STRUCTURAL STABILITY RESEARCH COUNCIL

(Formerly Column Research Council—Established in 1944)

Proceedings 1976

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The Council has its Headquarters at:
Fritz Engineering Laboratory #13
Lehigh University
Bethlehem, Pennsylvania 18015

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FOREWORD

The past year has been an unusually important one for the Council.

The most significant event was the publication of the third edition of the Council's Guide. This important and influential book has changed greatly both in coverage and in size. The widened scope is reflected in the change of title which, for the first two editions had been: Guide to Design Criteria for Metal Compression Members. Recognizing that the coverage now goes far beyond the area of compression members, the third edition is entitled: Guide to Stability Design Criteria for Metal Structures. The third edition's 19 chapters of 616 pages compare with the previous edition's 7 chapters of 217 pages. All this reflects the fact that under Dr. B.G. Johnston's able leadership as editor, a volume has been produced which covers just about every imaginable stability situation in a manner specifically oriented toward design application. All the Council's task groups have actively contributed to the writing of this volume. This edition is, therefore, truly the collective product of the entire Council.

The second major development, this past year, is closely related to the first, namely the adoption of a new name for the Council. What used to be for so many years the Column Research Council, by overwhelming vote of the membership has become the Structural Stability Research Council. This new name simply reflects the widening of the Council's field of activity which now, in addition to metal compression members, covers beams, cold-formed thin-walled members, plate girders, horizontally-curved members, shell-like structures, rigid frames, composite members, to mention only some of the fields.

With this name change it is possible that in the future the Council's interests will spread to other than metal structures. In fact, as possibly a first step in this direction, composite construction is now included in the list of task groups as well as, in a special chapter, in the third edition of the Guide.

This third edition now having been published, the activities of the Council for the next several years, will be oriented in a somewhat different direction. That is, more attention will be paid to direct involvement in, and cognizance of, ongoing research in the area of structural stability. One visible result of this stronger orientation toward ongoing research is the formation of a new task group on research priorities, chaired by J.S.B. Iffland.

A brief word about finances, always a somewhat touchy item. At the suggestion of the chairman of the finance committee, G.F. Fox, for the double purpose of widening our professional interactions and our financial base, a new category of membership, viz. participating firms, has been initiated. So far about twenty consulting engineers' organizations have joined the Council on this basis and support it with their contributions.

Finally, after a new category "life members" had been previously instituted, by membership vote at the annual business meeting the following were awarded this title in recognition of their many years of active contribution to the work of the Council: W.J. Austin, L.S. Beedle, E.L. Erickson, E.H. Gaylord, J.A. Gilligan, J.E. Goldberg, T.R. Higgins, N.J. Hoff, S.C. Hollister, L.K. Irwin, B.G. Johnston, T.C. Kavanagh, R.L. Ketter, N.M. Newmark, B. Thurlimann, and G. Winter.

This Foreword to our annual Proceedings would not be complete without an expression of the chairman's appreciation and gratitude for the dedicated and effective work of our director, Dr. Lynn S. Beedle, and of our secretary, Dr. Francois Cheong-Siat-Moy.

George Winter

George Winter
Chairman, Structural Stability
Research Council
Ithaca, New York
October 1976

SSRC Executive Committee

G. Winter, Chairman	- Cornell University
J. W. Clark, Vice Chairman	- Aluminum Company of America
L. S. Beedle, Director	- Lehigh University
W. J. Austin	- Rice University
K. P. Buchert	- Bechtel Power Corporation
J. L. Durkee	- Consulting Structural Engineer
S. J. Errera	- Bethlehem Steel Corporation
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T. V. Galambos	- Washington University, St. Louis
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T. R. Higgins	- Consultant, AISC
J. S. B. Iffland	- Iffland Kavanagh Waterbury
B. G. Johnston	- University of Arizona
W. A. Milek, Jr.	- American Institute of Steel Construction
J. Springfield	- C. D. Carruthers & Wallace, Ltd.

l to r: L. S. Beedle, F. Cheong-Siat-Moy, G. Winter, J. W. Clark, L. A. Boston, R. M. Meith, R. R. Graham (not shown), J.S.B. Iffland, G. F. Fox, R. Bjorhovde, J. Springfield, and W. A. Milek, Jr.

Annual Technical Session

One of the purposes of the Council is to maintain a forum where problems related to the design and behavior of columns and other compression elements in metal structures can be presented for evaluation and discussion. The Annual Technical Session provides an opportunity to carry out this function.

The 1976 Annual Technical Session was held on March 2 and 3 at the Sheraton-Biltmore Hotel in Atlanta, Georgia. Fifty-five persons attended the session and nineteen papers were delivered.

A panel discussion on "New Ideas on Stability of Multistory Buildings" was held in the evening of March 2, 1976. the panelists were D. Ruby, J. Springfield and W. J. LeMessurier. The moderator was J. S. B. Iffland.

In conjunction with the Technical Session, an Annual Business Meeting was held for the purpose of electing new members, and to discuss 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, March 2, 1976

8:00 a.m. - Registration

8:30 a.m. - MORNING SESSION

Presiding: W. A. Milek, Jr., American Institute of Steel Construction

INTRODUCTION

G. Winter, Chairman, SSRC

TASK GROUP REPORTS

Task Group 1 - Centrally Loaded Columns

Chairman, J. A. Gilligan, U. S. Steel Corporation
Vice Chairman, R. Bjorhovde, The University of Alberta

Buckling of Stayed Columns
M. C. Temple, University of Windsor

Task Group 7 - Tapered Members

Chairman, A. Amirikian, Amirikian Engineering Company
(G. C. Lee, State University of New York at Buffalo presiding)

Design of Tapered Beam-Columns

M. L. Morrell, Clemson University
G. C. Lee, State University of New York at Buffalo

Non-linear Analysis of Towers and Stacks

A. V. duBouchet, Rutgers University
M. Biswas, Stone and Webster Corporation

10:00 a.m. - BREAK

Task Group 13 - Thin-Walled Metal Construction

Chairman, S. J. Errera, Bethlehem Steel Corporation

Wrinkling Instability in Sandwich Columns with Cold-Formed Steel Facings

K. P. Chong, University of Wyoming
P. C. Liu, University of Wyoming

AISC Buckling Formulas as Constraints in Optimal Design of Welded Stiffened Girders

R. T. Douty, University of Missouri-Columbia

11:15 a.m. - BREAK

Task Group 15 - Laterally Unsupported Beams

Chairman, T. V. Galambos, Washington University

Design Rules for the Lateral Buckling of Steel Beams

D. A. Nethercot, The University of Sheffield

Lateral Buckling Calculations for Braced Beams

D. A. Nethercot, The University of Sheffield

Laterally Unsupported Beams

T. V. Galambos, Washington University

Design of Laterally Unsupported Beams

J. A. Yura, University of Texas at Austin

12:30 p.m. - LUNCH

1:30 p.m. - AFTERNOON SESSION

Presiding: J. W. Clark, Aluminum Company of America

Task Group 4 - Frame Stability and Effective Column Length

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

Non-linear Large Deflection Analysis of Geometric Unstable Framed Structures

K. P. Chong, University of Wyoming
D. C. Kunkee, U. S. Steel Corporation

Lateral Instability and Multi-story Frame Design

L. W. Lu, Lehigh University
F. Cheong-Siat-Moy, Lehigh University

Buckling of Space Frames

Z. Razzaq, Arizona State University

2:30 p.m. - BREAK

Task Group 18 - Unstiffened Tubular Members

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

Post Buckling Behavior of Tubular Truss Elements

P. W. Marshall, Shell Oil Company

Tests of Axially Loaded Fabricated Cylinders

W. F. Chen, Lehigh University
D. Ross, Lehigh University

Compressive Strength of Hollow Structural Sections

P. C. Birkemoe, University of Toronto

4:15 p.m. - ADJOURN

5:30 p.m. - EVENING SESSION

PANEL DISCUSSION

New Ideas on Stability of Multistory Buildings

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

Panelists:

W. J. LeMessurier, LeMessurier & Associates/SCI
J. Springfield, C. D. Carruthers & Wallace, Ltd.
D. Ruby, John Portman & Associates

Wednesday, March 3, 1976

8:30 a.m. - MORNING SESSION

Presiding: C. Birnstiel, Consulting Engineer

Task Group 3 - Columns with Biaxial Bending

Chairman, J. Springfield, C. D. Carruthers and Wallace, Ltd.

Analysis of a Fabricated Tubular Column Subject to Biaxial Bending

W. F. Chen, Lehigh University
D. Ross, Lehigh University

Task Group 16 - Plate Girders

Chairman, F. D. Sears, U. S. Department of Transportation

Ultimate Strength of Longitudinally and Transversely Stiffened Ship Bulkheads Under Inplane Loads

M. O. Critchfield, D. W. Taylor Naval Ship Research and Development Center

A Review of the So-called Incomplete Diagonal Tension Engineering Theories for Steel Plate Girder Ultimate Strength Design Under Pure Shear

M. Elgaaly, Bechtel Power Corporation

10:00 a.m. - BREAK

Task Reporter 11 - Stability of Aluminum Structural Members

J. W. Clark Aluminum Company of America

Effective Width vs. Average Stress Method of Designing Thin Gauge Structures

J. W. Clark, Aluminum Company of America

SSRC Guide

Chairman, E. H. Gaylord, University of Illinois

Report of Third Edition of SSRC Guide

B. G. Johnston (Editor), University of Arizona

Special Report

Stability Research in the Soviet Union

L. W. Lu, Lehigh University

11:00 a.m. - SSRC ANNUAL BUSINESS MEETING

11:30 a.m. - ADJOURN

M. C. Temple, R. Bjorhovde, W. A. Milek, Jr.

TASK GROUP REPORTS

TASK GROUP 1 - CENTRALLY LOADED COLUMNS

Chairman, J. A. Gilligan, U. S. Steel Corporation
Vice Chairman, R. Bjorhovde, American Institute of Steel Construction

Buckling of Stayed Columns

M. C. Temple, University of Windsor

Introduction. It has been shown that the elastic buckling load of metal columns can be increased many times by reinforcing the column with rigidly connected crossarm members and pretensioned stays. Such columns will be referred to as "stayed columns." Fig. 1 illustrates a single crossarm stayed column. The crossarms for the stayed columns being considered are arranged in a cruxiform manner. Buckling is confined to one plane, a plane containing one of the crossarms. Two procedures for determining the elastic buckling load for stayed columns are presented. One is based on the finite element method, while the second is an "exact" solution using stability functions.

Finite Element Method. From the finite element approach a stiffness relationship, $[K]\ \{\Delta\} = \{F\}$, is obtained. $[K]$ is the stiffness matrix of the complete structure, $\{\Delta\}$ the vector of nodal displacements, and $\{F\}$ the vector of disturbing forces. Including the appropriate nonlinear terms in the strain-displacement relations results in a stiffness matrix which is $[K] = [K_E] + [K_G]$ where $[K_E]$ is the conventional elastic stiffness matrix, and $[K_G]$ is the geometric stiffness matrix. $[K_G]$ can be written as $[K_G] = \lambda[K_G^*]$ where $[K_G^*]$ is the geometric stiffness matrix for a unit load. Thus at the critical load $|K_E + \lambda K_G^*| = 0$. The lowest value of λ gives the critical load for the stayed column. This is a typical eigenvalue problem. The solution procedure for this method is direct.

Exact Solution. In this approach the elements of the stiffness matrix are written in terms of the stability functions, s, c, and m, where s is a stiffness, c a carryover, and m a sidesway stability function. The buckling load by this method is determined using an iterative solution.

Numerical Examples. The dimensions of the stayed columns used for the examples are shown in Figs. 1 and 2. The column and crossarm members are steel tubes, 2.25 in. outer diameter and 1.75 in. inside diameter. The stays are 1/4 in. diameter steel rods.

It has been assumed that at the instant of buckling there is a small residual pretension force left in the stays. This implies that all the stays are effective in resisting displacements of the stayed column.

The following are the ratios of the elastic buckling load of the stayed columns, shown in Figs. 1 and 2, to the Euler load of a 20 ft. section of pipe (which is 4.0 kips): single-crossarm 5.8, double-crossarm 12.1, triple-crossarm (type 1) 17.1, and triple-crossarm (type 2) 15.5.

Conclusions. 1) The elastic buckling loads of stayed columns may be predicted by the methods outlined. 2) The buckling loads as determined by the two approaches are in excellent agreement when a reasonable number of elements are used in the finite element approach. 3) The buckling load may be increased many times by using stayed columns. 4) The finite element approach is the preferred procedure since the solution procedure by this method is direct (as opposed to iterative).

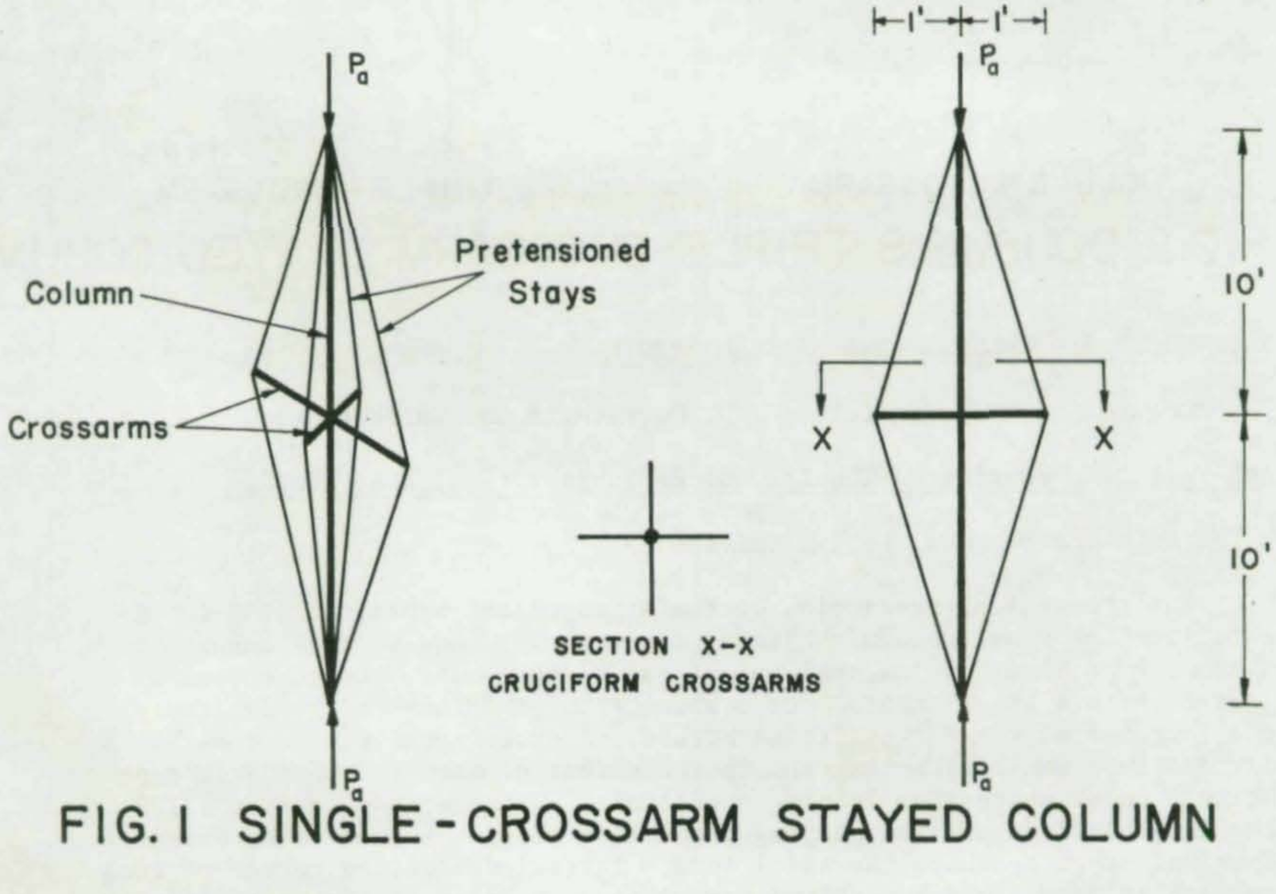

FIG. I SINGLE-CROSSARM STAYED COLUMN

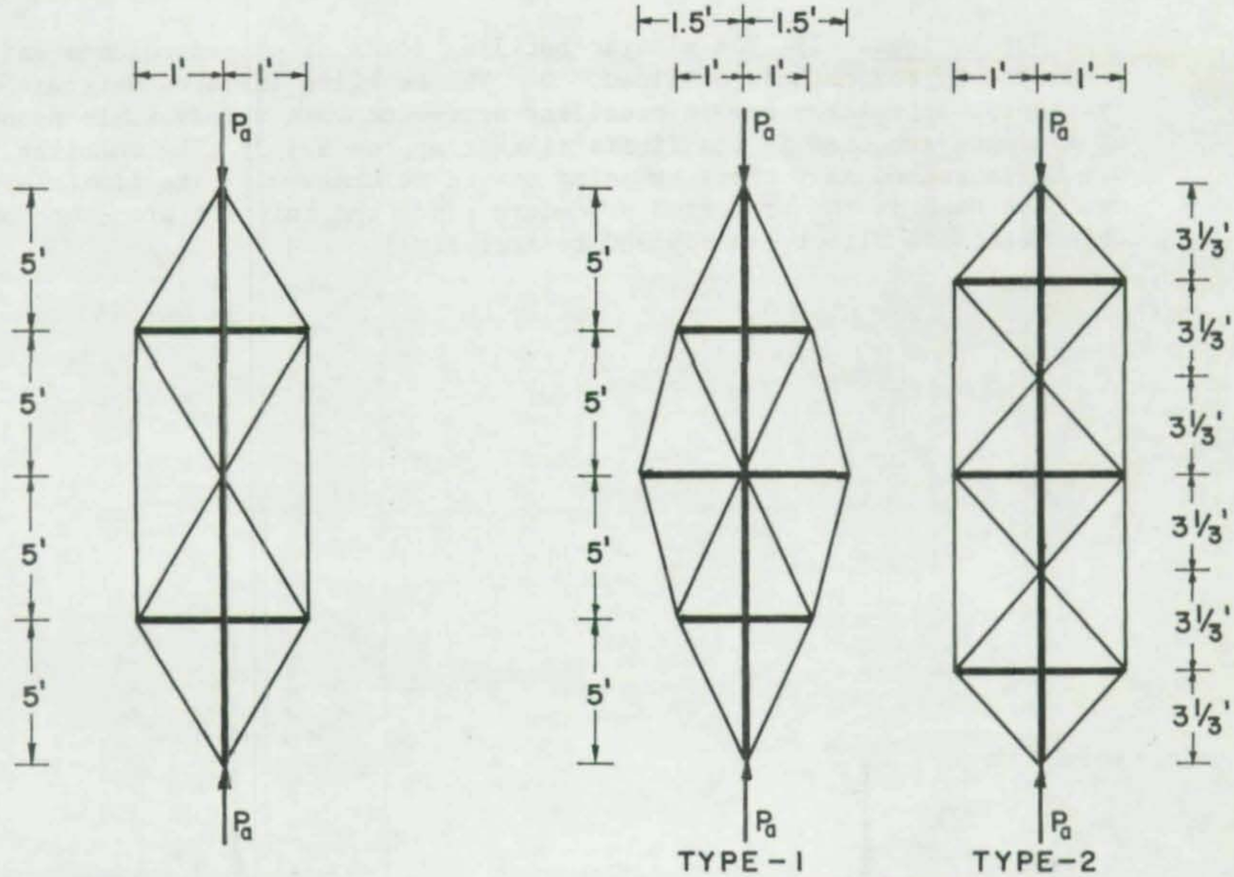

FIG. 2 DOUBLE & TRIPLE-CROSSARM STAYED COLUMN

TASK GROUP 3 - COLUMNS WITH BIAXIAL BENDING

Chairman, J. Springfield, C. D. Carruthers and Wallace, Ltd.

