brown & Root, Inc., Houston, TX
Haaijer, G.	U. S. Steel Corp., Monroeville, PA
Higgins, T. R.	Consultant, Epsom, NH
Hotchkies, J. W.	The Algoma Steel Corp., Ltd., Toronto, Ontario
Howerton, W.	University of Wisconsin - Madison (Student), Madison, WI
Hu, X. R.	Lehigh University, Bethlehem, PA

Iffland, J. S. B.	Iffland Kavanagh Waterbury, New York
Iqbal, M.	University of Chicago (Student), Niles, IL
Johnson, A. L.	American Iron & Steel Institute, Washington, D. C.
Johnson, D. J.	Butler Manufacturing Co., Grandview, MO
Johnston, B. G.	Consultant, Tuscon, AZ
Kam, T. Y.	Northwestern University (Student), Evanston, IL
Kassimali, A.	Southern Illinois University, Carbondale, IL
Ketter, R. L.	State University of New York at Buffalo, Buffalo, NY
Khan, F.	Skidmore, Owings & Merrill, Chicago, IL
Koo, B.	University of Toledo, Toledo, OH
Krajcinovic, D.	University of Illinois at Chicago Circle, Chicago, IL
Kulak, M.	U. S. Steel Research, Monroeville, PA
LaBoube, R. A.	Butler Manufacturing Co., Grandview, MO
Lang, G. R.	Mobil Research & Development Corp., Dallas, TX
Lee, G. C.	State University of New York at Buffalo, Buffalo, NY
LeMessurier, W. J.	LeMessurier Assoc./SCI, Cambridge, MA
Lu, L. W.	Lehigh University, Bethlehem, PA
Marsh, C.	Concordia University, Montreal, Quebec
McDermott, R.J.	Lehigh University (Student), Bethlehem, PA
Meith, R. M.	Chevron, U.S.A., New Orlean, LA
Miller, C. D.	Chicago Bridge & Iron Co., Plainfield, IL
Mitchell, J.A.	Combustion Engineering Inc., Windsor, CT
Mohamed Aly, H. N.	University of Notre Dame (Student), Notre Dame, IN
Murray, T. M.	University of Oklahoma, Norman, OK
Nair, R. S.	Alfred Benesch & Co., Chicago, IL
Palmer, F. J.	Copperweld Tubing Group, Pittsburgh, PA
Parmelee, R. A.	Northwestern University, Evanston, IL
Pekoz, T.	Cornell University, Ithaca, NY
Pillai, S. U.	Royal Military College of Canada, Kingston, Ontario
Plaut, R. H.	Virginia Polytechnic Institute & State University, Blacksburg, VA
Popov, E.	University of California, Berkeley, CA
Regl, R. R.	McDermott, Inc., New Orleans, LA
Ross, D. A.	University of Akron, Akron, OH
Rossow, E. C.	Northwestern University, Evanston, IL
Schwaighafer, J.	University of Toronto, Toronto, Ontario
Shen, S. Z.	Lehigh University, Bethlehem, PA
Shen, Z. Y.	Lehigh University, Bethlehem, PA
Sherman, D. R.	University of Wisconsin, Milwaukee, WI
Springfield, J.	Carruthers & Wallace, Rexdale, Ontario
Sridharan, S.	Washington, University, St. Louis, MO
Stafford-Smith, B.	McGill University, Montreal, Quebec
Stockbridge, J. G.	Wiss, Janney,Elstner, & Assoc., Northbrook, IL
Tung, D. H. H.	The Cooper Union, New York

Vinnakota, S.	University of Wisconsin, Milwaukee, WI
Wales, M. W.	Northwestern University (Student), Evanston, IL
Wang, C. K.	University of Wisconsin - Madison, Madison, WI
Wang, S. T.	University of Kentucky, Lexington, KY
Winter, G.	Cornell University, Ithaca, NY
Yoo, C. H.	Marquette University, Milwaukee, WI
Yu, W. W.	University of Missouri - Rolla, Rolla, MO
Zellin, M. A.	Sverdrup & Parcel and Assoc., Inc., St. Louis, MO

