Wei-Wen Yu

STRUCTURAL STABILITY RESEARCH COUNCIL

(Formerly Column Research Council—Established in 1944)

Proceedings 1978

9391

About the Council

The Structural Stability Research Council (formerly Column Research Council) was founded in 1944 to remove the confusion and lack of harmony that existed at that time in solutions to stability problems and to facilitate and promote economical safe design.

The Council gives guidance to practicing engineers and specification writers in offering both simplified and refin[illegible] in assessing the [illegible] in part, through its [illegible] Design Criteria for [illegible] that is a cri[illegible] field.

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Th[illegible] des the designer w[illegible] ics, but it also i[illegible] present understand- i[illegible] ings are a product [illegible] ceedings form a [illegible] ies and represent [illegible] latest solutions [illegible] ventually published

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

Supported by a grant from the National Science Foundation (PFR 78-08818)

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Foreword

The Council can look back on another year of significant activity, the last of my four years as chairman.

A major new publication is the proceedings of last year's international colloquium, published as an 800 page volume by ASCE under the title "Stability of Structures Under Static and Dynamic Loads". It contains all the contributions, domestic and foreign of that three-day colloquium.

It will be recalled that this was the American meeting of the 2nd International Colloquium on the Stability of Steel Structures, three other similar meetings having been held in Japan, Belgium and Hungary, with appropriate cross-liaison among all four. In preparation now is a Comparison/Summary Report of the entire set of colloquia. Coordinating Editors are Duiliu Sfintesco in Paris, France, Gerald W. Schulz in Innsbruck, Austria, and Riccardo Zandonini (of whom more later) in Bethlehem, Pa. Regional Editors are T. V. Galambos for USA, B. Kato for Japan, O. Halasz for Hungary and D. Sfintesco for ECCS.

The Third Edition of the Guide having been published and selling well, organizational preparations are being made for the next, Fourth Edition. T. V. Galambos has been named, and has accepted to be the Editor, while B. G. Johnston, the former Editor, will be chairman of the Guide Committee.

The Annual Technical Session and Meeting in Boston last May was well attended. The theme of the panel discussion and of some of the presentations was the stability of composite steel-concrete structures. As an outgrowth of this, Task Group 20 - Composite Members, has established an ad hoc sub-committee on Strength of Composite Columns. Both TG-20 and the ad hoc sub-committee are chaired by S. H. Iyengar. The formation of this ad hoc group has an unusual pre-history. Composite columns are now covered in the ACI Building Code. Unfortunately, this coverage is inadequate for developing the full potentialities of such members. Recognizing this, the Structural Specifications Liaison Committee, under my chairmanship, formed a joint AISC-ACI group to come up with improved design provisions. Nothing came of this attempt. It was because of this that the ad hoc sub-committee of TG-20 was formed and given the same task. Since its formation, that group has developed considerable activity and, on the basis of drafts by R. W. Furlong of the University of Texas, Austin, hopes to come up with design provision proposals and an appropriate paper by about the end of the year.

A new task group, Task Group 23, was formed under the chairmanship of W. F. Chen to deal with The Effects of End Restraints on Initially Crooked Columns. In connection with the widespread tendency to base column design in the future on the strength not of ideally straight, but of initially crooked columns, it is of considerable importance to clarify the influence of end-restraints on such columns, whose strength, so far, was mostly investigated for the hinged-hinged end conditions.

The draft of Technical Memorandum No. 5, General Principles for the Stability Design of Metal Structures, produced by an ad hoc committee chaired by T. V. Galambos, was submitted to ballot by the entire membership. While the ballot was overwhelmingly favorable, a number of questions were raised in connection with TM-5. John Springfield has undertaken to study these comments in detail and to propose appropriate changes in the draft, where advisable.

The Council's finances continue in fairly good shape. In consequence, the item "Research Support", which had to be omitted for several years, has been put back into the budget. Envisaged grants, of which one is underway, are necessarily modest and serve mostly as seed money, but they help the Council more fully to play the role implied by its name.

On the administrative side, Dr. Riccardo Zandonini, who was mentioned earlier, joined our staff as Technical Secretary, unfortunately for a fairly restricted time. Mrs. Lesleigh Federinic, who has so effectively run the day-to-day affairs for a considerable length of time, has been appointed Administrative Secretary and will continue to keep the house in order and things going. L. S. Beedle, I am happy to report, continues as Director.

At the expiration of my own term in October 1978, John W. Clark became Chairman and J. S. B. Iffland, Vice-Chairman, having been elected to these offices by ballot. Unfortunately, chairman Clark has experienced a set-back in his health problem so that, temporarily we hope, Jerry Iffland is acting as chairman. The entire Council's best wishes go to John Clark.

At the end of my years as Chairman of the Council, let me again express my gratitude to all those at Headquarters, beginning with L. S. Beedle, but too numerous by now for individual mention, for the continued, indispensable, and effective efforts and contributions they have made toward making the Council a success in advancing the art and science of structural engineering.

George Winter

George Winter
SSRC Chairman
1974 - 1978

SSRC Executive Committee

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J. S. B. Iffland, Vice Chairman	- Iffland Kavanagh Waterbury
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W. J. Austin	- Rice University
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W. A. Milek, Jr.	- American Institute of Steel Construction
J. Springfield	- C. D. Carruthers & Wallace, Ltd.
G. Winter	- Cornell University

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 1978 Annual Technical Session was held on May 16 and 17 at the Copley Plaza Hotel in Boston, Massachusetts. One hundred and twenty persons attended the Session and twenty-seven papers were delivered.

A panel discussion on "Mixed Steel-Concrete Structures" was held in the evening of May 16, 1978. The panelists were I. M. Hooper, R. W. Furlong, and W. J. Le Messurier. The moderator was S. H. Iyengar.