Analysis of a Fabricated Tubular Column Subject to Biaxial Bending

W. F. Chen and D. Ross, Lehigh University

The theoretical prediction of the strength and behaviour of a fabricated tubular steel column subjected to axial load and biaxial bending is a complex problem. Using residual stress measurements, the moment-axial load-curvature relationships for a short tubular column can be derived by the incremental tangent stiffness method. A cross section of a tube is divided into small elements, and the behaviour of each is considered separately under successive loading conditions. This method does not allow the length effect nor the varying properties along a column to be considered. Thus, a method by which the axial load - lateral deflection curves of long tubular columns could be derived was proposed to account for these effects.

TASK GROUP REPORTS

TASK GROUP 4 - FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

Chairman, J. S. B. Iffland, URS/Madigan-Praeger, Inc.

Non-Linear Large Deflection Analysis of Geometric Unstable Framed Structures

K. P. Chong, University of Wyoming and D. C. Kunkee, U. S. Steel Corporation

Geometric unstable structures represent a large class of structures that cannot resist acting loads in their initial configurations. However, under large deflections, such structures develop load-carrying capacities. A common example is a low pitch V-shaped roof frame subject to uplift or gravity loading. Usually, the deflection under loading is so large that it changes significantly the original geometry of the structure. Thus the small deflection theory is inadequate.

Using a damped oscillator approach, a computer program has been developed. The theory is based on joint equilibrium, force-displacement relationship and joint movements. The program can take care of the changes in geometry due to deflection, and iterate to arrive at the final equilibrium geometry and forces. Besides capable of analyzing geometric unstable structures, the program can also be applied to structures where large deflections occur. Some examples of the latter cases are illustrated using both small deflection theory (conventional method) and large deflection analysis furnished by the program. Another application of this method is to study the buckling and post-buckling behavior of a structure, accompanied by large deflections. A couple of geometric unstable structures, in which the conventional small deflection theory fails, are examplified. The following table shows the typical output for a three-hinge structure with a very low profile.

JOINT OUTPUT

JOINT	INITIAL		FREEDOM	FINAL		FORCES	
	X	Y		X	Y	FX	FY
1	10.000	10.000	1	10.000	10.000	-32764.	-2500.
2	34.000	10.000	1	34.000	10.000	32764.	-2500.
3	22.000	11.000	0	22.000	10.916	0.	0.

MEMBER OUTPUT

MEMBER	END JOINTS		INITIAL LENGTH	FINAL LENGTH	STRESS
1	1	3	12.042	12.035	-16429.7
2	3	2	12.042	12.035	-16429.7

NONLINEAR LARGE DEFLECTION ANALYSIS: -16429.7 psi

CONVENTIONAL SMALL DEFLECTION ANALYSIS: -15050.0 psi

TASK GROUP REPORTS

Frame Instability Considerations in Allowable Stress Design

L. W. Lu and F. Cheong-Siat-Moy, Lehigh University

Using the story stiffness concept (1), it was explained why certain unbraced, multistory steel frames can be proportioned by the allowable-stress design method neglecting the P-Δ effect. Yet, these frames will attain a load factor greater than 1.30 under proportionally increased combined loads. The parameters which define this class of inherently stiff frames were presented.

References

1. F. Cheong-Siat-Moy - "Multistory Frame Design Using Story Stiffness Concept", Journal of the Structural Division, ASCE, Proc. Paper 12221, June 1976.

Buckling of Space Frames

Z. Razzaq, Arizona State University

Results of a preliminary experimental model study of the behavior of unbraced orthogonal space frames have been presented. Two types of tests were conducted on frames made up of beams and columns of thin rectangular solid steel sections, statically loaded at column tops in a compression testing machine. The first frame having two columns only and with the strong axis of one column orthogonal to that of the other column carried a maximum load at complete buckling approximately equal to the sum of the buckling loads of the individual columns. The column with its weak axis in line with the weak axis of the connecting beam buckled first. Thereafter, an increase in the carrying capacity of the frame was observed until the second column also buckled. The bottom of the frame was fixed.

The second type of model frame with a single storey single bay had two of its columns oriented at right angles to the remaining two columns. The bottom ends were fixed and all connections welded. Upon buckling of two of the columns the frame reached its peak carrying capacity. This seems to be pointing out toward the significance of initial imperfections in the overall buckling behaviour of space frames.

The work is part of a more general experimental and theoretical investigation related to the study of elastic and inelastic behaviour of unbraced orthogonal metal space frames.

TASK GROUP REPORTS

TASK GROUP 7 - TAPERED MEMBERS

Chairman, A. Amirikian, Amirikian Engineering Company
(G. C. Lee, State University of New York at Buffalo presiding)

Design of Tapered Beam-Columns

M. L. Morrell, Clemson University and G. C. Lee, State University of New York at Buffalo

Since the adoption of tapered beam-column design formulas by AISC there has been two areas under further study: 1) the allowable axial stress, $F_{a\gamma}$, when weak-axis buckling governs, and 2) the beam-column coefficient, C_m, when joint translation occurs. This report presents an initial look at the weak-axis buckling question and a possible design curve for the C_m factor.

The theoretical basis for the present tapered design formulas assumes that the weak-axis axial buckling load is unaffected by the web taper. This is an acceptable assumption for elastic buckling. However, this is shown not to be true for inelastic buckling in Fig. 1. For values of slenderness less than C_c, buckling is inelastic. The curve for $\gamma = 0$, a prismatic column, has a lower critical stress than a column with $\gamma = 2$. The difference is due to the non-uniform yielding of the column along its length. The solutions on this figure for $\gamma = 0$, 2, and 4 were obtained using prismatic elasto-plastic beam elements. Also, plotted in Fig. 1 are the SSRC and AISC curves.

The second part of this report concerns the C_m factor for joint translation. The analysis is elastic and based on the slope-deflection method for tapered members. The frame sketched in Fig. 2 served as a model for the analysis. The complete study included the effect of the axial force ratio (P/P_E) in the columns and the restraint factors G_T and G_B (G = column stiffness/beam stiffness). Following the prismatic design assumptions for C_m, Fig. 2 shows the effect of the taper ratio on C_m. The current recommended value for tapered columns is $C_m = 0.85$ the same as prismatic, which is shown here to be conservative. With further study, Fig. 2 might be used in design.

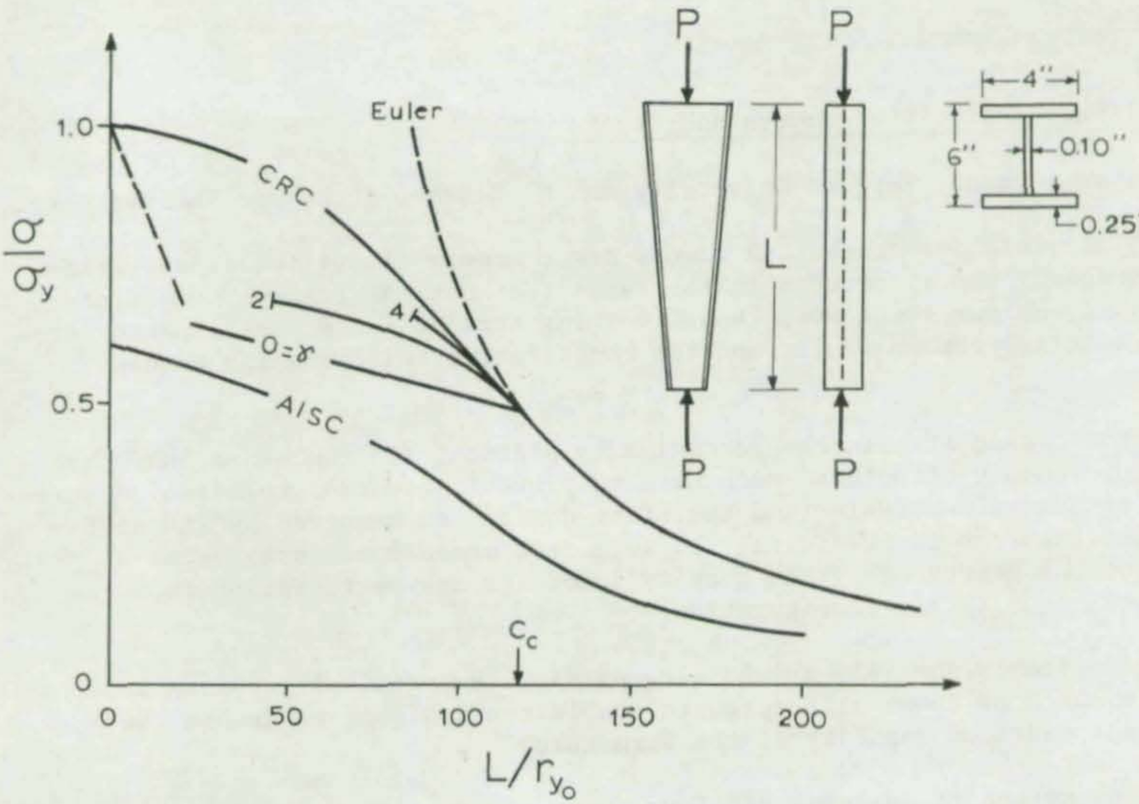

Figure 1: Weak-Axis Buckling for Tapered Column (Pinned Ends) with Welding Residual Stresses, $\sigma_{r_c} = 0.5\sigma_y$.

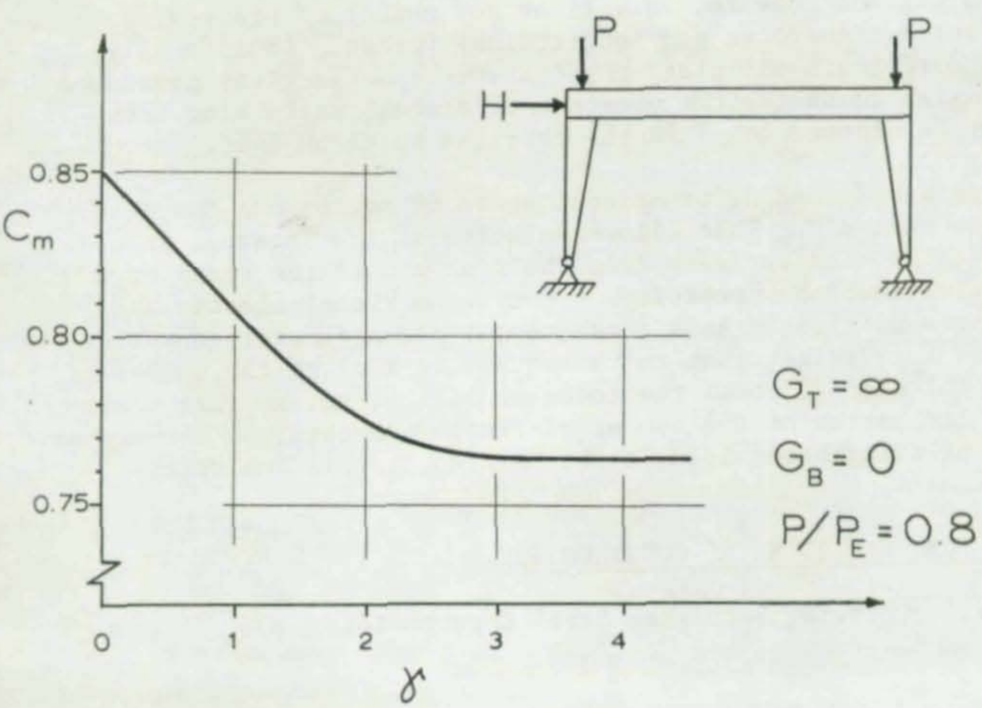

Figure 2: C_m Factor Curve for Tapered Columns (Rigid Beam and Pin Based Columns).

TASK GROUP REPORTS

Non-linear Analysis of Towers and Stacks

A. V. du Bouchet, Rutgers University and M. Biswas, Stone and Webster Corp.

Tall, slender stacks and towers are commonly classified as geometrically non-linear structures in the sense that maximum transverse deflections may become large even though working strains (under anticipated service loading) remain small, and the constitutive relationships remain linear.

While such structures are nominally statical (in the sense that there is no secondary structural mechanism to prevent potential collapse) a suitable structural analysis (and therefore design) is hampered by the difficulties inherent in predicting the augmented moments and transverse deflections generated by the gravity loads for any perturbation from the vertical equilibrium configuration.

A suitable analysis should also predict the critical buckling load of the stack or tower structure, to enable the designer to assess the inherent overload capacity of the structure.

The method of analysis developed uses a beam-column finite element in conjunction with transfer matrices and an incremental loading procedure to directly generate the total moments, and axial and transverse displacements of a stack or tower structure having a stepped or tapered cross-section. The structure may be subjected to any number of concentrated shears, axial forces and moments, as well as any number of piece-wise constant distributed transverse and longitudinal forces. Additionally, by use of a standard Southwell plot, the transverse deflections generated by compressive axial loads (in the presence of a small perturbing transverse force) may be plotted to yield the critical buckling load.

Finally, rigid-body and deformational modes of motion are completely separated in this analysis. This allows solution of the internal forces and deflections to proceed serially from the free end of the stack or tower structure, without the necessity of computing global elastic and geometric stiffness matrices. As a consequence, computer storage varies linearly (about 6 x n) rather than to the square (n x n) of the number of finite elements (n) used to model the tower configuration, so that there is no practical limitation on the number of finite elements, or the number of concentrated or distributed loads to be handled by this analysis.

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

Chairman, S. J. Errera, Bethlehem Steel Corporation

TASK GROUP REPORTS

Wrinkling Instability in Sandwich Columns with Cold-Formed Steel Facings

K. P. Chong and P. C. Liu, University of Wyoming

Sandwich panels with foam-in-place urethane cores and light-gage cold-formed metal faces are becoming more and more popular as building enclosures, due to their superior structural efficiency, insulation qualities, mass productivity, transportability, erectability, durability and reusability.

The purpose of this investigation is to study the axial load-bearing behavior of sandwich panels with foam-in-place cores and light-gage cold-formed metal facings. Heretofore the flexural behavior of such panels has been explored (1,2). The axial load-bearing capacities of panels with flat faces have been reported by Hoff and Mautner (3). However, as far as the authors know, experiments and theories on the axial bearing of the subject panels are non-existent.

Experimentally, full size panels measuring up to eight feet and forty inches wide were loaded axially with pinned end conditions. Strain gages were mounted at the mid-height of each panel, on both facings and at different profiles of the formed facing. Deflections were measured by a surveying transit.

At failure, most panels failed visibly by localized buckling in the shape of wrinkles across the middle portion of the panels. The wavelength of these wrinkles is of the same order as the thickness of the sandwich panels. Using a biharmonic equation in terms of a stress function, theoretical critical wrinkling stress can be predicted, as functions of facing and core properties.

REFERENCES

1. Chong K. P., and Hartsock, J. A. - "Flexural Wrinkling in Foam-Filled Sandwich Panels", Journal of the Engineering Mechanics Division, ASCE, Vol. II, No. EMI, Proc. Paper 10370, Feb. 1974, pp. 95-110.

2. Hartsock, J. A. and Chong, K. P. - "Deflections and Stresses in Sandwich Panels with Formed Faces", Journal of the Structural Division, ASCE, Vol. 102, No. ST4, Proc. Paper 12058, Apr. 1976, pp. 803-819.

3. Hoff, N. J. and Mautner, S. E. - "The Buckling of Sandwich-Type Panels", Journal of the Aeronautical Sciences, Jul. 1945, pp. 285-297.