SSRC Chronology

19-20 Oct 80	- Executive Committee Meetings, Pittsburgh, PA
18-19 Nov 80	- Cooperating Sponsor - 5th International Specialty Conference on Cold-Formed Steel Structures, St. Louis, MO
1 Jan 81	- Mr. Zu-Yan Shen assumed duties of SSRC Technical Secretary
9 Jan 81	- Chairman's Meeting, Lehigh University, Bethlehem, PA
23 Jan 81	- Program Sessions Committee Meeting, Lehigh University, Bethlehem, PA
12 Feb 81	- Finance Committee Meeting, New York City
5-8 Apr 81	- Annual Technical Session & Meeting; Executive Committee Meetings; Task Group Meetings; Chicago, IL
8 Apr 81	- Mr. J. L. Durkee appointed SSRC Treasurer (unsalaried)
12 May 81	- B. G. Johnston Symposium and Banquet, New York City SSRC - FERS cosponsorship
14 May 81	- SSRC Guide Committee Meeting, New York City
1 Aug 81	- Dr. Gulay Askar assumed duties of SSRC Technical Secretary

List of Publications

The following papers and reports have been received at Headquarters and have been placed in the SSRC library.

Bjorhovde, R.
RESEARCH NEEDS IN STABILITY OF METAL STRUCTURES, Journal of the Structural Division, ASCE, Vol. 106, No. ST12, Proc. Paper 15926, December, 1980

Chen, S. S.
STABILITY OF CIRCULAR CYLINDRICAL STRUCTURES SUBJECT TO FLUID FLOW, SSRC Annual Technical Session, Chicago, April, 1981

European Convention for Constructural Steelwork
EUROPEAN RECOMMENDATIONS FOR STEEL CONSTRUCTION - SECTION 4.6 - BUCKLING OF SHELLS, The Construction Press, 1981

Fritz Engineering Laboratory
LIST OF PUBLICATIONS 1980, Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania, 18015

Johnston, B. G.
BRUCE JOHNSTON SELECTED PAPERS, Fritz Engineering Research Society, SSRC, 1981

Johnston, B. G.
HISTORY OF STRUCTURAL STABILITY RESEARCH COUNCIL, reprint, Journal of the Structural Division, ASCE, Vol. 107, ST8, Proc. Paper 16451, August, 1981

Lee, G. C. Ketter, R. L. and Hsu, T. L.
DESIGN OF SINGLE STORY RIGID FRAMES, Metal Building Manufacturers Association, Cleveland, Ohio, 1981

Structural Stability Research Council
TECHNICAL MEMORANDUM NO. 5 - GENERAL PRINCIPLES FOR THE STABILITY DESIGN OF METAL STRUCTURES, Civil Engineering - ASCE, February, 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 (PART 1), Engineering Journal, AISC, Third Quarter, 1981, Vol. 18, No. 3, pp. 90-125

Galambos, T. V.
U.S. - JAPAN SEMINAR ON INELASTIC INSTABILITY OF STEEL STRUCTURES AND STRUCTURAL ELEMENTS, 25-29 May, 1981 at Sasagawa Kinenkaikan, Tokyo, Japan

Yu, W. W. and Senne, J. H. (eds.)
RECENT RESEARCH AND DESIGN TRENDS IN COLD-FORMED STEEL STRUCTURES, Fifth International Specialty Conference on Cold-Formed Steel Structures, held in St. Louis, Missouri, November 18-19, 1980