In conjunction with the Technical Session, an Annual Business Meeting was held for the purpose of electing new officers and 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, May 16, 1978

8:00 a.m. - Registration

8:45 a.m. - MORNING SESSION

Presiding: J. L. Durkee, Modjeski and Masters

INTRODUCTION

G. Winter, Chairman, SSRC

TASK GROUP REPORTS

Task Group 13 - Thin-Walled Metal Construction

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

Optimization of Thin-Walled T-Shape Struts

C. Marsh, Concordia University

Stability Considerations in the Design of Cold-Formed Steel Storage Racks

T. Pekoz, Cornell University

Current Research on Cold-Formed Steel Beam Webs

W. W. Yu, N. Hetrakul, N. Phung, University of Missouri-Rolla
R. A. LaBoube, Iowa State University

Task Group 18 - Unstiffened Tubular Members

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

The Axial Strength of Tubular Columns Under Hydrostatic Loading

W. F. Chen and S. Toma, Purdue University

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

Chairman, R. B. Testa, Columbia University

Weld Shrinkage Stress Patterns

J. B. Dwight and J. D. White, Cambridge University

10:20 a.m. - BREAK

Task Group 4 - Frame Stability and Effective Column Length

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

Elastic Buckling Behavior of Unbraced Single-Story Single-Bay Orthogonal Space Frames

Z. Razzaq, Southern Illinois University at Carbondale
M. Naim, Arizona State University

Computer Bifurcation Analysis of Cable Stayed Rigid Frames

C. K. Wang and J. K. Stiller, University of Wisconsin-Madison

The Effect of Multicomponent Earthquake Motion on Columns of Three-Dimensional Steel-Concrete Building Systems

F. Y. Cheng, University of Missouri-Rolla

12 NOON - GROUP LUNCH

1:00 p.m. - AFTERNOON SESSION

Presiding: G. Winter, Cornell University

Task Group 8 - Dynamic Stability of Compression Elements

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

Crossflow Induced Instability of Circular Tubes

S. S. Chen, Argonne National Laboratory

Dynamic Stability of Structures

D. Krajcinovic, University of Illinois at Chicago Circle

Dynamic Stability of A Simple Frame Subjected to a Circulatory Load

D. E. Panayotounakos and A. N. Kounadis, National Technical University of Greece

Dynamic Buckling of Simple Frames Under a Step-Load

G. J. Simitses and J. Giri, Georgia Institute of Technology
A. N. Kounadis, National Technical University of Athens

2:15 - BREAK

Task Group 20 - Composite Members

Chairman, S. H. Iyengar, Skidmore, Owings and Merrill

Composite Columns

S. H. Iyengar, Skidmore, Owings and Merrill

Status of Composite Column Design Provisions

R.W. Furlong, University of Texas at Austin

Composite Columns & AISC Specifications

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

Composite Columns in Japan

R. W. Furlong, University of Texas at Austin

Effect of Cracking in Concrete Shear Walls of Composite Structures Under Lateral Loading

J. Springfield, C. D. Carruthers & Wallace, Ltd.

U.S. - Japan Seminar on Composite Structures and Mixed Structural Systems - A Summary

R. W. Furlong, University of Texas at Austin

4:30 p.m. - RECEPTION

Sponsored by Structural Steel Fabricators of New England

6:00 p.m. - PANEL DISCUSSION

Mixed Steel-Concrete Structures

Moderator: S. H. Iyengar, Skidmore, Owings and Merrill

Panelists: I. M. Hooper, Seelye Stevenson Value & Knecht
R. W. Furlong, University of Texas at Austin
W. M. Le Messurier, Le Messurier Associates/SCI

Wednesday, May 17, 1978

8:30 a.m. - MORNING SESSION

Presiding: T. V. Galambos, Washington University

Task Group 1 - Centrally Loaded Columns

Chairman, R. Bjorhovde, The University of Alberta

Pretensioning of Single-Crossarm Stayed Columns

M. C. Temple and H. H. Hafez, University of Windsor

Collapse of Space Trusses With Post-Buckling Unloading of Struts

C. Marsh, Concordia University

Task Group 7 - Tapered Members

Chairman, A. Amirikian, Amirikian Engineering Company

Design of Tapered Columns with Unequal Flanges

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

Design of Tapered Member Gable Frames

C. J. Miller, Case Western Reserve University
T. G. Moll, Jr., Fluidyne, Inc.

9:45 - BREAK

Task Group 15 - Laterally Unsupported Beams

Chairman, T. V. Galambos, Washington University

Elastic Analysis and Design of Biaxially Loaded I-Section Beams

H. Yektai and Z. Razzaq, Southern Illinois University at Carbondale

Basic Tests of Lateral Buckling of Beams

Y. Fukumoto, M. Kubo and Y. Ito, Nagoya University
(presented by T. V. Galambos)

Task Group 16 - Plate Girders

Chairman, F. D. Sears, Federal Highway Administration

Effect of Flange Thickness on Web Capacity Under Direct In-Plane Loading

M. Elgaaly, Bechtel Associates Professional Corporation

Interaction Between Shear Lag and Buckling in Plates at Collapse

P. J. Dowling, Imperial College of Science and Technology

Task Reporter 11 - Stability of Aluminum Structural Members

J. W. Clark, Aluminum Company of America

Inelastic Torsional-Flexural Buckling of Aluminum Sections

T. Pekoz, Cornell University

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

12 Noon - ADJOURN

TASK GROUP REPORTS

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

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

Optimization of Thin-Walled T-Shape Struts

C. Marsh, Concordia University

Introduction. Many bolted lattice structures, such as transmission towers, for which the T-section were first proposed by Carpena in 1970 in Italy, and some space trusses in which T-sections are now being used, often demand simple open sections to facilitate fabrication and assembly.

This study is concerned with the interior diagonals of trusses. These members are not continuous and are usually pin ended, with slenderness ratios above 100, i.e., long with light loads.

The most popular section is the angle, either single or double; it does not, however, make an efficient strut, as the spreading of the material to increase the moment of inertia is limited by torsional buckling. A solution is proposed in which T-section struts are loaded eccentrically.

T-sections and double angle struts are similar in behaviour. In aluminum structures, the T-shape can be extruded; in light gauge structures, formed double angles would be used. This study deals first with double angles to facilitate comparisons.