AISC Buckling Formulas as Constraints in Optimal Design of Stiffened Girders

R. T. Douty, University of Missouri-Columbia

Difficulties in solving the nonlinear programming problem of generating the optimal design of detailed structural systems were discussed where those difficulties are due to using AISC buckling formulas in the set of constraints which govern the design. It was shown that allowable stress formulas, which can take alternate forms, depending on geometric and material properties of the structure, in effect, act as discontinuities distracting the iterative computational process from converging to a solution. A technique was demonstrated where the designer, interacting with the iterative progress of the solution in a timesharing environment, is able to use his intuitive talents as a designer and opportunity as an outside observer to the computational process to nudge that process in favorable directions and towards a conclusion.

Several examples were given to suggest a strategy for optimizing the cooperative capabilities of both man and computer for decision making in the design process.

The typical example involving the design of a welded stiffened plate girder demonstrates how the technique of interacting with the solution process to the nonlinear programming problem, as it is occurring, can be used to evolve its detailed design in an expeditious manner. In this case designer interaction every five cycles (shown underlined) is used to help the mathematical process of nonlinear programming converge to a conclusion. The tabular headings in Table 1 are for the flange width, flange thickness, web depth, web thickness, flange and web yield strengths, and the numbers of stiffened panels between the left reaction and left concentrated load, between the two concentrated loads, and between the right concentrated load and the right reaction, Fig. 1.

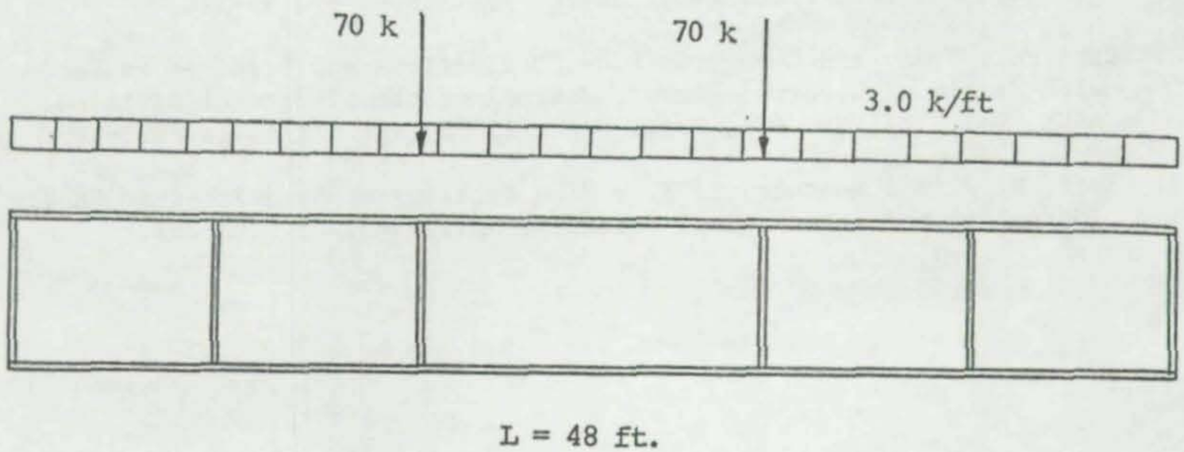

FIGURE 1 WELDED STIFFENED GIRDER

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  BF      TF      DW       TW      FYF     FYW     NL   N12    NR
 6.149   1.000   70.000   0.188   36.00   36.00   1.00 1.00   1.00
11.014   1.000   60.381   0.188   36.00   36.00   1.93 1.88   1.93 ( 92.5260)
12.507   1.000   70.000   0.217   36.00   36.00   2.83 2.63   2.83 ( 46.9232)
10.827   1.000   70.000   0.392   36.00   36.00   1.00 1.00   1.00 ( 80.1144)
14.460   1.000   70.000   0.188   36.00   36.00   2.03 1.00   2.03 (103.0620)
ENTER CHANGES OR 'GO'-
USE
TW .3125
GO
15.856   1.000   60.381   0.313   36.00   36.00   3.47 1.88   3.47 ( 87.9799)
13.275   1.000   63.625   0.313   36.00   36.00   1.00 1.00   1.00 ( 71.1437)
11.927   1.000   70.000   0.313   36.00   36.00   1.49 1.00   1.49 ( 49.3116)
12.097   1.000   70.000   0.313   36.00   36.00   1.96 1.00   1.96 ( 30.9741)
12.099   1.000   70.000   0.313   36.00   36.00   2.14 1.00   2.14 (  9.5939)
ENTER CHANGES OR 'GO'-
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BF 14
GO
14.000   1.000   70.000   0.313   36.00   36.00   2.16 1.00   2.16 ( 15.7167)
14.000   1.000   60.128   0.313   36.00   36.00   1.86 1.00   1.86 ( 14.1031)
14.000   1.000   61.741   0.313   36.00   36.00   1.91 1.00   1.91 (  2.6832)
14.000   1.000   61.800   0.313   36.00   36.00   1.91 1.00   1.91 (  0.0958)
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14.000   1.000   61.800   0.313   36.00   36.00   2.00 1.00   2.00 (  0.0    )
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TABLE 2 Progress of Solution for Welded Stiffened Girder

TASK GROUP 15 - LATERALLY UNSUPPORTED BEAMS

Chairman, T. V. Galambos, Washington University

Design Rules for the Lateral Buckling of Steel Beams

D. A. Nethercot, University of Sheffield and N. S. Trahair, University of Sydney

Current methods of designing unbraced single span steel I-beams that fail by lateral buckling are normally based on the elastic buckling and full plastic moments. This is not entirely satisfactory, especially when applied to beams of intermediate slenderness which fail by inelastic buckling.

An improved design method is therefore suggested (1,2) which makes direct use of the beam's buckling moment. For slender beams, the design is based largely upon the elastic buckling moment, while the full plastic moment is used for stocky beams. For beams of intermediate slenderness, a simple expression is used which provides good estimates of the inelastic buckling moments of single span steel I-beams under a variety of different loading conditions. This approach is more consistent than that used in some present methods, as recent studies have shown that inelastic buckling cannot be predicted from the elastic buckling and fully plastic moments alone.

For stocky beams, the full plastic moment is taken as the design ultimate moment capacity, but for more slender beams, the inelastic and elastic buckling moments are reduced to allow for the decreases in ultimate moment capacity caused by imperfections. Simple expressions are used to relate the ultimate moment capacity to the buckling moments and the full plastic moment. Two alternative sets of expressions are established by using available test data. The resulting design curves are illustrated in Fig. 1.

A design chart, which permits the direct determination of the maximum brace spacing for a given moment and shear, simplifies the method's application in practice (2).

REFERENCES

1. Nethercot, D. A. and Trahair, N. S. - "Inelastic Lateral Buckling of Determinate Beams", Journal of the Structural Division, ASCE, Vol. 102, No. ST4, (in press)

2. Nethercot, D. A. and Trahair, N. S. - "Design Rules for the Lateral Buckling of Steel Beams", The Institution of Engineers, Australia, Metal Structures Conference on Codes, Regulations and the Practising Engineer, Adelaide, Nov. 25-26, 1976, Preprints of Papers (in press)

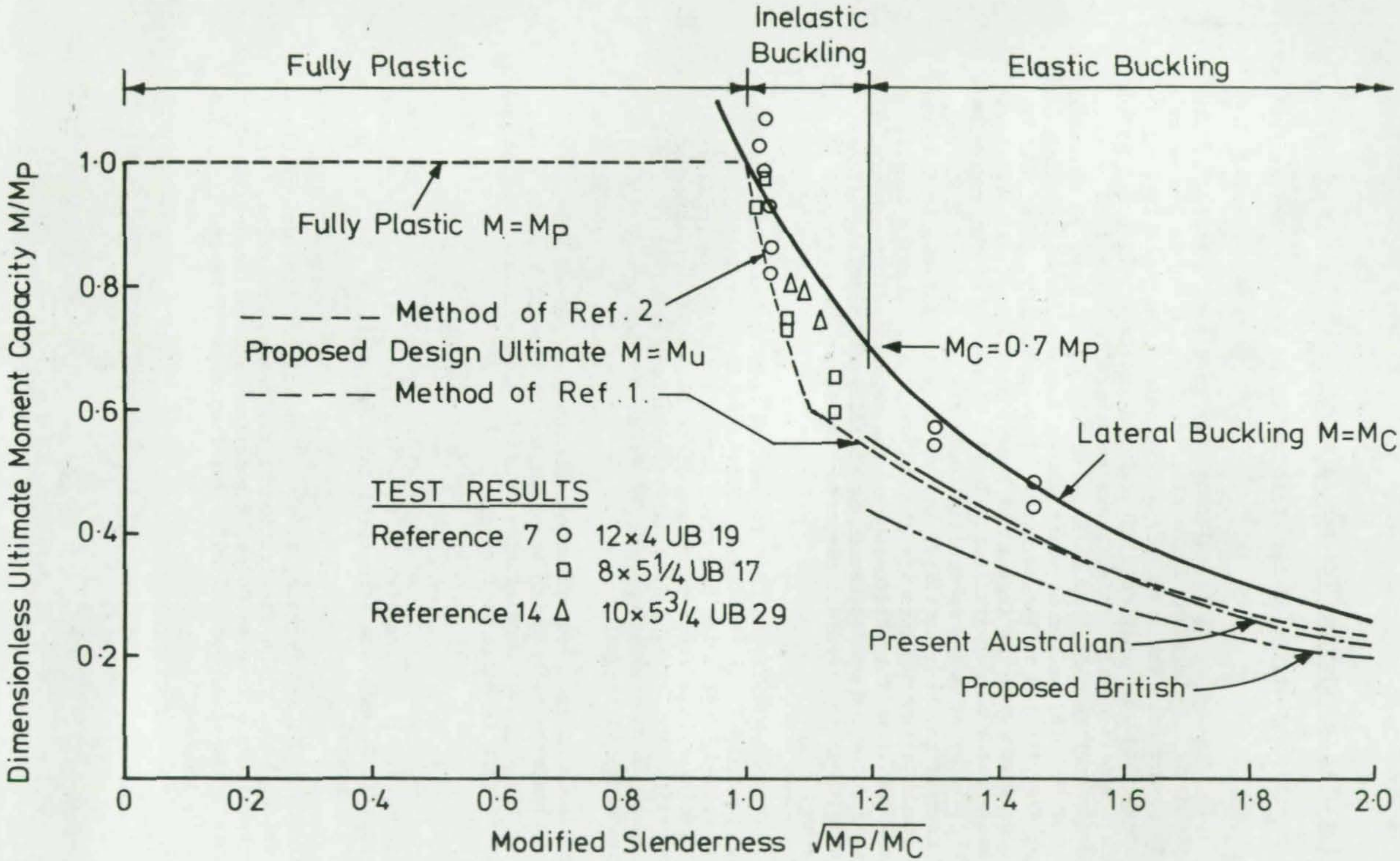


FIG.1. PROPOSED DESIGN BASIS FOR ULTIMATE MOMENT CAPACITY

TASK GROUP REPORTS

Lateral Buckling Calculations for Braced Beams

D. A. Nethercot, University of Sheffield and N. S. Trahair, University of Sydney

A simple hand method of calculating very good estimates of the elastic (1) and inelastic (2) buckling loads of laterally continuous braced steel I-beams is developed. The method is an extension of a commonly used approximate method which neglects lateral continuity at the brace points between adjacent segments. This allows a lateral buckling load to be estimated for each segment, and this is done by calculating an equivalent uniform elastic buckling moment which allows for the actual in-plane bending moment distribution in the segment. Where necessary, this is modified in accordance with the results of recent studies of inelastic buckling of beams under moment gradient. In the extension of the approximate method, the different segment buckling loads are compared so that the critical segment can be identified, and the buckling interactions between this and its adjacent segments are then allowed for. This is done by calculating the effective stiffnesses of the segments after making due allowances for the reductions caused by yielding and buckling effects, and by using these to calculate the effective length of the critical segment.

The method is simple and easy to use and requires no other computational aid than the freely available chart used to determine the effective lengths of braced compression members. Simple but accurate formulae are used to calculate the effects of moment distribution and yielding on lateral buckling, while the additional calculations required to allow for the brace point interactions are few because these are only carried out for the critical segment.

Comparisons of the predictions of the proposed method and of the existing approximate method with accurate finite element results show that the proposed method consistently leads to more accurate predictions. An example of this is shown in Fig. 1.

REFERENCES

1. Nethercot, D. A. and Trahair, N. S. - "Lateral Buckling Approximations for Elastic Beams", The Structural Engineer, Vol. 54, No. 6, Jun. 1976 (in press)

2. Nethercot, D. A. and Trahair, N. S. - "Lateral Buckling Calculations for Braced Beams", The Institution of Engineers, Australia, Metal Structures Conference on Codes, Regulations and the Practising Engineer, Adelaide, Nov. 25-26, 1976, Preprints of Papers (in press)

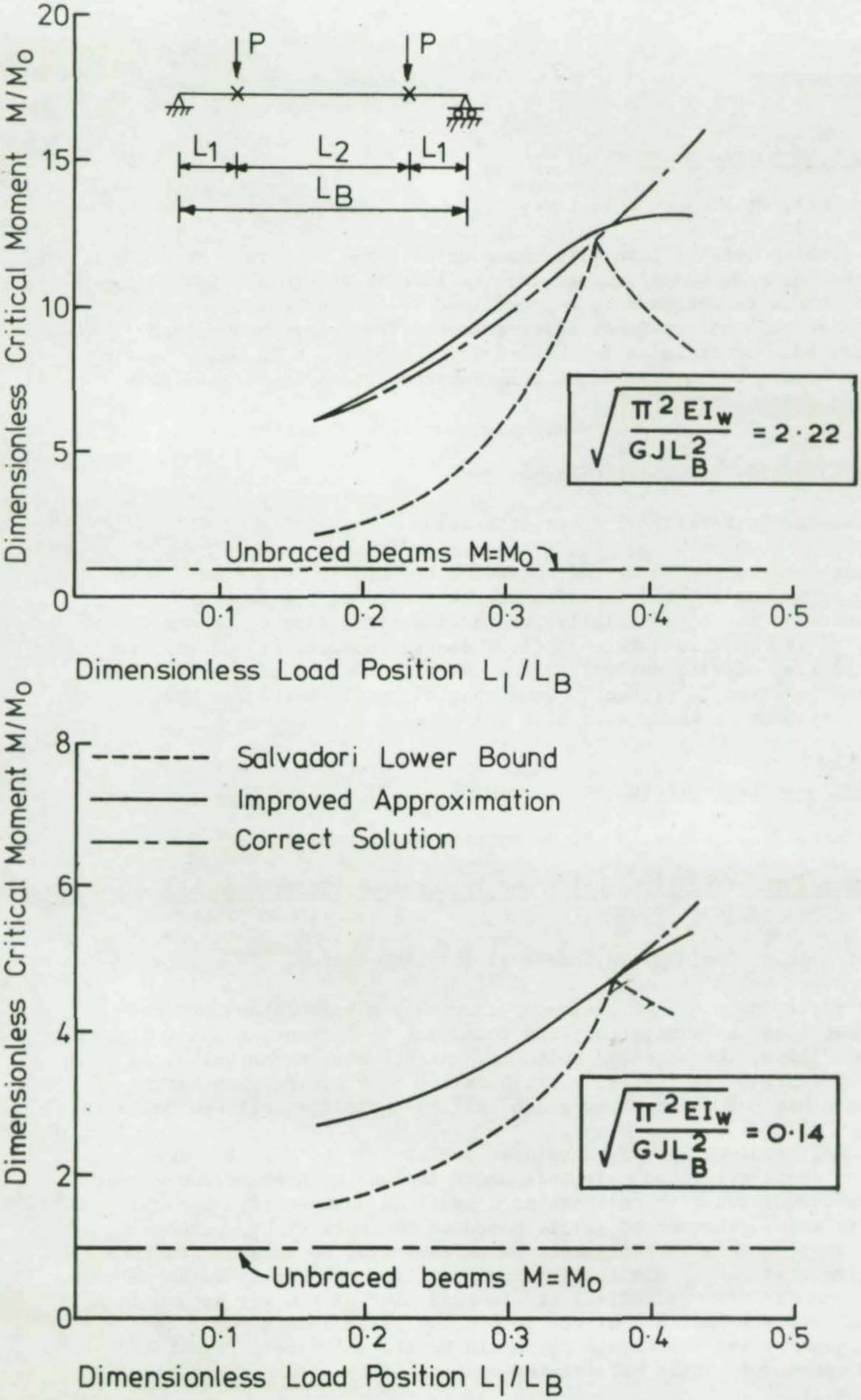


FIG.1. BUCKLING OF THREE SEGMENT I-BEAMS WITH SYMMETRICAL LOADS

TASK GROUP REPORTS

Laterally Unsupported Beams

T. V. Galambos, Washington University

The problem area of laterally unsupported beams concerns the study of unbraced beams or beam-segments when failure is due to the inability of the structure to continue to support load because of the uncontrolled growth of lateral and torsional deformations. The report summarized recent work not yet included in the 3rd Edition of the SSRC Guide, paying special attention to the review of the research in non-linear analysis, testing and design criteria.

Design of Laterally Unsupported Beams

J. A. Yura, The University of Texas at Austin

Recent work in the U. S. and abroad on the problem of laterally unsupported beams has greatly expanded our knowledge of the beam problem in the inelastic range, especially for the important case of moment gradient. Based on this work, practical design recommendations will be made that differ significantly from the current SSRC and AISC approaches when moment gradient is present. Some special requirements for cantilever and overhanging beams were also presented.