Finance

	Fiscal Year 10/80-9/81		Fiscal Year 10/81-9/82
	Budget (approved 4/30/80)	Cash Statement 10/1/80-9/30/81	Budget (approved 4/8/81)
BALANCE at Beginning of Period	$19,700.00	$16,108.88 (a)	$28,408.00
INCOME			
Contributions			
Sponsors			
AISC	4,000.00	4,000.00	4,000.00
AISI	5,000.00	---	5,000.00
API	100.00	1,000.00	1,000.00
CISC	1,000.00	1,000.00	1,000.00
MBMA	1,000.00	1,000.00	1,000.00
Participating Organizations	1,900.00	1,500.00 (b)	2,000.00
Participating Firms	2,500.00	5,200.00 (c)	4,500.00
Annual Meeting (81)	14,000.00	14,500.00 (d)	---
Annual Meeting (82)	---	17,900.00 (e)	---
Total Contributions	$29,500.00	$46,100.00	$18,500.00
Registration (Annual Meeting)	3,000.00	3,872.41	4,500.00
Member-at-Large Fees	1,800.00	50.00 (f)	200.00
Guide Royalties	500.00	1,355.34	500.00
Sale of Publications	---	100.31	500.00
Interest	200.00	1,102.17	200.00
TOTAL INCOME	$35,000.00	$52,580.23	$24,400.00
EXPENDITURES			
Technical Services (Hqtrs)			
Staff Salaries	18,000.00	14,317.72 (g)	20,300.00
Supply, phone, mailing	1,600.00	1,736.07	1,800.00
Travel	700.00	692.81	500.00
Total Technical Services	$20,300.00	$16,746.60	$22,600.00
Research Support	5,000.00	2,000.00 (h)	2,000.00
Annual Meeting & Proceedings			
Annual Proceedings	2,200.00	2,676.00	3,000.00
Expenses & Services	7,000.00	5,864.30	7,500.00
Travel	5,000.00	4,498.52	6,000.00
Total Annual Mtg & Proceedings	$14,200.00	$13,038.82	$16,500.00
SSRC Guide (4th Edition)			
Expenses & Services	3,000.00}	210.45	{ 1,500.00
Travel		1,186.21	
Total SSRC Guide	*$ 3,000.00*	*$ 1,396.66*	*$ 1,500.00*
United Engineering Trustees	100.00	100.00	100.00
Travel	1,000.00	619.65	1,000.00
Publications	---	2,095.00 (i)	600.00
Contingencies	200.00	1,034.43 (j)	200.00
TOTAL EXPENDITURES	$43,800.00	$37,031.16	$44,500.00
BALANCE at End of Period	$10,900.00	$31,657.92 (k)	$ 8,308.00

EXPLANATORY NOTES

(a) Depositories (as of 10/1/80)

General Account (UET)	\$12,196.26
Technical Services (Lehigh Univ.)	961.97
4th Edition Guide Account	2,934.08
FHWA Grant (New York City ATS&M)	16.57
	\$16,108.88

(b) Aluminum Association (\$500); American Society of Mechanical Engineers (\$100); Corps of Engineers, U.S. Army (\$100); European Convention for Constructional Steelwork (\$100); Federal Highway Administration (\$100); Institution of Engineers, Australia (\$100); International Conference of Building Officials (\$100); Langley Research Center, NASA (\$100); Naval Ship Research and Development Center, U.S. Navy (\$100); Steel Joist Institute (\$200)