Strut Behaviour

Single Angles. A single angle bolted through one leg becomes an eccentrically loaded strut which fails in combined torsion and flexure about both axes (Fig. 1). Any attempt to optimise it by making the moments of inertia about both axes equal is thwarted by the interaction of torsion and flexure about the U axis. The equal 90^{o} angle is the most efficient.

To improve efficiency, it is most effective to reduce the buckling length by sub-struts, cross bracing, or in a space truss, double cross bracing.

Double Angles. A greater efficiency can be obtained from axially loaded double angles (Fig. 2) which fail either in flexure about the Y axis, or in combined torsion and flexure about the X axis. This combined failure means that the equal inertia section is not the most efficient. Optimum design requires:

$$(L/r_y)^2 = (L/r_x)^2 + k(5b/t)^2$$

Double equal angles are the most efficient because failure is in torsion (there is no local buckling in axially loaded angles) the addition of lips actually reduces the efficiency.

Eccentrically Loaded Double Angle. The critical load for the combined torsional/flexural buckling of a double angle strut loaded eccentrically about the Y axis is given by the solution of:

$$(P_y-P)\left[\frac{I_o}{A}P_t - P\left(e\beta + \frac{I_o}{A}\right)\right] - P^2(x-e)^2 = 0$$

where (x-e) is the distance from the load to the shear centre.

If the load coincides with the shear centre, i.e. (x-e) = 0, the three modes of buckling (torsion and flexure about the two axes) are independent, and moreover, the load for torsional buckling is so greatly increased that the thinness of the section is no longer controlled by torsional buckling, but by local buckling of the legs in compression.

The optimum proportions will be obtained when the load to cause buckling about the x axis equals that to cause local buckling of the flanges in compression due to bending about the Y axis (Fig. 3):

$$\sigma_c = \sigma_{ey}\left[1 + \frac{(x/r_y)^2}{(1-{}_{ex}/{}_{ey})}\right]$$

σ_c = local buckling stress for legs in compression

σ_{ex}, σ_{ey} = Euler stresses for buckling about X and Y axes

x = distance from shear centre to centroid

r_y = radius of gyration about Y axis

For a simple flange, the buckling stress may be taken as:

$$\sigma_c = \pi^2 E/(3a/t)^2$$

This limiting stress makes the eccentrically loaded double angle no more efficient than the axially loaded section, but if lips are added, the moment of inertia is increased and the local buckling stress becomes

$$\sigma_c = \pi^2 E/(1.5a/t)^2$$

giving significant increase in efficiency. An extruded T-section is superior to the double angle because of its single thickness stem.

Comparison of Efficiencies. As each combination of load and length gives a different optimum shape, in practice a choice must be made of the range over which a given set of proportions will be used. Typically, an optimum double angle strut having a slenderness ratio of 100 is compared with the other sections discussed (Fig. 4).

Ad hoc tests on a specific T-section, used on a project, confirmed the validity of the analysis. Deflection is large at failure, but not observable in the working range. A direct comparison between two comparable space trusses showed that double cross-braced single angles, at that time the lightest system, weighed some 50% more than the system of T-sections.