TASK GROUP 16 - PLATE GIRDERS

Chairman, F. D. Sears, U. S. Department of Transportation

Ultimate Strength of Longitudinally and Transversely Stiffened Ship Bulkheads Under Inplane Loads

M. O. Critchfield, David W. Taylor Naval Ship R&D Center

This paper describes an ultimate strength analysis capability for an important load carrying structural component in conventional and high performance ships, the strength bulkhead. At present, these bulkheads are usually designed to initial plate buckling or yielding with little knowledge in hand of the real margin of safety against a bulkhead collapse.

The analytical method was developed for evaluating the ultimate strength of cross-stiffened bulkheads under inplane high shear/low moment conditions associated with collapse in a panel as opposed to a general instability mode. The method (1) is based on the work of Ostapenko and Chern (2) and essentially represents an extension of their analysis from a plate girder with a single longitudinal stiffener to a ship bulkhead having any number of longitudinal stiffeners. Two of the key assumptions in the analysis are that all of the subpanels (plates) in a panel have the same geometry and that these subpanels buckle and develop tension fields independently while maintaining compatibility of shear strain.

TASK GROUP REPORTS

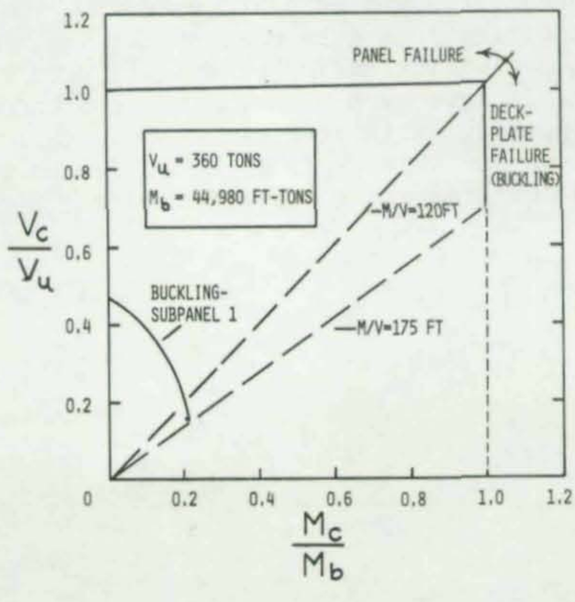
The analysis involves a calculation of shear, bending moment and a number of other related variables associated with the progressive buckling of the subpanels in a bulkhead panel, the tension field development following buckling in each subpanel and finally the simultaneous tension field yielding of all subpanels. The effect on panel strength of buckling of the first longitudinal stiffener, the most severely stressed one, can be taken into account. The combined shear V_c and bending strength M_c of a bulkhead panel, together with an effective portion in bending of adjoining deck structures, is displayed on the usual interaction type of diagram (see figure). A computer program has been written which implements the entire analysis and generates the strength interaction diagram coordinates for any range of M/V conditions specified by the program user.

The numerical interaction diagram results in the figure below were generated for a typical bulkhead panel-deck system taken from a preliminary design of a U. S. Navy 4000 ton small waterplane twin hulled ship (SWATH) for a specified range of M/V conditions. For this particular example, the bulkhead panel ultimate strength was found to vary from two to five times the initial plate (subpanel 1) buckling strength. Experimental confirmation of the analytical approach for ship bulkheads is still needed.

REFERENCES

1. Critchfield, M. O. - "An Ultimate Strength Analysis Technique for Cross-Stiffened Bulkheads Under Combined Inplane Bending and Shear", Naval Ship Research and Development Center Report 4611, May 1975.

2. Ostapenko, A. and Chern, C. - "Strength of Longitudinally Stiffened Plate Girders Under Combined Loads", Fritz Engineering Laboratory Report 328.10, Lehigh University, Pa., Dec. 1970.



INTERACTION DIAGRAM FOR BULKHEAD PANEL - DECK STRENGTH

A Review of the So-Called Incomplete Diagonal Tension Engineering Theories for Steel Plate Girder Ultimate-Strength Design Under Pure Shear

M. Algaaly, Bechtel Power Corporation

In 1928 Wagner demonstrated that a thin shear web with transverse stiffeners does not fail when it buckles between the stiffeners. It merely forms diagonal folds and functions as a series of tension diagonals while the stiffeners act as compression posts. By neglecting the shear carrying capacity according to the classical beam theory and considering the web as a membrane resistant only to tension, Wagner formulated the theory of pure diagonal tension. The stress state described by the theory of pure diagonal tension is a theoretical limiting case and it is possible to approach this case fairly closely by making the web very thin.

Near failure, plate girder webs will be in a state of stress which is intermediate between the state of pure diagonal tension and the state of stress that exists before buckling which is described by the elementary beam theory. In 1960, Basler and his colleagues at Lehigh University presented an ultimate load method for the design of shear webs. The method was adopted in the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings in 1961, and later on in the AASHTO standard specifications for highway bridges. Theoretical studies by Leggett and Hopkins and by Bergman of infinitely long shear webs; experimental work by Rockey and subsequent studies by Djubek and by Rockey and Martin on shear webs of aspect ratios 2 & 3, showed that the post-buckling behaviour is influenced by the flexural rigidity of the flanges.

Basler's method does not take into consideration the effect of the flange rigidity and many authors have introduced alternative methods to find the ultimate shear strength of plate girder webs taking into consideration the effect of flange rigidity. Among those are Fujii, Rockey and Skaloud, Ostapenko, Chern and Parsanejad, Komatsu and finally Rockey, Porter and Evans. In all these methods semi-empirical formulae were given to predict the ultimate shear strength of plate girders. To derive these formulas considerable simplifications were made at the expense of locally violating equilibrium, compatibility and/or plasticity conditions. However in every case reasonable agreement between the predicted values and test results have been reported. With several methods available, the question is which one to use and this paper is an effort to provide an answer. The methods were compared and a unified approach was suggested by the author.

TASK GROUP REPORTS

TASK GROUP 18 - UNSTIFFENED TUBULAR MEMBERS

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

Post Buckling Behavior of Tubular Truss Elements

P. W. Marshall, Shell Oil Company

Realistic evaluation of the response of tubular structures to extreme earthquakes requires understanding of the inelastic behavior of its constituent members. Previous work on inelastic bending, reported at the 1974 CRC meeting, relates to the portal action of structural columns and laterally loaded piles. For tubular cross bracing, the preliminary model shown in Fig. 1 has been developed to describe the behavior of struts under cyclic axial overloads. These elements have been combined in a nonlinear dynamic analysis, using time step integration of absolute displacement, to study the inelastic behavior of several proposed offshore structures. Additional work is underway to improve both the member models and the methods of dynamic structural analysis.

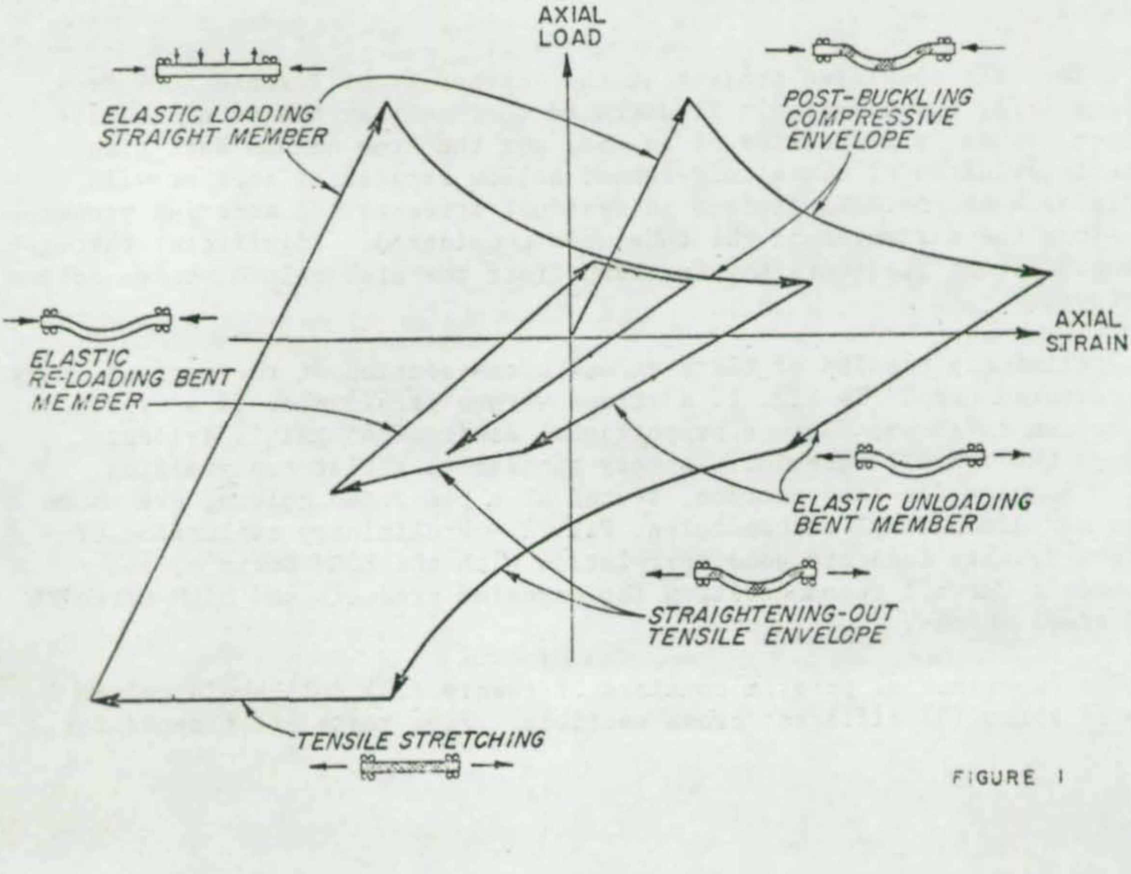


FIGURE 1

Experimental Testing of Fabricated Tubular Columns

W. F. Chen and D. A. Ross, Lehigh University

This presentation summarized the results of an experimental investigation into the axial strength of long, steel, fabricated tubular columns. Such columns are commonly used in offshore structures. Included in the report were results of residual stresses in a typical column, the results of stub column tests, and the results of the buckling behaviour of ten long columns with ℓ/r_y ratios between 39 and 83. Residual stresses in at least two perpendicular directions are induced in a column during manufacture, and the experimental results obtained were compared with available theoretical predictions. The stub column tests allowed derivation of column buckling curve.

Compressive Strength of Hollow Structural Sections

P. C. Birkemoe, University of Toronto

The concept of multiple column curves has been accepted in Europe and North America as an improved method of column strength assessment. Over the past years, research on numerous columns of a wide variety in manufacture has produced sufficient data to suggest that certain types of columns be treated differently in design than others because of their inherent manufacturing tolerances and residual stress distributions. The objective of the present study is to evaluate the column behaviour of hot finished class H, Hollow Structural Shapes covered by the CSA G40.20 standard.

A recently completed project at the University of Toronto (CRC Proceedings 1975) quantitatively illustrated that neither the usual tensile specimen (taken from the side of an HSS) nor the stub column data give realistic evidence of how a cold-formed hollow structural section will perform as a column. Variations in residual stresses and material properties along the perimeter of the tube were considered. Significant through-thickness stress gradients may further affect the stub column versus column behaviour.

Preliminary results of tests on one cross-section of the current study are presented here. In Fig. 1, a stress versus strain plot of a typical stub column is shown. Note a proportional limit of 45 ksi is evident although the overall behaviour is very similar to a flat top yielding steel. Results for same section, tested as a pin-ended column, are shown in the non-dimensional column curve, Fig. 2. Preliminary evaluation of the test results indicate good correlation with the ECCM Curve a, and Bjorhovde's Curve 1 recommendation for annealed products and high strength Q & T steel shapes.

The experimental program consists of twenty (20) full scale column tests of three (3) different cross sections. Five tests are planned for each set of parameters which include two wall thicknesses and two slenderness ratios. For each cross section three stub column tests were performed for a total of nine (9). Various supportive material tests and measurements include initial straightness measurements, tensile properties tests, and residual stress evaluation.

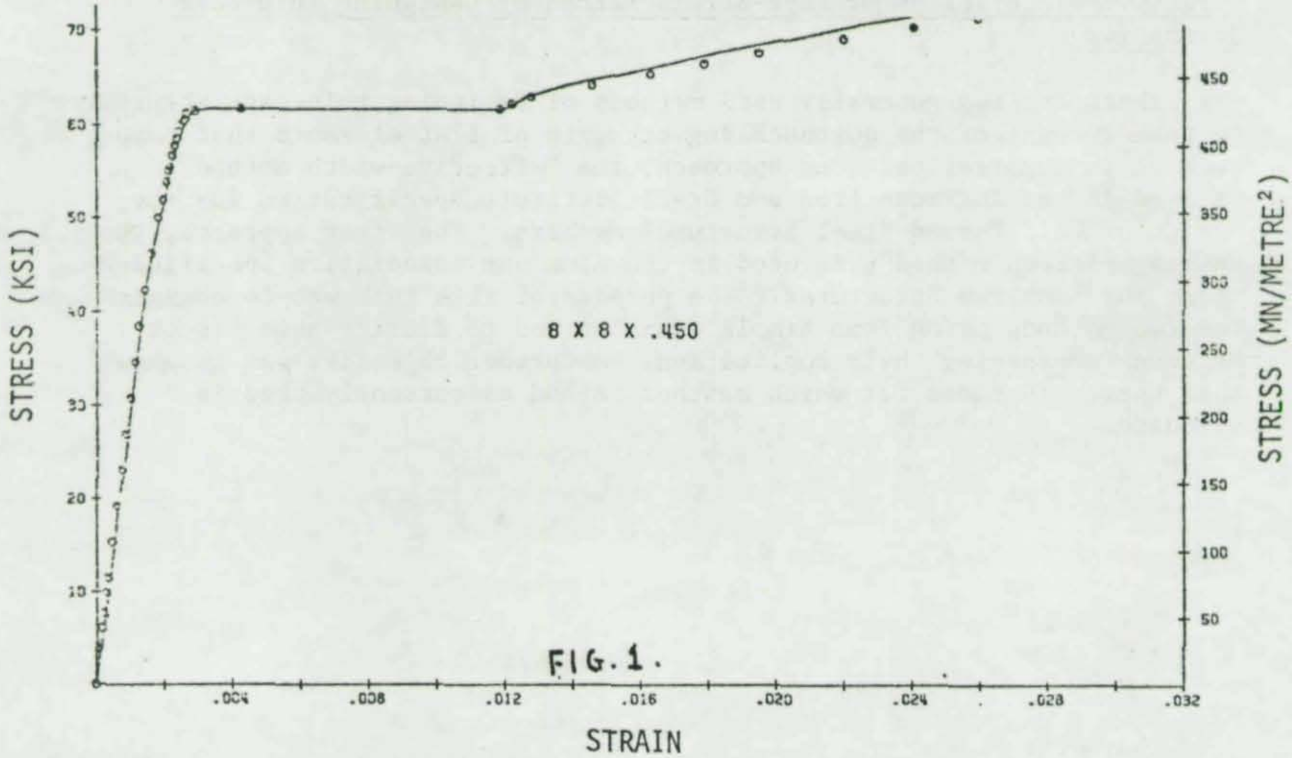


FIG. 1.

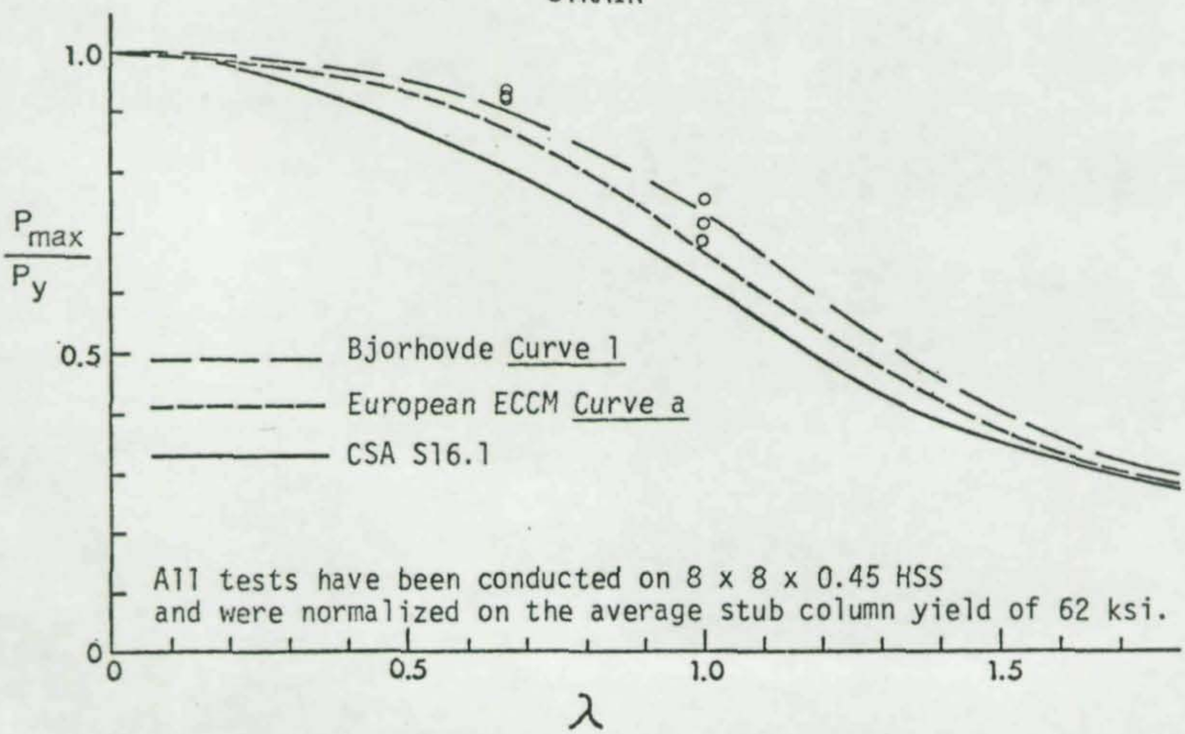


FIG. 2: COMPARISON OF COLUMN TEST DATA AND VARIOUS COLUMN DESIGN CURVES

CONTRIBUTIONS OF TASK REPORTERS

TASK REPORTER 11 - STABILITY OF ALUMINUM STRUCTURAL MEMBERS

J. W. Clark, Aluminum Company of America

Effective-Width Versus Average-Stress Method of Designing Thin-Gage Structures

There are two generally used methods of designing thin-gage structures to take account of the postbuckling strength of flat elements that have buckled in compression. One approach, the "effective-width method", is used in the American Iron and Steel Institute Specification for the Design of Cold Formed Steel Structural Members. The other approach, the "average-stress method", is used in the Aluminum Association Specifications for Aluminum Structures. The purpose of this talk was to compare the two methods using some simple examples and to clarify some misconceptions concerning their application. A further objective was to show that there are cases for which neither method as currently used is adequate.