(c) \$100 each - Amirikian Engineering Company; Ammann & Whitney; Balke Engineers; Basil Engineering Corporation; Alfred Benesch & Company; Martin Berkowitz Associates; Blauvent Engineering Company (\$200); Butler Manufacturing Company; Carruthers and Wallace Limited; Caudill Rowlett Scott, Inc.; Copperweld Tubing Group; DRC Consultants, Inc.; Delon Hampton & Associates; Dravo Van Houten, Inc.; Edwards and Kelcey, Inc.; Gannett Fleming Corddry and Carpenter, Inc.; Gilbert Associates, Inc.; Green International, Inc.; Hardesty & Hanover; Hazelet & Erdal; Howard Needles Tammen & Bergendoff; Iffland Kavanagh Waterbury, P.C.: Bernard Johnson Incorporated; LeMessurier Associates/SCI; Lev Zetlin Associates, Inc.; A. G. Lichtenstein & Associates, Inc.; Lockwood, Andrews & Newnam, Inc.; Loomis and Loomis, Inc.; Chas. T. Main, Inc.; Modjeski and Masters; Walter P. Moore & Associates, Inc.; Parsons, Brinckerhoff, Quade and Douglas, Inc.; Richardson, Gordon and Associates; Rummell, Klepper & Kahl (\$200); Sargent & Lundy; Seelye, Stevenson, Value & Knecht, Inc.; Skidmore, Owings & Merrill (\$300); Skilling, Helle, Christiansen, Robertson, P.C.; Steinman, Boynton, Gronquist & Birdsall; Sverdrup & Parcel and Associates, Inc.; Tippetts-Abbett-McCarthy-Stratton; URS/John A. Blume & Associates; URS Company; Vollmer Associates, Inc. (\$200); Weiskopf & Pickworth; Wiss, Janney, Elstner and Associates, Inc. (\$200).

(d) A grant received from the National Science Foundation in support of the 1981 Annual Technical Session & Meeting in Chicago.

(e) Grants received from Brown & Root, Inc.; Chevron U.S.A. Inc.; Chicago Bridge & Iron Company; Earl & Wright; McDermott Incorporated; Mobil Research & Development Corp; and Shell Oil Company in support of the 1982 Annual Technical Session & Meeting in New Orleans.

(f) Member-at-Large canvass for 1981 deferred due to proposed Bylaws changes.

EXPLANATORY NOTES - cont'd

(g) Technical Services (Hqtrs)

	SSRC FUNDS	NSF 1981 ATS&M
Director	$1,815.53	
Technical Secretary	947.30	
Administrative Secretary	5,120.09	$3,501.44
Secretarial/Clerical	1,713.18	1,220.18
(includes employee benefits)		
	$9,596.10	$4,721.62

(h) $2000 grant to Texas A&M University CE Department (Biswas)

(i) SSRC History reprints; BGJ Anniversary Volume

(j) Executive Committee approved a stipend to Johnston for his work in connection with the SSRC History; SSRC Guide to departing Technical Secretary.

(k) Depositories (as of 9/30/81)

General Account (UET)	$ 2,773.20
Technical Services (Lehigh Univ)	685.34
4th Edition Guide Account	2,892.76
NSF Grant (1981 ATS&M Chicago)	4,325.11
New Orleans Grant (1982 ATS&M)	6,379.83
Capital Preservation Fund	14,601.71
	$31,657.92

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TASK GROUPS

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R. Bjorhovde, Chairman	J. L. Durkee	B. G. Johnston
L. S. Beedle	J. A. Gilligan	C. Marsh
W. F. Chen	R. R. Graham*	T. Pekoz
J. W. Clark	D. H. Hall	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

J. Springfield, Chairman*	L. W. Lu	S. U. Pillai
M. J. Abrahams	D. A. Nethercot	Z. Razzaq
W. F. Chen		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*	H. de Clercq	L. W. Lu
P. F. Adams	A. J. Gowens	W. A. Milek
C. Birnstiel	P. Grundy	Z. Razzaq
M. Biswas	I. M. Hooper	C. K. Wang
F. Y. Cheng	T. Kanchanalai	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

T. Pekoz, Chairman	S. J. Errera*	H. H. Spencer
P. C. Birkemoe	B. G. Johnston	D. R. Sherman
R. Bjorhovde		L. Tall

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)