TASK GROUP REPORTS

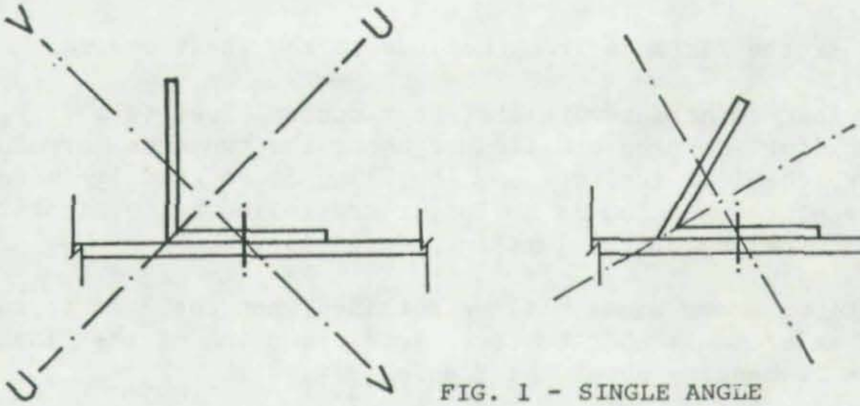


FIG. 1 - SINGLE ANGLE

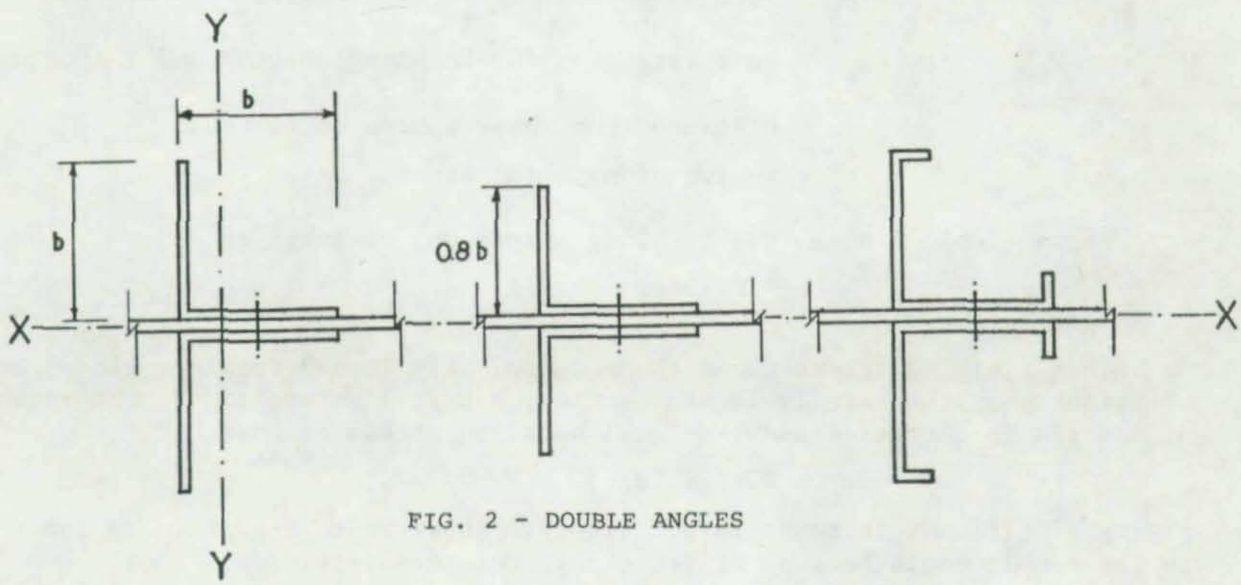


FIG. 2 - DOUBLE ANGLES

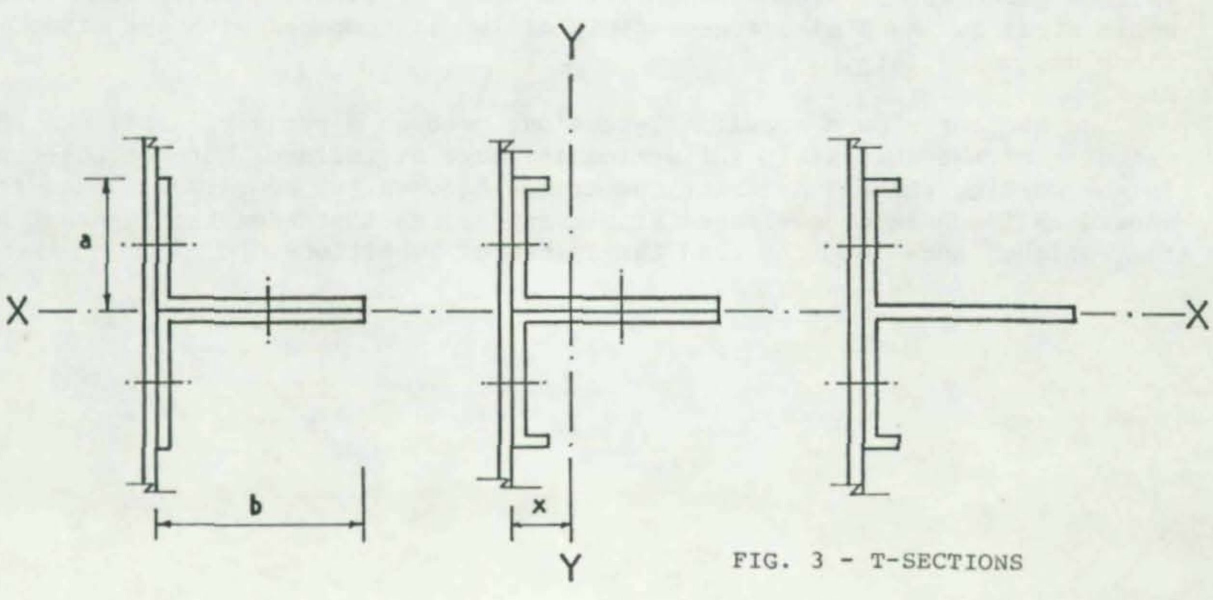
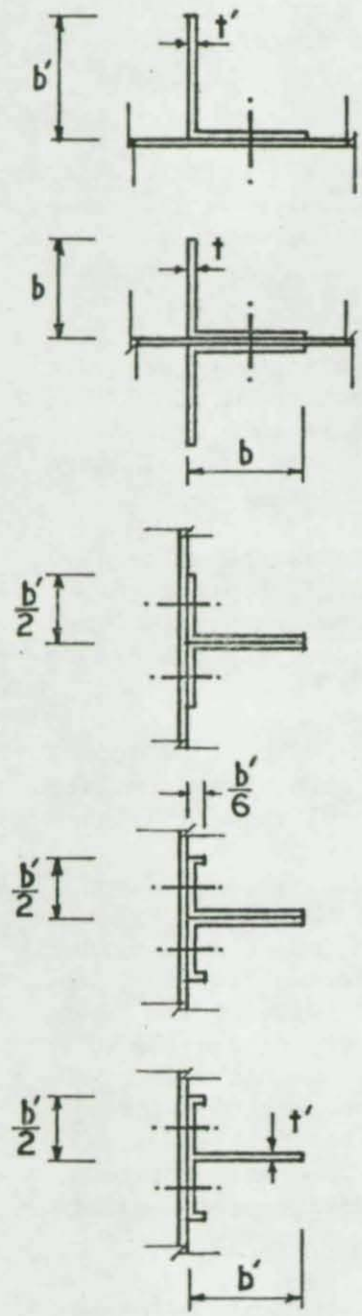


FIG. 3 - T-SECTIONS



$(L/r)_{eff}$	b'/b	b'/t'	A'/A
115	1.6	16	1.3
100	1.0	17.5	1.0
102	1.9	45	1.04
82	1.75	67	0.67
73	1.65	52	0.53

FIG. 4 - EFFICIENCY OF STRUTS
CONSTANT LOAD AND LENGTH

TASK GROUP REPORTS

Stability Considerations in the Design of Cold-Formed Steel Storage Racks

T. Peköz, Cornell University

Rack structures are important applications of cold-formed steel. Cold-formed steel members are used in a large majority of rack structures. This is due to the versatility and economic feasibility of cold-formed steel through optimization in such applications.

Rack structures are quite different from usual building structures in several ways. Their design presents many interesting and challenging problems. For the past eight years the author has been conducting research on the behavior of racks. Based on these studies an industry specification was formulated in 1972. Since that date the continuing work has been aimed at refining, and when justified, liberalizing the 1972 specifications.

Racks are highly indeterminate structures with complex modes of behavior. This behavior necessitates the consideration of the interaction of local and overall behavior. There are several types and kinds of racks each with its own peculiarities. The work reported on in this paper included several large and small scale tests on a variety of racks and rack components.

The overall stability and the behavior of rack structures are strongly influenced by the semi-rigid nature of the joints. Semi-rigid connections exist between the columns and the beams as well as at the column bases. The research included experimental and analytical studies of assessing the properties of the joints and reflecting them in the design.