S S R C   G U I D E   R E P O R T

COMMITTEE ON THE SSRC GUIDE (3RD EDITION)

Chairman, E. H. Gaylord, University of Illinois

Progress Report on the 3rd Edition

B. G. Johnston (Editor), University of Arizona

The 3rd Edition of the Guide was published in April 1976 and is available from J. Wiley & Sons, Inc., 605 Third Avenue, New York, New York 10016. As of 30 September 1976, 850 copies had been sold.

An errata list was prepared for distribution in the fall as well as to provide a basis for second printing corrections.

A talk on the 3rd Edition was presented at the AISC Engineering Conference in Atlanta in May and a paper entitled "Design Applications: The SSRC Guide, Third Edition" opened the SSRC-ASCE half-day session in Philadelphia in late September. In this paper, preprinted for the convention, it is shown how a combination of the SSRC multiple column strength curves, together with an accurate determination of effective length, provides a means of reducing uncertainties in column strength calculations. Procedures are illustrated by several simple examples.

J.S.B. Iffland, J. Springfield, G. Winter

P A N E L   D I S C U S S I O N

NEW IDEAS IN STABILITY

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

PANELISTS: W. J. Le Messurier, Le Messurier & Associates/SCI
J. Springfield, C. D. Carruthers & Wallace, Ltd.
D. I. Ruby, John Portman & Associates

FOREWARD

Traditionally, the Council has used the annual Panel Discussion as a medium for providing an interface between practicing engineers and those in the research field. Many of the ideas expressed are controversial and conflicting with each other. This is one of the objectives of this discussion - to get these thoughts out into the open so that those working in the academic area can direct their efforts toward solution of the current problems.

The Panel Discussion is divided into a formal presentation by the panelists and an informal question and answer period between the audience and the panel. Some questions will be answered unsatisfactorily or not at all. Again, this is a session objective - to formulate the unknowns that require research before answers can be given.

The following documents that portion of the formal discussion by those panelists who prepared advance notes for distribution.

SYNOPSIS OF THE SSRC GUIDE CHAPTER 15 MULTISTORY FRAMES (J. S. B. Iffland)

Within the context of the subject, "New Ideas in Stability", the stability of a structure can be defined as its capacity to recover from displacements induced by an applied force or disturbance. This concept, illustrated in Fig. 1, depicts two types of frames, one joint loaded and the other with a realistic loading pattern.

The joint loaded frame is shown for reference since it forms the basis of the effective length design procedures. It is the frame equivalent of the Euler column and the load displacement curve shown can only exist if the frame is without geometric and material imperfections. The critical load on such a joint loaded frame is referred to as the bifurcation load and from this can be determined the effective lengths of the columns.

The second frame represents a real structure. As the load on it is increased, the members are stressed elastically until parts of the frame are strained into the inelastic region. The ultimate capacity is reached when the combination of progressive yielding, axial force and joint displacements reduce the stiffness of the structure so that the frame becomes unstable. All tests show conclusively that unbraced frames are likely to fail through instability before the formation of a plastic mechanism and that any rational analysis and design procedure should attempt to include this effect.

A mathematical model may be constructed to duplicate test results as indicated by the load displacement curve for the real frame in Fig. 1. Of necessity, the model must consider second order effects. Chapter 15 of the Guide lists some 17 possible factors that could be considered. Two of these outweigh all of the others by far, viz: (1) the effect of joint displacements on bending moments and forces, and (2) the influence of axial force on member stiffness. Of these two prime factors, the first, the effect of joint displacement, is the most important.

Fig. 2 depicts a frame subjected to both horizontal and vertical load. For illustration, these loads are applied separately. The horizontal force induces an initial horizontal deflection. The vertical load is carried through this initial displacement with the resulting P-delta moment creating additional horizontal deflection which in turn increases the P-delta moment. The horizontal deflection increases progressively to instability of the structure unless the structure has sufficient internal stiffness to stabilize the movement at some amplified initial deflection. If this stiffness exists, the structure is stable. From this description of the failure process, it is seen that formulation of the equilibrium equations on the deformed structure is a primary consideration in assessing a structure's stability.

Since all unbraced frames fail by instability, the problem is how to consider stability requirements in design. Chapter 15 outlines four methods which are: (1) bifurcation analysis, (2) second order analysis, (3) working stress P-delta method, and (4) drift control.

Since the advent of the 1963 AISC Specification, which introduced the effective length procedure as a code provision, engineers have been faced with the difficulty of determining effective length factors. Accordingly, Chapter 15 has devoted considerable space to this problem. The starting point is the AISC Specification Nomograph given in the Commentary. Chapter 15 than discusses ways to modify the Nomograph for 5 different non-Nomograph special conditions.

The second method of stability design treated in Chapter 15 is a second order analysis. This will not be discussed here since it is not a practical office design method.

The third method of stability design treated in Chapter 15 is the working stress P-delta procedure. This procedure is based on the fact that the P-delta effect is the primary consideration in stability. If in any ultimate strength determination of a frame, working loads are determined by dividing ultimate loads by a load factor F, it is consistent to determine the lateral force due to the P-delta effect by dividing the lateral force computed at ultimate load by the same load factor. This is illustrated below:

Working load story shear on a column

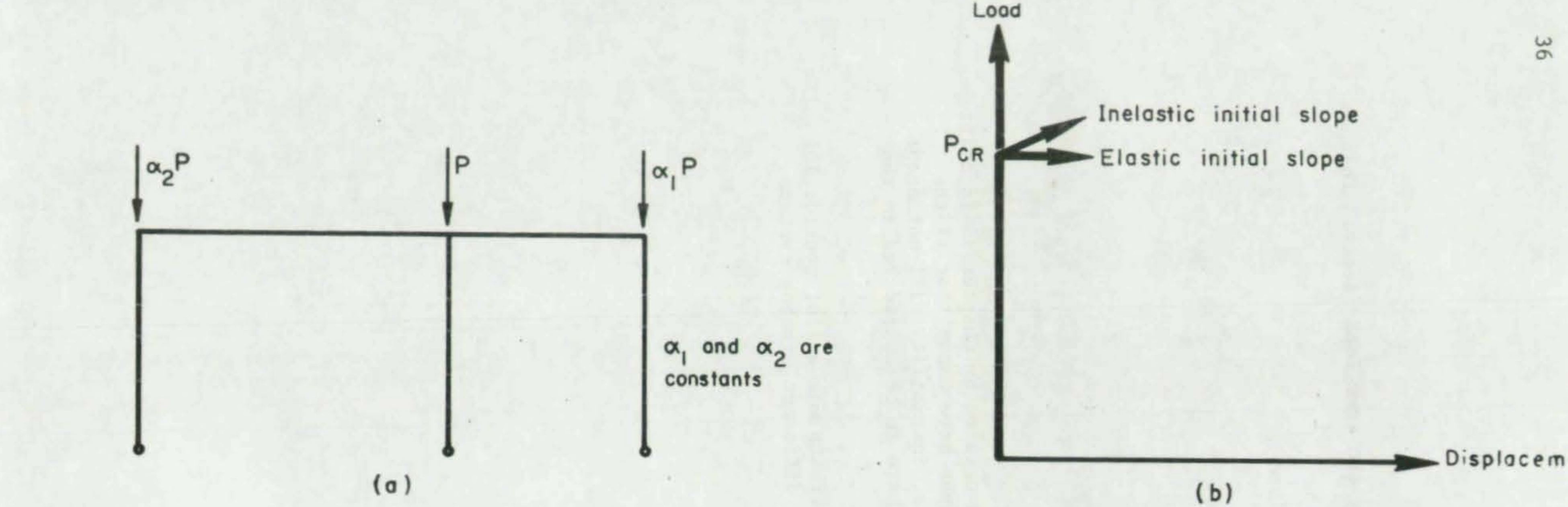
$$V' = \frac{FP\Delta Ult.}{h} \cdot \frac{1}{F} = \frac{P\Delta Ult.}{h}$$

Assume $\Delta Ult. = \overline{F}\Delta$

And then $V' = \overline{F}\ \frac{P\Delta}{h}$

The fourth method of stability design covered by Chapter 15 is drift control. Studies of specific types of frames have shown that if the drift is controlled within specified limits, instability will not occur and for these types of frames it does not have to be considered in the design procedure.

As a final consideration, Chapter 15 treats methods of considering stability for buildings of combinations of different bracing systems.



Load - Displacement curve for ideal joint-loaded frame

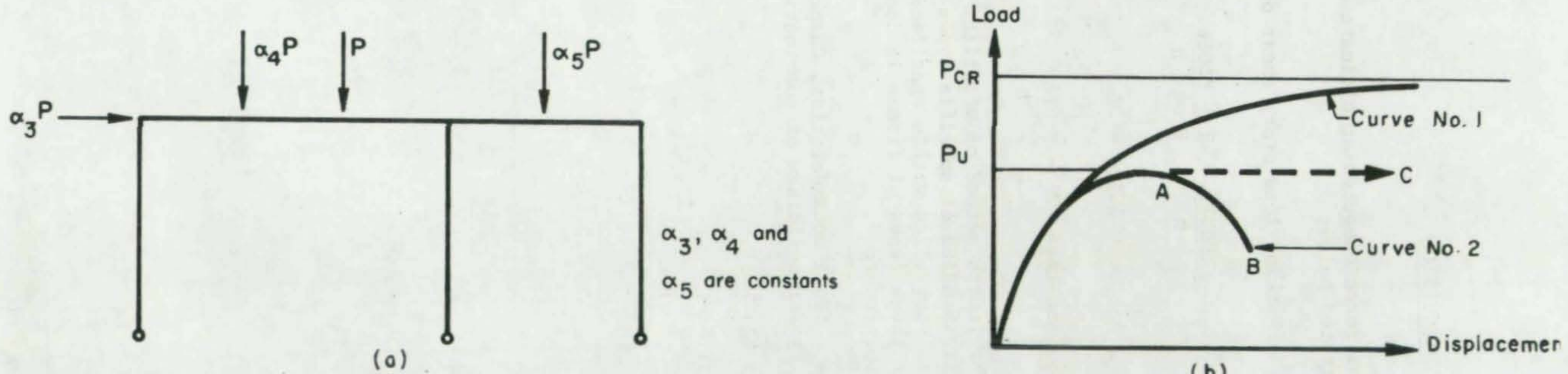
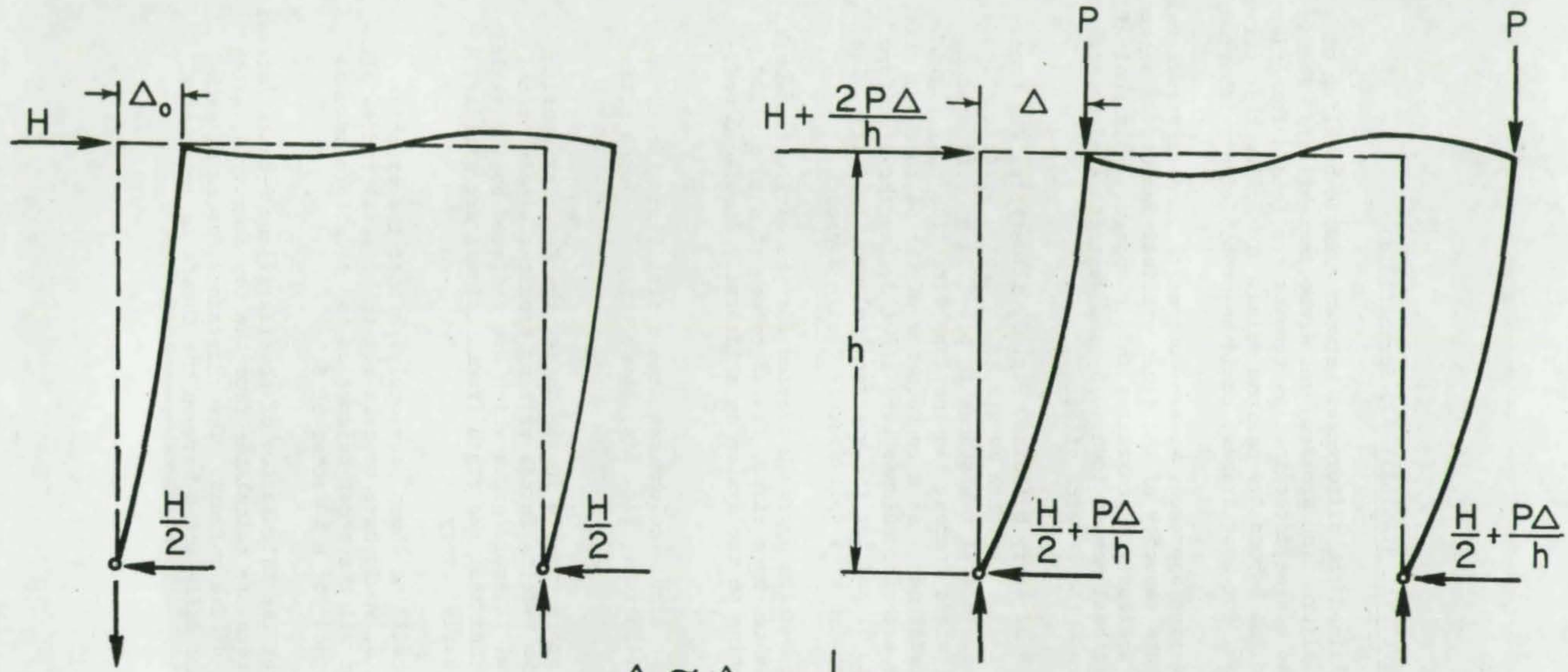


Fig. 1 – Load - Displacement curves for frames not loaded at the joints



$$\Delta \approx \Delta_o \frac{1}{1 - \frac{P}{P_e}}$$

Fig. 2 – EFFECT OF JOINT DISPLACEMENTS ON BENDING MOMENTS AND FORCES

PANEL DISCUSSION

NEW APPROACHES IN STRUCTURAL STABILITY (J. Springfield)

Iffland has explained the differences between elastic buckling of a frame, that is the bifurcation problem, and frame instability. The inelastic stability load can be determined by tracing the load/deformation path of a frame up to and beyond its maximum value. To follow this curve accurately to its peak, for most frames, could be prohibitively laborious.

In our current design approach, we are content to predict structural response to the ultimate capacity of the first critical member. A second order elastic analysis closely approximates this response. This analysis accounts for the additional moment induced by the vertical load acting in the deformed column - the P-delta effect.

The framing for a 57 story building, Fig. 1, illustrates the breakdown of the simple K-factor approach as stated in design specifications. In the short direction, lateral resistance is provided by core braced rigid frames and pure rigid frames. In the long direction only rigid frames provide the resistance. As a consequence of the 56 foot bay size, the K-factors for the exterior columns were about 3, for the cruciform interior columns, 5 to 6, and for the unframed columns, infinity. The problem was, how to arrive at a satisfactory design approach.

Fig. 2 shows the P-delta approach adopted for the entire framework. The problem is represented by a simply framed column tied to a rigid frame. The forces acting on the system in a laterally displaced position are depicted.

For stability, the simple column requires a stabilizing force of $P_2\Delta/h$ to maintain equilibrium. Fig. 2(d) shows the added force acting on the rigid frame.

In Iffland's 60 story example in the Guide, the K-factor approach suggested is to add the P-delta forces arising from the simply framed columns to the external lateral forces which are resisted by the rigidly framed columns. Provided that the rigid framed columns are similar, this approach may be satisfactory.

In the 57 story building shown previously, either the exterior columns, with K about 3, would have to take all the lateral forces plus stabilizing forces for all the other columns or the cruciform columns would have to be designed for a K factor of 6.

Fig. 3 illustrates the calculation of the fictitious P-delta lateral forces. The story shears are calculated from the top downwards, using the total load on all of the columns. The fictitious forces at each floor are the algebraic differences between the shears in successive stories.

PANEL DISCUSSION

Important findings on the 57 story building were that:

1. The second order effects were more severe in the braced direction than the rigid frame direction. Simple application of the design code would have included the P-delta forces in the bracing design but not in the complementary rigid frames.

2. Working stress design can give a misleading understanding of the structure. The columns were stressed fully but the spandrel beams were not, for drift control, suggesting a column weak structure. In fact, M_{pc} of the columns far exceeded M_p for the beams. The structure was desirably beam weak. The sway forces were more severe in the braced direction than in the rigid framed direction.

The load combinations required by the Canadian Limit-States Standard are shown below, followed by transposed equations for comparison with the AISC. AISC factors in the first equation would be 1.7 and in the last 1.3. The AISC does not require the second combination, but for a tall building, it controls generally.