A. Amirikian, Chairman	C. R. Felmley, Jr.	D. L. Johnson
G. C. Lee, Vice Chairman	R. R. Graham*	G. W. Oyler
D. S. Ellifritt	N. Iwankiw	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, Chairman	B. G. Johnston*	M. A. J. G. da Silva
J. Amazigo	R. H. Plaut	G. J. Simitses
S. S. Chen	D. Shilkrut	J. C. Simonis
S. M. Holzer		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	M. Crainicescu	P. Marek
W. A. Milek, V. Chairman*	T. V. Galambos	J. J. Melcher
G. A. Alpsten	M. P. Gaus	G. W. Schultz
L. S. Beedle	O. Halasz	J. Strating
A. Carpena	J. S. B. Iffland	L. Tall
J. H. Chen	B. Kato	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	A. Gjelsbik	L. W. Lu
G. A. Alpsten	A. L. Johnson	E. P. Popov
G. F. Fox	B. G. Johnston	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	T. M. Murray	S. Sridharan
S. J. Errera	A. Ostapenko	W. P. Vann
A. L. Johnson	T. Pekoz	S. T. Wang
C. Marsh		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	C. G. Culver	W. A. Milek, Jr.
R. Behling	J. L. Durkee*	M. Ojalvo
H. R. Brannon	E. R. Latham	W. M. Thatcher
A. P. Cole	P. M Marek	

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

J. A. Yura, Chairman	A. J. Hartman	D. A. Nethercot
Y. Fukumoto	S. Kitipornchai	M. Ojalvo
T. V. Galambos*	C. P. Mangelsdorf	N. S. Trahair

Scope: To study the behavior of and 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

W. Hsiong, Chairman	E. Karamuk	H. H. Spencer
P. B. Cooper	C. Massonnet	B. T. Yen
J. L. Durkee*	A. Ostapenko	R. C. Young
R. S. Fountain	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

A. Chajes, Chairman	W. K. Gillespie	N. F. Morris
W. J. Austin*	S. X. Gunzelman	E. P. Popov
A. C. T. Chen	D. Krajcinovic	D. T. Sherman
M. Crainicescu	C. D. Miller	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

D. R. Sherman, Chairman	S. L. Chin	P. W. Marshall
B. O. Almroth	J. W. Cox	R. M. Meith*
M. D. Bernstein	E. D. George, Jr.	C. D. Miller
P. C. Birkemoe	R. R. Graham	F. J. Palmer
C. Capanoglu	S. X. Gunzelman	R. Regl
A. Chajes		D. A. Ross

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	J. W. Roderick	M. Wakabayashi
P. J. Dowling	D. Sfintesco	G. Winter*
R. W. Furlong	B. Taranath	J. Zils
B. Kato		

Scope: To develop stability criteria for various types of composite columns, beam-columns and mixed steel-concrete systems.

Task Group 21 - Box Girders

R. C. Young, Chairman	D. R. Schelling	M. C. Tang
G. F. Fox*	F. D. Sears	D. H. H. Tung
F. Moolani	H. H. Spencer	R. Wolchuk
B. Morgenstern		

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	P. J. Dowling	R. K. Kinra
C. Babcock	N. W. Edwards	R. M. Meith*
M. D. Bernstein	G. Foss	K. Minhaus
C. Capanoglu	S. X. Gunzelman	R. Regl
J. W. Cox	E. H. Killam	G. J. Simitses
R. C. DeHart		

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 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	T. V. Galambos	D. A. Ross
M. H. Ackroyd	J. S. B. Iffland	J. Springfield
R. Bjorhovde	B. Koo	S. Vinnakota
F. Cheong-Siat-Moy	D. A. Nethercot	G. Winter*
R. O. Disque	Z. Razzaq	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. Monsour, Monsour Engineering

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

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B. Thurlimann	1976
G. Winter	1976

A D D E N D U M

The following people are added to the mailing list. Some have been added just recently; others were inadvertently omitted.

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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.

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*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.

C O U N C I L   M E M B E R S H I P

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.

F E E S

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.

E L E C T I O N S

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.