The behavior of the posts with a wide variety of types of perforations was another challenging topic studied. Design provisions were also formulated to reflect their behavior. These provisions include flexural as well as torsional-flexural buckling.

Since most of the columns used in rack structures are thin-walled singly-symmetric open sections, torsional-flexural behavior is an important consideration. The complexity at the torsional-flexural behavior arises from the following two facts. First, the posts are restrained against twisting and bending about the two principal axes at discrete points with varying degrees of effectiveness. The second reason for the complex behavior is that the axial loads are in general applied with an eccentricity about the axis of symmetry of the posts. The application of the classical theory of torsional-flexural behavior does not give satisfactory results primarily due to cross-sectional distortions under this type of loading. In the course of the research a satisfactory approach was developed.

All the approaches developed had to be simplified for general design use and inclusion in the forthcoming specifications in the United States.

TASK GROUP REPORTS

Current Research on Cold-Formed Steel Beam Webs

W. W. Yu, N. Hetrakul, N. Phung, University of Missouri-Rolla and R. A. LaBoube, Iowa State University

Thin-walled, cold-formed steel structural members have gained increasing use in building construction and other types of structures during the recent years. Because of the use of new configurations in conjunction with high strength steel sheet and strip, the design methods used for such members may be beyond the scope of the present specification. For this reason, an investigation of cold-formed steel beam webs has been conducted at the University of Missouri-Rolla under the sponsorship of the American Iron and Steel Institute. This paper deals with the research work on the structural behavior of beam webs subjected to bending, shear, combined bending and shear, web crippling, combined web crippling and bending. In addition it discusses the findings on the beam webs reinforced by transverse or longitudinal stiffeners.

TASK GROUP 18 - UNSTIFFENED TUBULAR MEMBERS

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

The Axial Strength of Tubular Columns Under Hydrostatic Loading

W. F. Chen and S. Toma, Purdue University

Fixed offshore structures are being built in water depths to 500 feet. The concept of guyed tower for 2000 feet of water is currently being considered as deepwater platforms. Tubular members are the most important components in these offshore structures. Among problems associated with prediction of tubular member behavior are the effects of two-dimensional residual stresses in members introduced during fabrication, the unknown importance of initial imperfections in fabrication, and the interaction of axial and bending stresses with compressive hoop stresses caused by the external hydrostatic pressure. The work described herein attempts to investigate theoretically the axial load carrying capacity of fabricated tubular steel columns when subjected to external hydrostatic pressure.

The effect of hydrostatic pressure on the axial load-carrying capacity of tubular columns is investigated from the standpoint of beam-column theory. Shell theory related to local buckling is not considered. The influence of external hydrostatic pressure on the AISC-CRC column strength curve has been determined, using the computer model developed. Comparisons of the computer solutions have been made with the results of 10 tests on actual columns conducted recently at Lehigh University, providing final confirmation of the validity of the computer model. The computer model is found accurate for predicting the behavior and strength of fabricated tubular steel columns and can be used to generate design information from which design criteria and recommendations can be derived and developed.

TASK GROUP REPORTS

TASK GROUP 12 - MECHANICAL PROPERTIES OF STEEL IN INELASTIC RANGE

Chairman, R. B. Testa, Columbia University

Weld Shrinkage Stress Patterns

J. B. Dwight and J. D. White, Cambridge University

A vital ingredient in any realistic study of buckling (columns, beams, plates) is the pattern of residual stress in the member concerned, since this controls the premature onset of yielding as loading proceeds. The proposed paper will summarize recent studies at Cambridge, England, of residual stresses in welded members fabricated from plate. The results will be of interest to some of the other task groups.

The main object of the work has been to evolve simple formulae for estimating the longitudinal "tendon" forces locked into welds. The resulting stress field in the rest of a member can then be obtained by simple statics. It is well known that the material around a weld plus the weld itself carry yield tension, but reliable rules for determining the extent of this zone (and hence the tendon force) have not been available. Such information is now provided.

The basis of the study is a finite difference program which gives the build-up of stress in a plate while a weld is being laid. Results from this have indicated reasonable simplifying assumptions that can be made, leading to a greatly simplified theory. This has proved a powerful tool and has enabled a parametric study to be conducted covering various kinds of weld -- bead-on-plate, butt, multi-pass, T-fillet. It has also been possible to study other factors, including: preheat, existing stress-field, asymmetry of cross section. The final result is a series of simple formulae, which could be used in a design office.

The theoretical work has been partly validated by test results. These have also covered flame cutting, and the effect of slip in T-joints.

TASK GROUP 4 - FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

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

Elastic Buckling Behavior of Unbraced Single-Story Single-Bay Orthogonal Space Frames

Z. Razzaq, Southern Illinois University at Carbondale and M. Naim, Arizona State University

An analytical study of the elastic buckling behavior of perfect space frames has been completed. The systematic formulation of a matrix stiffness equation given by Livesley has been extended to include Renton's stability functions condensed by Chu and Rampetsreiter. The numerical study presented includes the buckling loads and the corresponding deflected shapes for six different types of space frames subjected to equal as well as unequal loads applied at column tops. The influence of the relative magnitudes of the axial load-multiplication factors and the orientation of the principal axes of the column cross-sections upon the behavior of the frames was studied. The frame buckling loads were also compared to those obtained by the effective-length procedure. Based on the results presented, several interesting conclusions have been drawn regarding the behavior of space frames as well as the validity of the effective-length approach.

Computer Bifurcation Analysis of Cable Stayed Rigid Frames

C. K. Wang,and J. K. Stiller, University of Wisconsin, Madison

Smith, McCaffrey, and Ellis in January of 1975 published their paper "Buckling of a Single-Crossarm Stayed Column," in the ASCE Structural Journal in which a closed solution was made using force and deformation relationships. Tang in September of 1976 published his paper "Buckling of Cable-Stayed Girder Bridges," also in the ASCE Structural Journal, in which the critical load was estimated by the energy method. In the present study, a computer program was developed so that these and similar problems may be conveniently solved to any desired degree of accuracy.