C.S.A. S16.1 - 1974
LOAD COMBINATIONS

$$0.9R = 1.25D + 1.5L$$
$$0.9R = 1.25D + 1.5W$$
$$0.9R = 1.25D + 0.7\ (1.5L + 1.5W)$$

$$R = 1.39D + 1.67L$$
$$R = 1.39D + 1.67W$$
$$R = 1.39D + 1.17L + 1.17W$$

The Canadian recommended procedure for second order elastic analysis or the P-delta method is described below. Note that the cycling procedure may be impractical for normal design office practice. I usually will estimate the drift first, enter at state 3, go through 4, 5, and 2. If the results seem reasonable, I stop there.

1. Apply factored loads.
2. Δ - first order elastic analysis.
3. P-delta story shears.
4. Ficticious lateral loads.
5. Add P-delta loads to lateral loads.
6. Repeat 2-5 until convergence.

If working stress design is being used, it is necessary to amplify the delta values to account for the non-linear load deformation curve up to ultimate load. Generally, the same value for $\overline{F}$ as the implied safety factor is used.

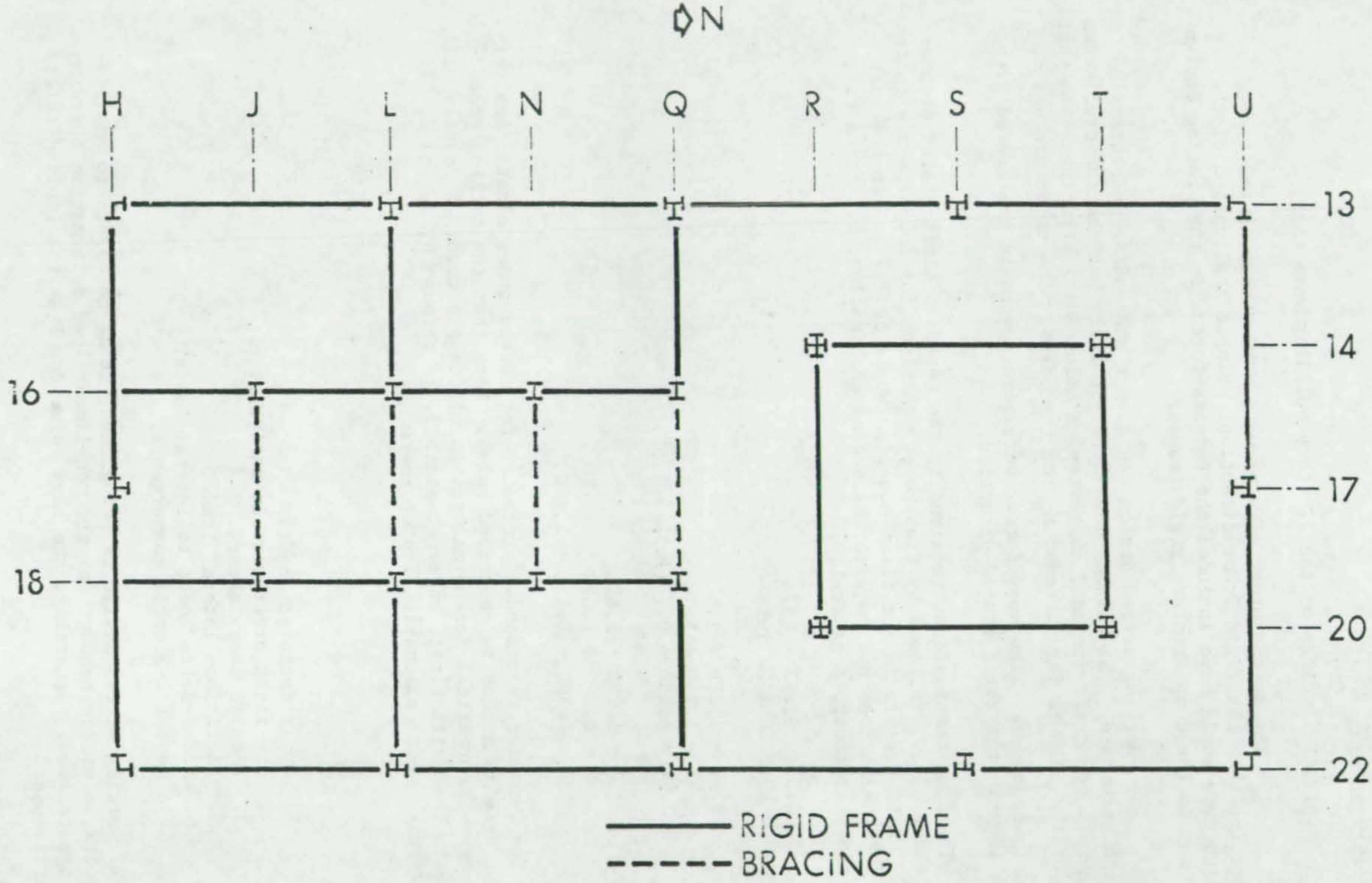


FIG. 1

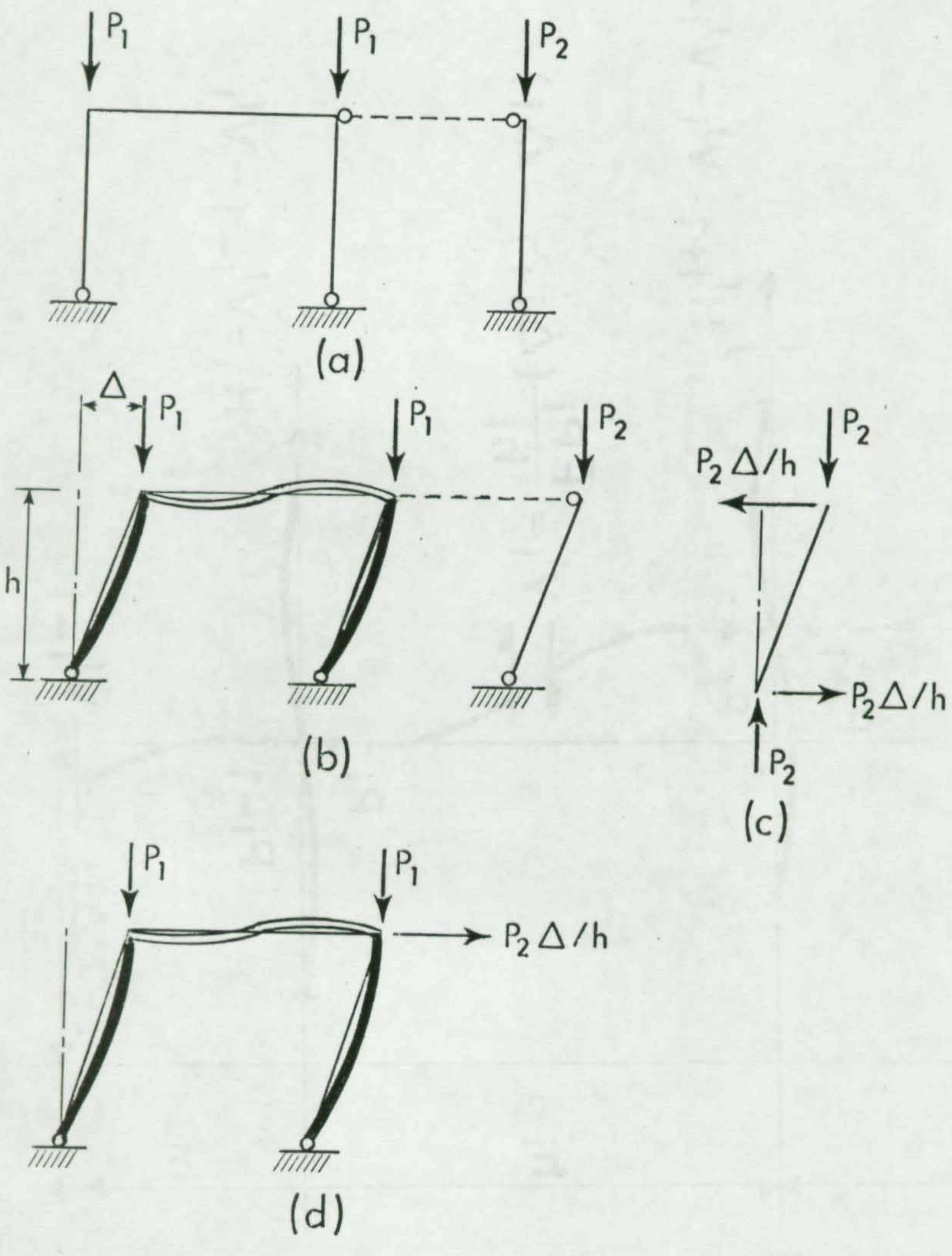
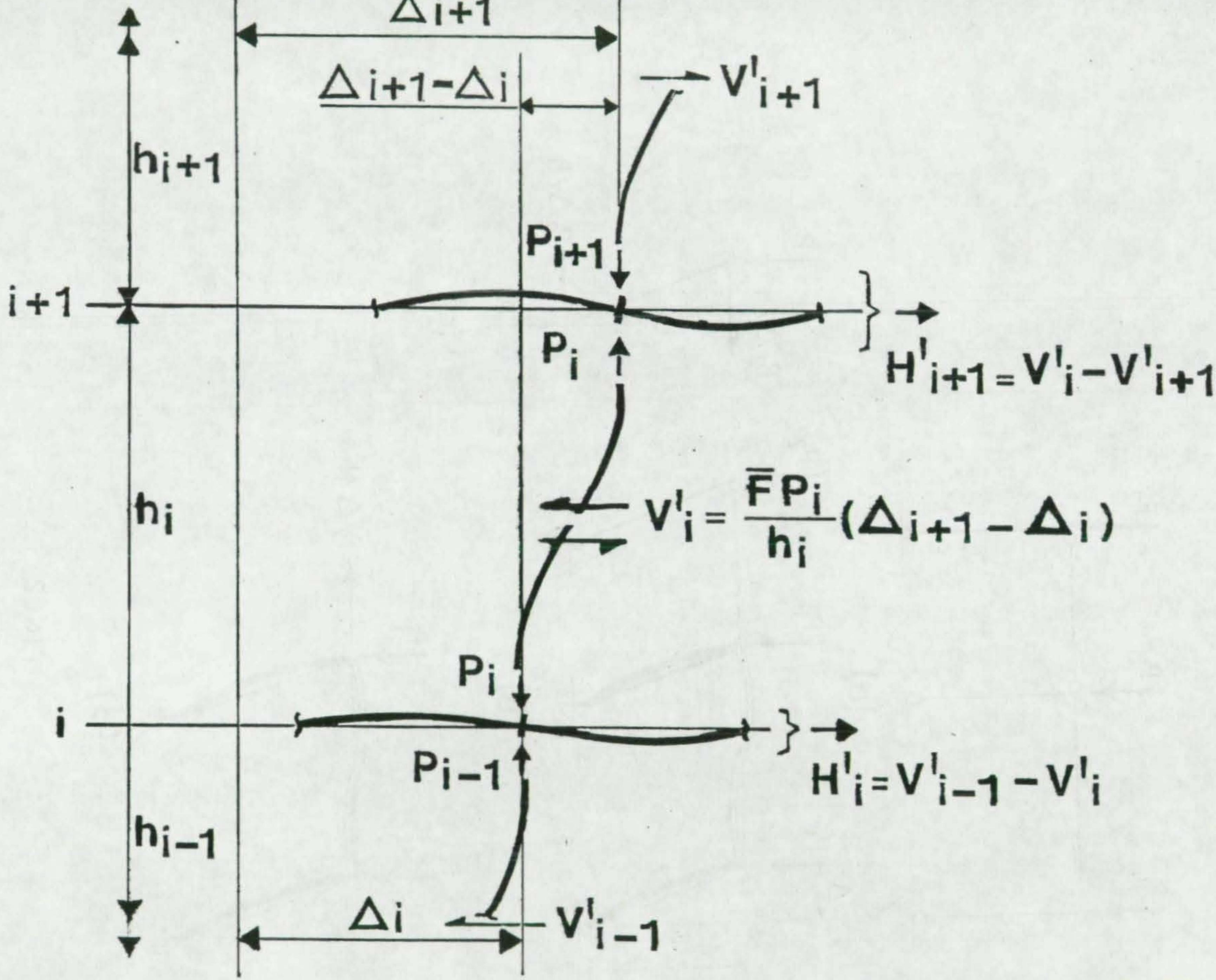
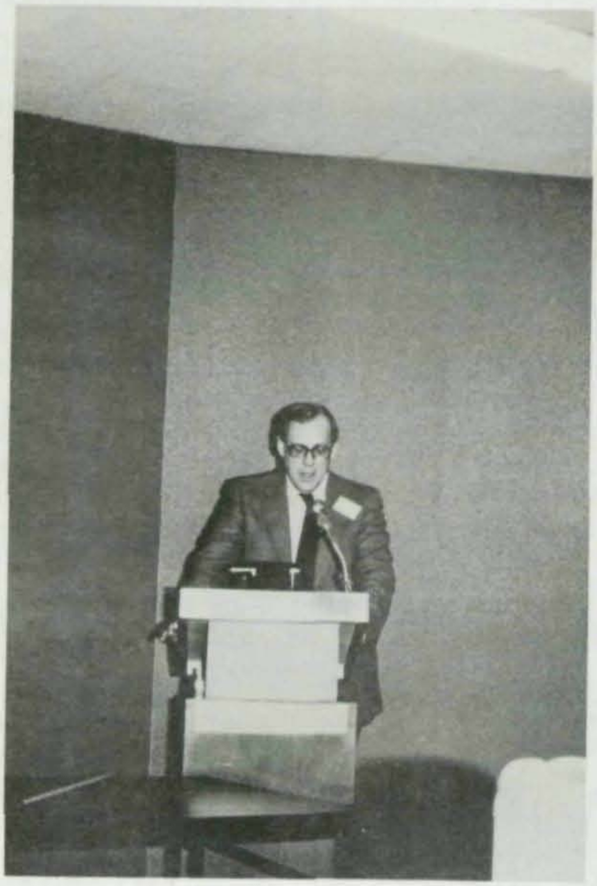


FIG. 2



Δi+1
Δi+1−Δi
V'i+1
hi+1
Pi+1
i+1
Pi
H'i+1 = V'i − V'i+1
hi
V'i = F̄Pi/hi (Δi+1 − Δi)
Pi
i
Pi−1
H'i = V'i−1 − V'i
hi−1
Δi
V'i−1

FIG. 3



J.S.B. Iffland, Moderator

1976 ANNUAL BUSINESS MEETING

The Structural Stability Research Council (formerly Column Research Council) holds an annual meeting for the purpose of reporting activities, election of members and officers, and presentation of the budget for the following year. The Structural Stability Research Council held its first Annual Meeting under its new name on March 3, 1976, in conjunction with the Technical Session at the Sheraton-Biltmore Hotel in Atlanta, Georgia.

The minutes of the 1976 Annual Meeting follow:

CALL TO ORDER

The meeting was called to order at 11:30 a.m. by the Chairman of the Council, Professor George Winter. Thirty-eight people were present, the majority being members of the Council.

The Chairman introduced himself, the Vice-Chairman, Dr. J. W. Clark, the Secretary, Dr. F. Cheong-Siat-Moy, and welcomed the members and friends.

ELECTION OF EXECUTIVE COMMITTEE MEMBERS

D. R. Sherman, chairman of the Nominating Committee, presented the slate. W. J. Austin was nominated for a three-year term, S. J. Errera was nominated to fill the one-year unexpired term of L. A. Boston, and W. A. Milek, Jr. and J. S. B. Iffland were re-nominated for three-year terms. The motion that the four nominees be elected was carried unanimously.

MEMBERS-AT-LARGE

The following were nominated for election to Member-at-Large: M. C. Temple, P. C. Birkemoe, T. M. Baseheart, and R. C. Young. All were elected by unanimous vote.

The following were also nominated and elected pending inquiry as to their desire to become Members-at-Large: D. O. Brush, University of California; B. O. Almroth, Lockheed; Prof. G. J. Simitses, Georgia Institute of Technology; and R. B. Testa, Columbia University.

LIFE MEMBERS

R. R. Graham, chairman of the Nominating Committee for Life Members, submitted the following slate to the attendance for voting: W. J. Austin, L. S. Beedle, E. L. Erickson, E. H. Gaylord, J. A. Gilligan, J. E. Goldberg, T. R. Higgins, N. J. Hoff, S. C. Hollister, L. K. Irwin, B. G. Johnston, T. C. Kavanagh, R. L. Ketter, N. M. Newmark, B. Thurlimann, and G. Winter. The motion that all 16 nominees be elected Life Members was carried unanimously.

SECOND INTERNATIONAL COLLOQUIUM

The Chairman of the Council gave the following details about the Second International Colloquium on Structural Stability:

DATE:	May 17-19, 1977
PLACE:	Washington, D. C.
ORGANIZERS:	European Convention of Constructional Steelworks (ECCS) Structural Stability Research Council (SSRC) International Association for Bridge & Structural Engineering (IABSE) Column Research Committee of Japan (CRC (Japan))

He also mentioned that a unique feature of the Colloquium is that it will be held as three separate sessions in three different countries: Belgium, Japan and U. S. A.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Chairman of the Council, including the proposed budget for the Fiscal Year 1976-77.

Budget 1976-77:

Expected balance, Oct. 1, 1976	$ 4,300
Income	22,500
Expenditures	22,825
Expected balance, Sep. 30, 1977	3,975

The budget was approved unanimously.

The Chairman thanked G. F. Fox, Chairman of the Finance Committee, for his successful attempt in obtaining contributions from Participating Firms. He also emphasized that additional sources of funds need be found to help the Council's financial situation.