The displacement method is used to establish the global stiffness matrix for which the stiffness coefficients of members in bending are modified according to the relative magnitude of the primary axial forces already existent in them. In addition, the second-order effect on the equilibrium of each member due to the separation of the primary axial forces in the buckled state is considered. These relative primary axial forces are raised gradually in proportion until the determinant of the global stiffness matrix is zero. Higher modes may be obtained by further increasing the relative primary axial forces.

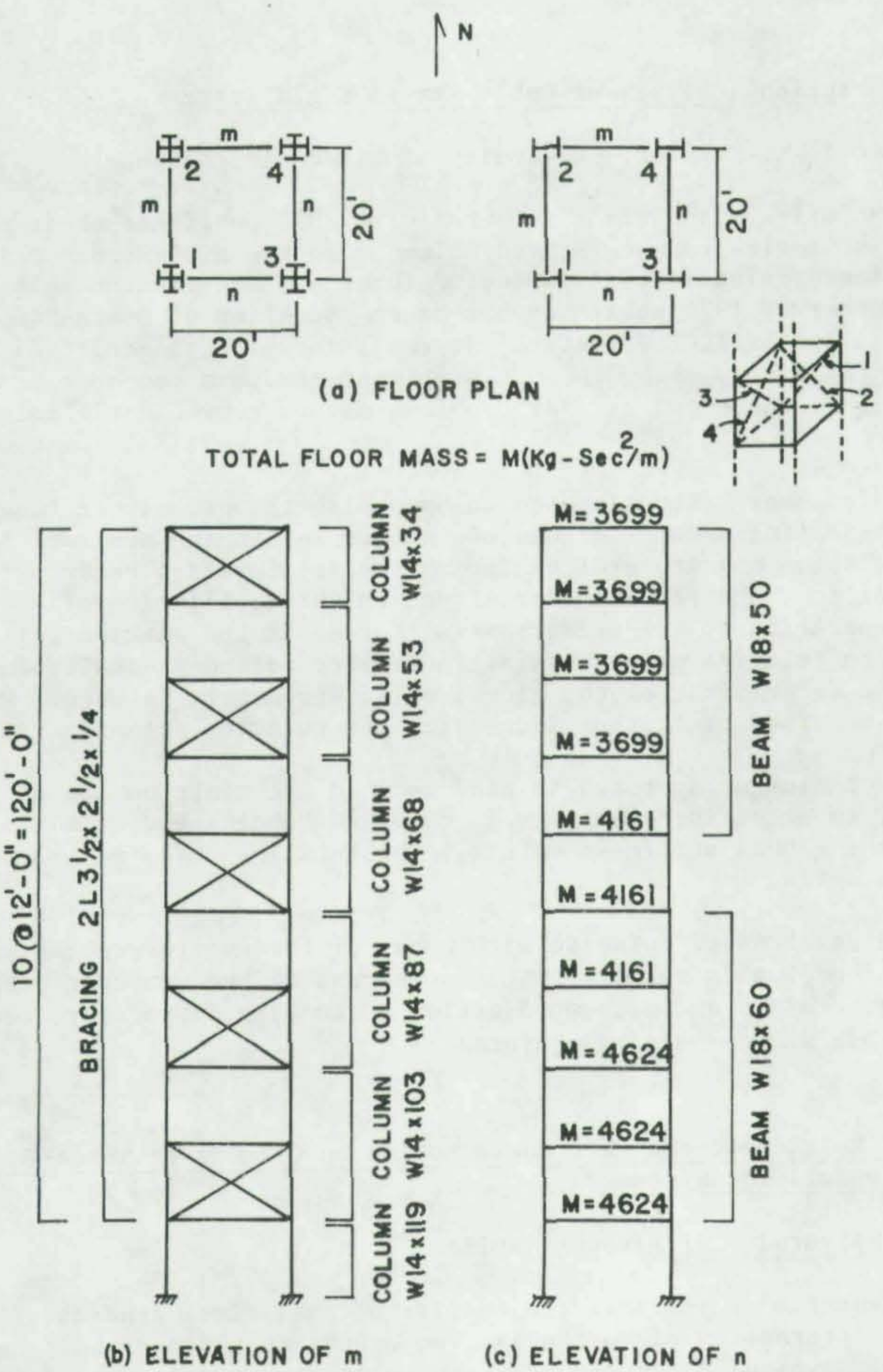
The direct element approach is used to feed the contribution of each element, whether in axial force only or in combined bending and axial forces, directly into the global stiffness matrix. The buckling modes may be obtained through a subroutine.

The results obtained coincide with those of the two references mentioned at the beginning of this summary. The usefulness of the computer program lies in its generality and easy application to complex, irregular, real structures with cable stays and rigid joints.

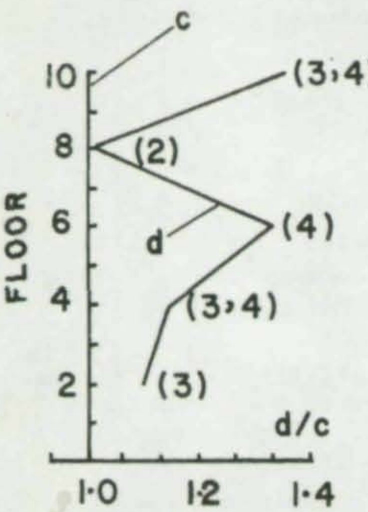
The Effect of Multicomponent Earthquake Motion on Columns of Three-Dimensional Steel-Concrete Building Systems

F. Y. Cheng, University of Missouri-Rolla

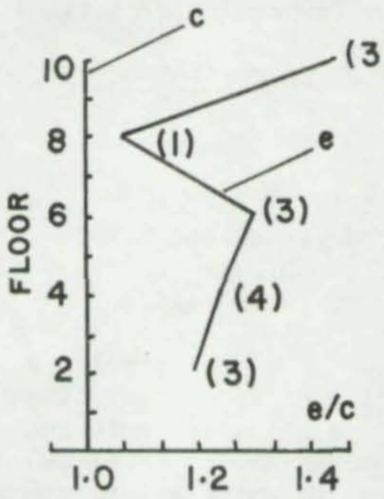
The presentation summarizes the results of analytical studies of the effect of the interaction of earthquake components on three-dimensional structural systems. Typical space structures varying from two to ten stories, having symmetric and unsymmetric structural planes or elevations, double or single symmetric columns, and with and without bracing members were selected to study the dynamic response to the El Centro, 1940, and Taft, 1952 earthquakes. It has been found that the interaction of three earthquake components significantly increases both the internal axial forces and moments and that the increase of some members is several times greater than that resulting from one component only. The increase becomes more significant for taller structures. Braced systems are more sensitive to the interaction of earthquake motions than unbraced systems. Typical results are shown in the accompanying figures.