NEXT ANNUAL MEETING

The Chairman announced that the next Annual Meeting of the Council will be held in Washington, D. C. in conjunction with the Second International Colloquium.

ADJOURNMENT

The meeting was adjourned at 11:45 a.m.

TECHNICAL SESSION & ANNUAL MEETING ATTENDANCE

Participant	Affiliation
Abrahams, M. J.	Parsons, Brinkerhoff, Quade & Douglas, New York
Anand, S. C.	Clemson University, South Carolina
Austin, W. J.	Rice University, Houston, Texas
Beedle, L. S.	Lehigh University, Bethlehem, Pa.
Beil, R. E.	Sverdrup & Parcel and Assoc., Inc., St. Louis
Bernstein, M. D.	Foster Wheeler Energy Corp., Livingston, N. J.
Birkemoe, P. C.	University of Toronto, Toronto
Bjorhovde, R.	American Institute of Steel Construction, New York
Boston, L. A.	J. Ray McDermott & Co., New Orleans, La.
Chen, W. F.	Lehigh University, Bethlehem, Pa.
Cheong-Siat-Moy, F.	Lehigh University, Bethlehem, Pa.
Chong, K. P.	University of Wyoming, Laramie
Clark, J. W.	Aluminum Company of America, Alcoa Center, Pa.
Critchfield, M. O.	D. W. Taylor Naval Ship R&D Center, Bethesda, Md.
Douty, R. T.	University of Missouri-Columbia, Mo.
duBouchet, A. V.	Rutgers University, New Brunswick, N. J.
Edwards, W. E.	Bethlehem Steel Corporation, Bethlehem, Pa.
Elgaaly, M.	Bechtel Corporation, Ann Arbor, Michigan
Ellifritt, D. S.	Metal Building Manufacturers Assoc., Cleveland
Errera, S. J.	Bethlehem Steel Corporation, Bethlehem, Pa.
Fox, G. F.	Howard, Needles, Tammen & Bergendoff, New York
Galambos, C. F.	Federal Highway Administration, Washington, D. C.
Galambos, T. V.	Washington University, St. Louis, Mo.
Gilmor, M. I.	Canadian Institute of Steel Construction, Willowdale
Graham, R. R.	U. S. Steel Corporation, Pittsburgh, Pa.
Hall, D. H.	Bethlehem Steel Corporation, Bethlehem, Pa.
Hartmann, A. J.	Westinghouse Electric Co., Penna.
Hollister, S. C.	Cornell University, Ithaca, New York
Householder, J.	Georgia Institute of Technology, Atlanta, Ga.
Iffland, J. S. B.	URS, Madigan-Praeger, Inc., New York
Johnson, D. L.	Butler Manufacturing Co., Grandview, Mo.
Keck, D. W.	Georgia Institute of Technology, Atlanta, Ga.
Lee, G. C.	State University of New York at Buffalo, New York
LeMessurier, W. J.	Sippican Consultants International, Cambridge, Mass.
Lu, L. W.	Lehigh University, Bethlehem, Pa.

Marshall, P. W.	Shell Oil Company, Houston, Texas
McGarry, R.	American Bridge Dividion, U.S. Steel, Pittsburgh
Meith, R. M.	Chevron Oil Company, New Orleans, La.
Milek, W. A.	American Institute of Steel Construction, New York
Miller, C. D.	Chicago Bridge & Iron Company, Plainfield, Ill.
Morrell, M. L.	Clemson University, South Carolina
Nethercot, D. A.	University of Sheffield, England
Palmer, F. J.	American Institute of Steel Construction, New York
Paulet, E. G.	Federal Highway Administration, Washington, D. C.
Razzaq, Z.	Arizona State University, Tempe, Arizona
Ringo, B. C.	University of Cincinnati, Ohio
Ross, D. A.	Lehigh University, Bethlehem, Pa.
Ruby, D.	John Portman & Associates, Atlanta, Ga.
Sears, C.	American Bridge Division, U. S. Steel, Chicago
Sherman, D. L.	University of Wisconsin-Milwaukee, Wisconsin
Springfield, J.	C. D. Carruthers & Wallace, Rexdale, Ontario
Will, K. M.	Georgia Institute of Technology, Atlanta, Ga.
Yu, W. W.	University of Missouri-Rolla, Mo.
Yura, J. A.	University of Texas, Austin, Texas
Winter, G.	Cornell University, Ithaca, New York

List of Publications

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

American Petroleum Institute
RECOMMENDED PRACTICE FOR PLANNING, DESIGNING, AND CONSTRUCTING FIXED OFFSHORE PLATFORMS, API RP2, Sixth Edition, Jan 1975

European Convention for Constructional Steelwork
MANUAL ON THE STABILITY OF STEEL STRUCTURES, Introductory Report, Second International Colloquium on Stability, 1976

Ingvarsson, H., and Sundquist, H.
ELASTICITETSTEORETISK BEHANDLING AV PLATTOR UPPLAGDA PÅ KANT-ELLER HÖRNPELARE, Institutionen för Byggnadsstatik Kungl. Tekniska Högskolan, Maddelande No. 116, Stockholm 1975

Johansson, B.
LÅDBALKAR MED TUNNA LIV (BOX GIRDERS WITH THIN WEBS), Institutionen för Byggnadsstatik Kungl. Tekniska Högskolan, Maddelande No. 117, Stockholm 1976

Johns, T. G., Mesloh, R. E., Winegardner, R., and Sorenson, J. E.
INELASTIC BUCKLING OF PIPELINES UNDER COMBINED LOADS, Paper No. OTC 2209 presented at Seventh Annual Offshore Technology Conference, Houston, 5-8 May 1975, Copyright 1975

Johnston, B. G.
THE NEW GUIDE TO STABILITY DESIGN CRITERIA FOR METAL STRUCTURES, Engineering Journal, Third Quarter, 1976/Vol. 13, No. 3, pp. 65-67

Johnston, B. G.
DESIGN APPLICATIONS: SSRC GUIDE, THIRD EDITION, ASCE Annual Convention and Exposition Preprint 2802, Philadelphia, Sep. 27-Oct 1, 1976

Nylander, H., and Holmgren, J.
UPPLAGSHÅLLFASTHET VID HÖG BALK FÖRBUNDEN MED BJÄLKLAGSPLATTA och UPPLAGSHÅLLFASTHET VID BETONGBALK MED REKTANGULÄRT TVÄRSNITT, Institutionen för Byggnadsstatik Kungl. Tekniska Högskolan, Maddelande No. 113, Stockholm 1975

Ross, D. A., and Chen, W. F.
TESTS OF FABRICATED TUBULAR COLUMNS, Paper presented to Structural Division of ASCE Convention, San Diego, 5-8 Apr 1976, Lehigh/FL/393-7(76), Jan 1976

Ross, D. A., and Chen, W. F.
THE AXIAL STRENGTH AND BEHAVIOR OF CYLINDRICAL COLUMNS, Paper No. OTC 2683 presented at Eighth Annual Offshore Technology Conference, Houston, 3-6 May 1976, Lehigh/FL/393-6(76), Feb 1976

Springfield, J.
DESIGN OF COLUMNS SUBJECT TO BIAXIAL BENDING, Reprinted from AISC Engineering Journal, Third Quarter/1975

Structural Stability Research Council
GUIDE TO STABILITY DESIGN CRITERIA FOR METAL STRUCTURES, 3RD EDITION, B. G. Johnston, Editor, John Wiley & Sons, Apr 1976

Vinnakota, S.
INFLUENCE OF IMPERFECTIONS ON THE MAXIMUM STRENGTH OF BIAXIALLY BENT COLUMNS, Reprinted from the Canadian Journal of Civil Engineering, Vol. 3, No. 2, 1976, pp. 186-197

SSRC Chronology

Oct 75	-	New category "Participating Firms" established
24-25 Nov 75	-	Third International Specialty Conference on Cold-Formed Steel Structures, St. Louis (CRC - Cooperating Organization)
12 Dec 75	-	Chairman's Meeting, New York City
1-3 Mar 76	-	Executive Committee Meeting, Technical Session and Annual Meeting, Atlanta, Georgia
1 Mar 76	-	Council name officially changed to "Structural Stability Research Council (formerly Column Research Council)"
3 Mar 76	-	First Life Members elected to Council (16)
Apr 76	-	Third Edition of SSRC "Guide to Stability Design Criteria for Metal Structures" published by John Wiley & Sons
9 Sep 76	-	Second International Colloquium on Stability of Steel Structures, Pre-Session, Tokyo, Japan
30 Sep 76	-	ASCE Meeting, Philadelphia (Session on Design Applications of the SSRC Guide)

Finance

	Fiscal Year 10/75 - 9/76		Fiscal Year 10/76 - 9/77
	Budget (approved 5/7/75)	Cash Statement 10/1/75-9/30/76	Budget (approved 3/3/76)
BALANCE at Beginning of Period	$ 4,000.00	$ 8,942.14 (a)	$ 4,300.00
INCOME			
Contributions			
Alum. Assoc.	500.00	500.00	500.00
AISC	4,000.00 (b)	7,500.00 (c)	4,000.00 (b)
AISI	5,000.00	--- (d)	5,000.00
CISC	1,000.00	---	1,000.00
ICBO	50.00	50.00	---
MBMA	---	1,000.00	---
NSF	6,000.00 (e)	---	7,000.00 (e)
SEASC	100.00	---	---
SESA	100.00	100.00	100.00
SJI	200.00	200.00	200.00
Participating Firms	---	1,800.00	1,500.00
Total Contributions	$16,950.00	$11,150.00	$19,300.00
Registration Fees	1,200.00	1,050.00	3,000.00
Subscription Fees	1,000.00	756.00	---
Guide Royalties	200.00	131.37 (f)	--- (f)
Interest	150.00	171.40	200.00
TOTAL INCOME	$19,500.00	$13,258.67	$22,500.00
EXPENDITURES			
Technical Services (Hqtrs)			
Director	500.00	700.00	1,000.00
SSRC Secretary	4,000.00	4,060.53	4,500.00
Secretarial Services	3,500.00	3,833.58	3,000.00
Supply, phone, mailing	750.00	1,355.84 (g)	750.00
Travel	750.00	233.86	750.00
Total Tech. Services	$ 9,500.00	$10,183.81	$10,000.00
Research	---	---	1,000.00
Guide			
Contract on Revision	2,100.00	2,100.00	525.00
Travel and Expenses	1,400.00	1,400.00	300.00
SSRC Secretary and Secretarial Services	400.00	469.10	---
Hqtrs Expenses (Mailing,copying)	400.00	---	---
Total Guide	$ 4,300.00	$ 3,260.59	$ 825.00
Annual Meeting & Proceedings			
Expenses and Services	3,000.00	2,120.23 (h)	8,000.00
Travel & Conf. Hotel Bill	2,000.00	1,090.62	2,000.00
Total Annual Meeting	$ 5,000.00	$ 3,210.85	$10,000.00
United Engineering Trustees	100.00	100.00	100.00
Travel	300.00	98.31	500.00
Contingencies	300.00	100.00 (i)	400.00
TOTAL EXPENDITURES	$19,500.00	$16,953.56	$22,825.00
BALANCE at End of Period	$ 8,942.14	$ 5,247.30 (j)	$ 3,975.00

EXPLANATORY NOTES

(a) Depositories (as of 9/30/75)

General Account (UETI)	$ 9,643.15
Technical Services (Lehigh Univ.)	(1,170.11)
SSRC Guide	469.10
	$ 8,942.14

(b) AISC has increased its general contribution, exclusive of Guide, from $2,500 to $4,000.

(c) Includes $1,500 (Guide Support for 1975), $2,500 (General Support for 1975), $1,500 (Guide Support for 1976) and $2,000 (General Support for 1976). $2,000 for General Support received after 9/30/76. Not reflected in Cash Statement.

(d) $5,000 has been received after 9/30/76. Not reflected in Cash Statement.

(e) Although budgeted at a lower amount, an application for $23,000 for the International Colloquium has been made.

(f) Cash Statement figure for 2nd Edition of Guide. No royalties are budgeted for FY 76-77. Council received $2,000 advance on 3rd Edition royalties.

(g) Includes Guide and some Annual Meeting expenses.

(h) Proceedings costs were lower than anticipated. Sheraton-Biltmore Hotel provided the conference room and some other expenses free of charge.

(i) Purchase of Springfield's paper.

(j) Depositories (as of 9/30/76)

General Account (UETI)	$11,888.85
Technical Services (Lehigh Univ.)	(6,641.55)
	$ 5,247.30

Register

OFFICERS

Chairman	G. Winter
Vice Chairman	J. W. Clark
Secretary	F. Cheong-Siat-Moy

EXECUTIVE COMMITTEE

G. Winter (77)
J. W. Clark (77)
L. S. Beedle (Director)
W. J. Austin (79)
K. P. Buchert (77)
J. L. Durkee *
S. J. Errera (77)
G. F. Fox (78)
T. V. Galambos **
R. R. Graham (78)
T. R. Higgins (Technical Consultant)
J. S. B. Iffland (79)
B. G. Johnston (78)
W. A. Milek, Jr. (79)
J. Springfield (77)

* Past Vice Chairman
** Past Chairman

STANDING & AD HOC COMMITTEES

A. Committee on Guide to Stability Design Criteria for Metal Structures

B. G. Johnston, Editor 3rd Edition
T. V. Galambos, Editor 4th Edition

B. Committee on Finance

G. F. Fox, Chairman
G. Winter
F. Cheong-Siat-Moy

C. Ad Hoc Committee on Research Priorities

J. S. B. Iffland, Chairman
R. Bjorhovde
S. J. Errera
T. V. Galambos
R. M. Meith

TASK GROUPS

Task Group 1 - Centrally Loaded Columns

J. A. Gilligan, Chairman	J. L. Durkee	A. F. Kirstein
R. Bjorhovde, V. Chairman	M. P. Gaus	W. A. Milek, Jr.
L. S. Beedle	R. R. Graham*	E. G. Paulet
W. F. Chen	D. H. Hall	T. Pekoz
J. W. Clark	A. L. Johnson	L. Tall

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 - Columns With Biaxial Bending

J. Springfield, Chairman*	L. W. Lu	S. U. Pillai
M. J. Abrahams	P. W. Marshall	Z. Razzaq
W. F. Chen	D. A. Nethercot	B. C. Ringo
		S. Vinnakota

Scope: To investigate the behavior of columns subjected to biaxial bending, and to develop rational stability criteria based on the ultimate strength of such members.

Task Group 4 - Frame Stability and Effective Column Length

J. S. B. Iffland, Chairman*	E. H. Gaylord	L. W. Lu
P. F. Adams	O. Halasz	W. A. Milek, Jr.
C. Birnstiel	T. R. Higgins	C. K. Wang
J. H. Daniels	I. M. Hooper	J. A. Yura
W. E. Edwards	B. G. Johnston	

Scope: To develop procedures for investigating the stability of structural frameworks. The evaluation of when the effective-column-length concept should be used, and when not, is an important consideration.

Task Group 6 - Test Methods for Compression Members

T. Pekoz, Chairman	J. W. Clark	B. G. Johnston
L. S. Beedle	S. J. Errera*	B. M. McNamee
R. Bjorhovde	T. R. Higgins	D. R. Sherman
		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.

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* Executive Committee Contact Member

Task Group 7 - Tapered Members

A. Amirikian, Chairman
J. H. Adams
D. J. Butler
T. R. Higgins*
D. L. Johnson
R. L. Ketter
K. H. Koopman
C. F. Larson
G. C. Lee
L. W. Lu
W. A. Milek, Jr.
A. A. Toprac
I. M. Viest
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*
I. K. McIvor
G. J. Simitses
J. C. Simonis

Scope: To define the strength of columns and other compression members subjected to time-dependent loading.

Task Group 11 - International Cooperation on Stability Studies

D. Sfintesco, Chairman
W. A. Milek, Jr., V. Chairman*
G. A. Alpsten
L. S. Beedle
A. Carpena
C. A. Cornell
T. V. Galambos
M. P. Gaus
B. Kato
M. G. Lay
P. Marek
R. K. McFalls
B. M. McNamee
E. O. Pfrang
G. W. Schulz
J. Strating
L. Tall
I. M. Viest

Scope: To coordinate American, Japanese and European 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, and Committee 8 of the European Convention of Constructional Steelwork; and to suggest joint research projects.

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

R. Testa, Chairman
G. A. Alpsten
G. F. Fox*
A. L. Johnson
B. G. Johnston
L. W. Lu
M. Shinozuka
W. J. Wilkes

Scope: To obtain and interpret data on the mechanical properties of 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

S. J. Errera, Chairman	A. L. Johnson	E. P. Popov
J. W. Clark	C. Marsh	W. P. Vann
J. A. Gilligan	A. Ostapenko	G. Winter*
	T. Pekoz	W. W. Yu

Scope: To investigate the 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 the fabrication processes.

Task Group 14 - Horizontally Curved Girders

A. P. Cole, Chairman	C. G. Culver	W. A. Milek, Jr.
R. Behling	J. L. Durkee*	M. Ojalvo
H. R. Brannon	E. R. Latham	S. Shore
	P. Marek	W. M. Thatcher

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

T. V. Galambos, Chairman*	A. J. Hartmann	M. Ojalvo
Y. Fukumoto	D. A. Nethercot	N. S. Trahair
		J. A. Yura

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

F. D. Sears, Chairman	R. S. Fountain	C. Massonnet
K. Basler	K. L. Heilman	A. Ostapenko
P. B. Cooper	B. G. Johnston	B. T. Yen
J. L. Durkee*	H. S. Lew	R. C. Young

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 - Stability of Shell-Like Structures

A. Chajes, Chairman	J. W. Clark	C. D. Miller
J. H. Adams	A. L. Johnson	E. P. Popov
L. O. Bass	A. Kalnins	C. F. Scheffey
J. Bruegging	D. Krajcinovic	D. R. Sherman
K. P. Buchert*	C. Libove	J. C. Simonis
		D. T. Wright

Scope: To investigate the stability of shell-like structures (those structures where the load-carrying elements also serve the functional requirements of enclosing space).

Task Group 18 - Unstiffened Tubular Members

D. R. Sherman, Chairman	A. Chajes	P. W. Marshall
B. O. Almroth	R. R. Graham*	R. M. Meith
M. D. Bernstein	T. G. Johns	C. D. Miller
P. C. Birkemoe	J. N. MacAdams	R. L. Rolf

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

S. H. Iyengar, Chairman	R. W. Furlong	D. Sfintesco
L. S. Beedle*	B. Kato	M. Wakabayashi
P. Dowling	J. W. Roderick	

Scope: To develop stability criteria for various types of composite columns.

Task Group 21 - Box Girders

R. C. Young, Chairman	G. F. Fox*

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	J. W. Cox	R. K. Kinra
M. D. Bernstein	R. C. DeHart	R. M. Meith
K. P. Buchert *	N. W. Edwards	R. L. Rolf
	R. F. Jones	R. C. Tennyson

Scope: Will consider cylinders with longitudinal or circumferential stiffening alone or in combination. Stability criteria will be developed for axial load, external or internal pressure, beam type bending and torsion. Consideration will be given to local buckling and general instability type failures. Available test data will be compared with suggested stability criteria. Recommendations will be made where insufficient data is available. The first task will be to develop criteria for axial load. External pressure criteria presented in Chapter 10 of the SSRC Guide will be reviewed.

TASK REPORTERS

Task Reporter 11 - Stability of Aluminum Structural Members

J. W. Clark, Aluminum Company of America

Task Reporter 13 - Local Inelastic Buckling

L. W. Lu, Lehigh University

Task Reporter 14 - Fire Effects on Structural Stability

L. S. Seigel, U. S. Steel Corporation

Task Reporter 15 - Curved Compression Members

W. J. Austin, Rice University

Task Reporter 16 - Stiffened Plate Structures

R. G. Kline, David J. Seymour, Ltd.

Task Reporter 17 - Laterally Unsupported Restrained Beam-Columns

L. W. Lu, Lehigh University

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* Executive Committee Contact Member

PARTICIPATING ORGANIZATIONS

Organization	Representatives	Officers
Aluminum Association	J. W. Clark G. S. Gresham	P. V. Mara Vice Pres - Technical
American Association of State Highway & Transportation Officials	H. W. Derthick	H. E. Stafseth Executive Director
American Institute of Architects	P. E. Kirven T. F. Mariani J. S. Hawkins, Jr.	J. R. Dowling Dir., Codes & Regs. Ctr.
American Institute of Steel Construction	T. R. Higgins J. L. Durkee	W. A. Milek, Jr. Dir. of Engr. & Research
American Iron and Steel Institute	W. G. Kirkland A. L. Johnson	R. T. Willson Sr. Vice President
American Petroleum Institute	J. E. Ubben W. D. Greenfield	J. H. Sybert
American Society of Civil Engineers	S. C. Hollister B. G. Johnston T. C. Kavanagh	E. Zwoyer Executive Director
American Society of Mechanical Engineers	M. D. Bernstein	R. B. Finch Executive Dir. & Secy
American Water Works Association	R. E. Vansant	E. F. Johnson Executive Director
Association of American Railroads	L. S. Beedle	E. W. Hodgkins Executive Secretary
Canadian Institute of Steel Construction	H. A. Krentz M. I. Gilmor	R. G. Johnson President
Corps of Engineers U. S. Army	G. M. Matsumura	LTG W. C. Gribble, Jr. Chief
Earthquake Engineering Research Institute	H. J. Degenkolb R. D. Hanson T. R. Higgins	H. J. Degenkolb President
Engineering Foundation		J. A. Zecca Secretary
Engineering Institute of Canada	J. S. Ellis R. F. Hooley D. T. Wright	B. T. Kerr General Manager
Federal Highway Administration	E. G. Paulet R. Varney F. D. Sears	N. T. Tiemann Administrator

European Convention of Constructional Steelworks		D. Sfintesco Tech. Secy General E. B. Evers Admin. Secy General
General Services Administration	A. Dailey	Administrator
Institution of Engineers, Australia	G. Sved	R. D. Henderson Committee Secretary
International Conference of Building Officials	D. R. Watson	J. E. Bihr Managing Director
International Nickel Company, Inc.		W. S. Mounce Mgr. Commercial Development
Langley Research Center National Aeronautics & Space Administration	M. Stein	E. M. Cortright Director
Metal Building Manufacturers Association	D. L. Johnson F. A. Petersen D. S. Ellifritt	F. A. Petersen General Manager
National Bureau of Standards	A. F. Kirstein H. S. Lew	A. F. Kirstein
National Research Council	R. C. Edgerton J. R. Smith	J. S. Coleman Executive Officer
Naval Facilities Engineering Command, U. S. Navy	M. Yachnis	RADM A. R. Marschall Commander
Naval Ship Research and Development Center	M. O. Critchfield	CAPT M. C. Davis Commander
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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 problems relating to the design and behavior of columns and other compression elements in metal structures can be presented for evaluation and pertinent structural research problems proposed for investigation.

2. To digest critically the world's literature on structural behavior of compression elements and to study the properties of metals available for their construction, and make the results widely available to the engineering profession.

3. To organize, administer, and guide cooperative research projects in the field of compression elements, and to enlist financial support for such projects.

4. To promote publication and dissemination of original research information in the field of compression elements.

5. To study the application of the results of research to the design of compression elements; to develop comprehensive and consistent strength and performance criteria, and to encourage their consideration by specification-writing bodies.

*Revised: August 21, 1947; October 1, 1948; November 1, 1949; August 15, 1951; May 20, 1955; October 1, 1960; May 7, 1962; May 21, 1965; May 31, 1968; March 27, 1974, May 7, 1975 and November 15, 1976

MEMBERSHIP OF THE COUNCIL

The membership of the Council shall consist of Members-at-Large, Corresponding Members, Representatives of Sponsoring Organizations, and Representatives of Participating Organizations.

An individual who has expressed interest in the work of the Council, and who has done or is doing work germane to its interest, may be elected Member-at-Large by the Council, following nomination by the Executive Committee.

Corresponding Members are appointed by the Executive Committee to maintain contact with organizations in other countries that are active in areas of interest to the Council.

A Representative is appointed by the Sponsoring Organization or by the Participating Organization subject to the approval of the Executive Committee, and continues to serve until replaced by the organization which he represents. A Sponsoring Organization may appoint up to five representatives, and a Participating Organization may appoint up to three representatives. Organizations concerned with investigation and design of metal compression members and structures may be invited by the Council to become Sponsoring Organizations or Participating Organizations.

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 contact each Member-at Large and each Corresponding Member to determine whether he wishes to continue his membership.

Every three years the Secretary of the Council shall canvass the Sponsoring Organizations and the Participating Organizations to determine their Representatives for the next three-year period.

SUBSCRIPTION FEES

The subscription fee for each Member-at-Large shall be $25.00 for a three-year period, and shall be billed concurrently with the regular triennial membership review. Interim subscriptions shall be $17.00 for a two-year period and $8.50 for a one-year period.

Subscription fees for Corresponding Members and Representatives shall be on a voluntary basis.

There shall be no subscription fees for Life Members.

The subscription fee for each Sponsoring Organization shall be a minimum of $1000 per year.

The subscription fee for each Participating Organization shall be a minimum of $100 per year, except that any such organization whose By-Laws specifically prohibit payment of such a fee shall be exempted.

M E E T I N G S O F T H E C O U N C I L

The Council shall hold at least one regular annual meeting each fiscal year, and such additional meetings as may be deemed necessary by the Executive Committee. A quorum shall consist of at least twenty members.

F I S C A L Y E A R

The fiscal year shall begin on October 1.

D U T I E S O F T H E C O U N C I L

1. To establish policies and rules.

2. To solicit funds for the work of the Council, and to maintain a general supervision of said funds, including the appropriation of grants for specific purposes.

3. To maintain and operate a central office for the administration of the work of the Council, and for the maintenance of its records.

4. To prepare an annual budget.

5. To issue annual reports.

6. To organize and oversee the committees and task groups established to carry out the projects authorized by the Council.

O F F I C E R S O F T H E C O U N C I L

1. The elected officers of the Council shall be a Chairman and a Vice Chairman. The Chairman shall exercise general supervision over the business affairs of the Council, subjected to the direction of the Council, shall perform all duties incident to this office, and shall be Chairman of the Executive Committee. It shall be the duty of the Chairman to preside at meetings of the Council and of the Executive Committee. The Vice Chairman shall perform all the duties of the Chairman in his absence.

2. The terms of office of the Chairman and Vice Chairman shall begin on October 1st and shall continue for 3 years. They shall be eligible for immediate re-election for only one term of one year. In the event of a 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 unexpired term.

3. There shall be a Director engaged by the Executive Committee subject to the approval of the Council, who shall be the chief executive paid officer of the Council. Additional paid officers may be appointed by the Council as may be necessary. If there is no paid Secretary, the Chairman may appoint a Secretary, who need not be a member of the Council.

4. The Director of the Council shall conduct the regular business of the Council subject to the general supervision of the Council and of the Chairman. The Director shall be expected to attend all meetings of the Council, Executive Committee, and main committees. The Director shall be ex-officio a member of the Council and the Executive Committee. The Director shall conduct the official correspondence of the Council, shall handle the financial affairs of the Council in accordance with an approved budget, and shall keep full records thereof. He shall carefully scrutinize all expenditures and exert every effort to secure economy in the business administration of the Council, and shall personally certify to the accuracy of all bills or vouchers on which money is to be paid. He shall engage such employees as may be authorized, shall be responsible for their work, and shall determine their salaries within budget limitations, subject to the approval of the Executive Committee. The salary of the Director and other paid officers shall be fixed by the Executive Committee. The Director shall draw up and execute all contracts authorized by the Council and its Executive Committee.

ELECTION OF OFFICERS

1. Each year, the Executive Committee shall appoint 3 members of the Council to serve as the Nominating Committee. One of the three shall be named Chairman by the Chairman of the Council. Members of the Executive Committee or of the previous year's Nominating Committee shall not be eligible to serve on the Nominating Committee.

2. The Nominating Committee shall name a slate for Chairman and Vice Chairman of the Council, and members of the Executive Committee. The Committee shall submit its nomination for Chairman and Vice Chairman to the Executive Committee prior to the Annual Meeting. Nominations for members of the Executive Committee will be submitted to the membership at the regular Annual Meeting.

3. The election of Chairman and Vice Chairman of the Council shall be by letter ballot. The ballots shall be canvassed at the regular Annual Meeting of the Council. Should no candidate for an office receive a majority of the ballots cast for such office, the annual meeting shall elect the officer by ballot from the two candidates receiving the largest number of votes in the letter ballot.

EXECUTIVE COMMITTEE

1. An Executive Committee of nine members shall be elected by the Council from its membership. The term of membership shall be for three years, and three of the members shall be elected each year at the time of the regular Annual Meeting of the Council. Nominations shall be made by the Nominating Committee as described in the section "Election of Officers". In addition, the Chairman, Vice Chairman, Director, and the most recent Past Chairman and Past Vice Chairman of the Council shall be ex-officio members of the Executive Committee. Members shall take office upon their election. They shall be eligible for immediate re-election.

Vacancies shall be filled by appointments by the Chairman from the membership of the Council, such appointees to serve for the remainder of the unexpired term.

2. The Executive Committee shall transact the business of the Council and shall have the following specific responsibilities and duties;

(a) To direct financial and business management for the Council, including the preparation of a tentative annual budget.

(b) To review and approve proposed research projects and contracts.

(c) To appoint Nominating Committee.

(d) To appoint chairmen of committees and task groups, and approve committee and task group members.

(e) To review reports and manuscripts.

(f) To advise Council on proposed research projects.

(g) To prepare program for Council meeting.

(h) To correlate and give general supervision to research projects.

(i) To refer inquiries relating to design practice to the Committee on Recommended Practice for definition, evaluation, and suggestions for task group assignment.

3. From time to time, the Executive Committee may ask additional consultants particularly interested in definite projects to act with it in an advisory capacity.

4. The Chairman, with the approval of the Executive Committee, shall appoint a Finance Committee to solicit the support required to carry out its projects.

5. The meeting of the Executive Committee shall be at the call of the Chairman or at the request in writing of two members of the Executive Committee. A quorum shall consist of five members, two of whom may be the Chairman and Vice Chairman of the Council.

6. The Executive Committee shall transact the business of the Council subject to the following limitations:

The minutes of the Committee shall be transmitted promptly to all members of the Council. If no objection is made by any member of the Council within two weeks after the minutes have been mailed, then the acts of the Executive Committee shall be considered as approved by the Council. If disapproval of any Committee action is made by three or more Council members, then the question raised shall be submitted to the Council for vote at a meeting called for that purpose, or by letter ballot.

CONTRACTS

The Council may make contracts or agreements, within its budget. Contracts for research projects preferably should be for the fiscal year period. Contracts with the Director or other paid employees of the Council may, with the approval of the Executive Committee, be for periods exceeding one fiscal year. At the end of such one-year period, contracts may be renewed or extended by the Council for an additional period, preferably not exceeding the new fiscal year.

STANDING AND SPECIAL COMMITTEES

1. The standing committees shall be a Committee on Finance and a Committee on the "Guide to Stability Design Criteria for Metal Structures". There shall be such special committees as may be approved by the Council.

2. Standing and special committees and their chairmen, shall be appointed by, and responsible to, the Executive Committee. They shall be named at a regular Annual Meeting of the Council, shall take office upon appointment, shall serve for three years, and shall be eligible for immediate reappointment. Vacancies shall be filled in the same manner as regular appointments except that such appointees will complete the term of office vacated.

3. The Committee on Finance shall solicit the support required to carry on the work of the Council. The Chairman and the Vice Chairman shall be appointed from among the membership of the Executive Committee.

4. The Committee on the "Guide to Stability Design Criteria for Metal Structures" shall direct the preparation and publication of the various editions of the "Guide".

RESEARCH COMMITTEES AND TASK GROUPS

1. The Executive Committee may authorize one or more research committees or task groups, each for a specific subject or field. Each committee or task group shall consist of a number of members as small as feasible for the work in hand. Members need not be members of the Council.

2. Research committee chairmen or task group chairmen shall be appointed by the Executive Committee, adequately in advance of the Annual Meeting of the Council.

3. All research committee or task group appointments shall expire at the time of the regular Annual Meeting of the Council. Prior to the Annual Meeting, each committee chairman or task group chairman for the ensuing year shall review the personnel of his committee or task group with the idea of providing the most effective organization, and shall make recommendations thereon to the Executive Committee. Committee or task group personnel shall be approved or modified by the Executive Committee, prior to the conclusion of the Annual Meeting of the Council.

4. The duties of a research committee or task group shall be:

(a) To review proposed research projects within its field, and to render opinions as to their suitability.

(b) To make recommendations as to needed research in its field.

(c) To give active guidance to research programs within its field, in which connection research committees or task groups are empowered to change details of programs within budget limitations.

(d) To make recommendations as to the time when a project within its field should be temporarily discontinued, or terminated.

(e) At the request of the Executive Committee to prepare summary reports covering results of research projects and/or existing knowledge on specific topics.

5. Each project handled by a research committee or task group shall be of definite scope and objective.

6. Each research committee or task group shall be responsible to the Executive Committee for organizing and carrying out its definite projects, which must be approved by the Executive Committee.

7. Each research committee or task group shall meet at least once in each fiscal year before the Annual Meeting of the Council, to review progress made, and to plan activities for the ensuing year.

8. Each research committee chairman or task group chairman shall make a report to the Executive Committee at the time of the Annual Meeting.

REVISION OF BY-LAWS

These By-Laws may be revised at any time upon a majority vote of the entire membership of the Council, by letter ballot or at a meeting of the Council.

Rules of Procedure*

I. OUTLINE OF ROUTE OF A RESEARCH PROJECT FOR CONSIDERATION BY THE STRUCTURAL STABILITY RESEARCH COUNCIL

Projects are to be considered under three classifications:

(1) Projects originating within the Structural Stability Research Council.

(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.

(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.

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

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.

II. OUTLINE OF A PATH OF A PROJECT THROUGH THE COUNCIL (FOR RECOMMENDED PRACTICE

1. Task Group submits its findings to the Executive Committee.

2. Executive Committee acts and forwards to Recommended Practice Committee.

3. Recommended Practice Committee acts and forwards recommendations to Executive Committee.

4. Council votes on the matter.

5. Executive Committee transmits recommendations and findings to specification-writing bodies, and/or Publications Committee arranges for publication.

III. 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 Stability Research Council.

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 make further limited distribution with a view of obtaining technical advice. General distribution will only be made after approval by the task group.

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.

IV. SSRC LIFE MEMBERS

Reason for Life Member Category - To facilitate continued participation in and contributions to SSRC activities on the part of Council members who:

1. Have given exceptionally long service to SSRC, or

2. Have given long service to SSRC and are on a reduced schedule of regular professional activity.

Guidelines for Nomination to Life Member Category

1. 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

2. Has made significant contributions to the work of SSRC: and

3. Expects to continue active participation in the work of SSRC.

Nominating Procedure

1. 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.

2. This committee will submit recommendations for Life Member nominees to the Executive Committee at its spring meeting.

3. Approved candidates will become Executive Committee nominees.

Election Procedure

The names of the Executive Committee nominees will be presented to the Council at its Annual Meeting, for election to Life Membership.