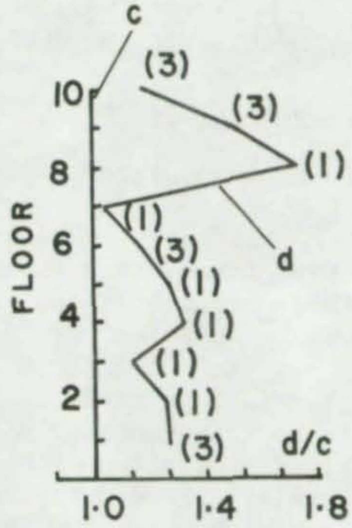
(d) TEN STORY STEEL BUILDING WITH R.C. SLABS AND STEEL BRACINGS FOR (I) DOUBLY SYMM. COLS. AND (II) SINGLY SYMM. COLS. EL CENTRO, 1940, (1 ft = 0.305 m)



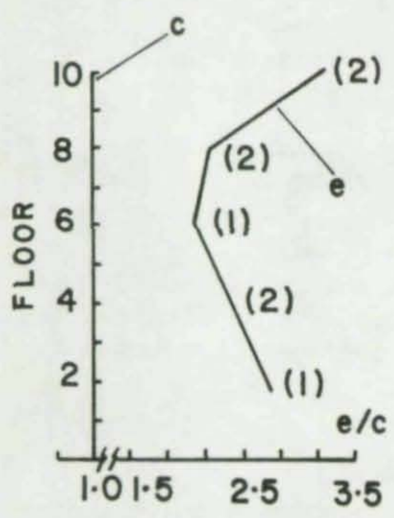
(e) MAX. RATIOS OF AXIAL FORCES, d/c, OF BRACINGS, ( ), c=N-s, P-Δ(DL): d-N-S, E-W, P-Δ(DL) FOR (I)



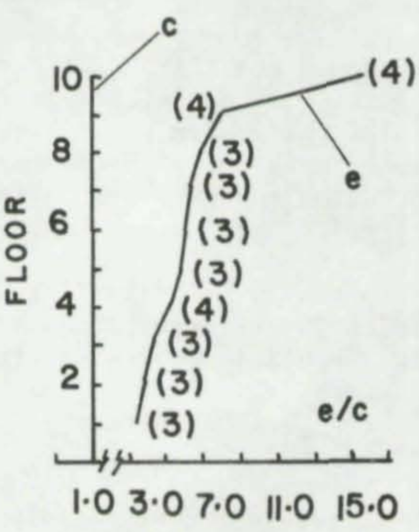
(f) MAX. RATIOS OF AXIAL FORCES, e/c, OF BRACINGS, ( ), c=N-S, P-Δ(DL); e=N-S, E-W, VE, P-Δ(DL+VE) FOR (I)



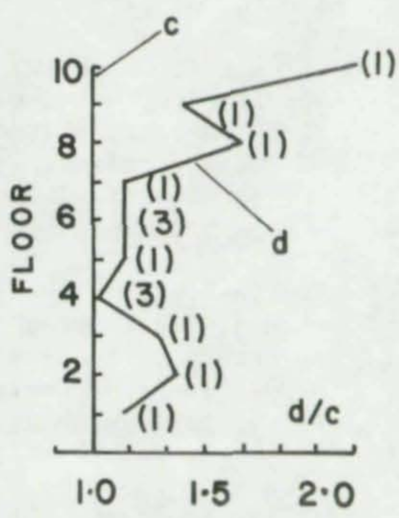
(g) MAX. RATIOS OF MOMENTS, d/c, ABOUT N-S PLANE AT TOP OF COLS ( ) OF (I)



(h) MAX. RATIOS OF AXIAL FORCES, e/c, OF BRACINGS ( ) OF (II)



(i) MAX. RATIOS OF AXIAL FORCES, e/c, OF COLS. ( ) OF (II)



(j) MAX. RATIOS OF MOMENTS, d/c, ABOUT N-s PLANE AT TOP OF COLS. ( ) OF (II)

TASK GROUP 8 - DYNAMIC STABILITY OF COMPRESSION ELEMENTS

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

Crossflow Induced Instability of Circular Tubes

S. S. Chen, Argonne National Laboratory

Many structural and mechanical components consist of long, slender circular tubes, such as heat exchanger tubes, offshore structures, and pipelines. Those structures are frequently subjected to fluid flow. Consequently, they may be subjected to flow-induced instability.

In this paper, a brief review of the problem will be presented. Topics to be discussed include examples of structural damages induced by fluid flow, fluid excitation mechanisms, analytical/experimental techniques, and design considerations. Particular emphasis will be placed on the flutter of tube banks subjected to liquid cross flow. A short film will be shown on the instability of a tube array of 25 tubes.

Dynamic Stability of Structures - A Review of Problems

D. Krajcinovic, University of Illinois at Chicago Circle

A somewhat ambitious task of this short review is to touch on some of the basic tenets of dynamic stability as applied to engineering structures and help in defining the objective of the Task Group.

When considering the dynamic stability of a structure it is necessary to keep in mind that the stability must be defined in relation to:

- a particular motion of the system,
- a particular parameter or group of parameters of the motion,
- a particular perturbation (or excitation), and
- a defined time interval.

Restricting ourselves to the dynamic stability of a column we can distinguish at least three sufficiently different classes of problems in function of the nature of the load. In a specific problem a structure can be subjected to:

- loads of periodic nature,
- rapidly changing ("blast" or "impulsive") loads, and
- nonconservative loads.

In addition, the stability of a structural element, such as a column, in fluid flow is a problem of increasing significance. As a result of a host of possible excitation sources (such as vortex-shedding, cavitation, turbulence or fluid/structure interaction) the dynamic stability of a column in a fluid flow presents in itself not one but a series of different problems.

Finally, the load might be of random nature which (in addition to the randomness of the material properties and geometrical imperfections) necessitates application of statistical methods of analysis.

The different nature of the above mentioned problems accounts for the fact that they are usually analyzed using almost entirely different methods. However, it is still possible, and certainly advisable, to establish common stability criteria in order to avoid possible confusion.

Dynamic Stability of a Simple Frame Subjected to a Circulatory Load

D. E. Panayotounakos and A. N. Kounadis, National Technical University of Athens

In the recent past, some technological demands have brought into focus the importance of stability of equilibrium of elastic systems subjected to (non-potential) nonconservative loads; circulatory (also called tangential or follower) loads are examples of nondissipative forces which are nonconservative. Structural systems under the application of circulatory loads exhibit two types of instability: divergence (Pflüger's column) and flutter (Beck's column). Divergence (static) buckling loads are evaluated by virtue of static methods, while flutter (dynamic) buckling loads can only be determined by employing the kinetic criterion.

In this investigation a variational methodology, based on the kinetic criterion, is developed for studying the dynamic stability of a simple rectangular two-bar frame with one end simply supported and the other hinged on an immovable support. The frame is subjected to a circulatory load applied at the joint which during the deformation remains tangent, at that point, to the center line of the bar with the immovable hinge. Recently, a nonlinear buckling analysis (1) of the foregoing nonconservative frame, based on a static approach, is presented.

One of the main purposes of this investigation is to compare the present findings derived on the basis of flutter instability with those of Ref. (1) based on nonlinear divergence instability.

References

(1) Kounadis, A. N., J. Giri and J. Simitses, "Nonlinear, Divergence Buckling of a Simple Frame Subject to a Follower Force," J. App. Mech., to appear.

Dynamic Buckling of Simple Frames Under a Step-Load

G. J. Simitses and J. Giri, Georgia Institute of Technology
A. N. Kounadis, National Technical University of Athens

The dynamic stability of simple two-bar frames subjected to a suddenly applied eccentric load of constant magnitude and infinite duration is investigated. The eccentric load is constant-directional and parallel to one of the bars which is immovably hinged. The other bar, which is of equal length and stiffness to the first bar, is supported by a pin with the following three variations: (a) immovable, (b) on rollers along a plane normal to this bar, and (c) on rollers along a plane parallel to this bar. A criterion for dynamic stability is presented, which is employed in estimating dynamic critical loads. The analysis is based on nonlinear kinematic relations and linear constitutive relations. The effects of the slenderness ratio of the frame bars and of the small load eccentricity upon the critical load are fully assessed. Among the most important conclusions, one may list the following: (a) Frames, which under static conditions exhibit limit instability, buckle dynamically provided that the eccentricity, $\bar{e}$, is algebraically smaller than a critical one, $\bar{e}_{cr}$ (slenderness ratio dependent). For eccentricities larger than the critical one there is no dynamic buckling. This observation is also true for static loading. (b) For all three frames and all values of the bar slenderness ratio, the dynamic critical load decreases as $\bar{e} - \bar{e}_{cr}$ increases. (c) The effect of the bar slenderness ratio upon the dynamic critical load is appreciable. (d) The discrepancy between theory and experimental results (limited in availability) is smaller than 1.5%.

TASK GROUP 20 - COMPOSITE MEMBERS

Chairman, S. H. Iyengar, Skidmore, Owings and Merrill

Effect of Cracking in Concrete Shear Walls of Composite Structures Under Lateral Loading

J. Springfield, C. D. Carruthers & Wallace, Ltd.

Stability considerations peculiar to mixed steel and concrete structural systems arise when lateral forces are to be resisted jointly by steel and concrete elements.

Investigation of the action of the types of reinforced concrete shear walls used in residential buildings has been concerned with the interaction of coupled walls, and then primarily with the resistance rather than the response.

In steel framed buildings employing concrete service core structures, it has been usual to rely entirely on the concrete core structures to provide lateral stability. In this case, the response to lateral loads is important primarily in the proper assessment of the second order effects, the P-Δ forces.

This presentation summarizes some of the problems encountered in designing a structure in which reinforced concrete core elements were to act in unison with rigidly framed steelwork to provide lateral stability (Fig. 1). Previously constructed foundations limited the size and reinforcement of the core elements. In designing for more stories than originally intended, mobilization of the steel framework was one method of supplementing the lateral resistance of the concrete core.

To properly analyze such a mixed system, the lateral response of both systems has to be known or at least bracketed in design. Little previous work seems to have considered either the effect of tensile flexural cracking in the concrete walls, which produces dramatic reduction in stiffness, or even the effect on stiffness of the horizontal construction joints present at each storey in all but slip formed core structures. In this case, a modified form of the ACI recommended effective moment of inertia was used. Other considerations relative to the concrete are the probable strength and modulus of elasticity for transient loads, at some future time rather than the minimum 28 day values.

Because of the marked effect of cracking, the primary response and second order effects under ultimate or factored loads is the necessary design criterion. The steel framework either must be proportioned to be elastic under factored lateral loading or the inelastic response must be determined.

The effects of lateral wind and earthquake appear to be different. While both depend on the natural frequency of lateral vibration, wind load is externally applied and accumulates from the top downwards, mobilizing all elements. Earthquake on the other hand is generated from the ground upwards: yielding of the lower stories tends to limit the transmission of inertia forces to stories above.

The analysis proceeded iteratively, succeeding analyses using better assessments of concrete wall stiffness, based on the previous cycle stress level (Fig. 2).

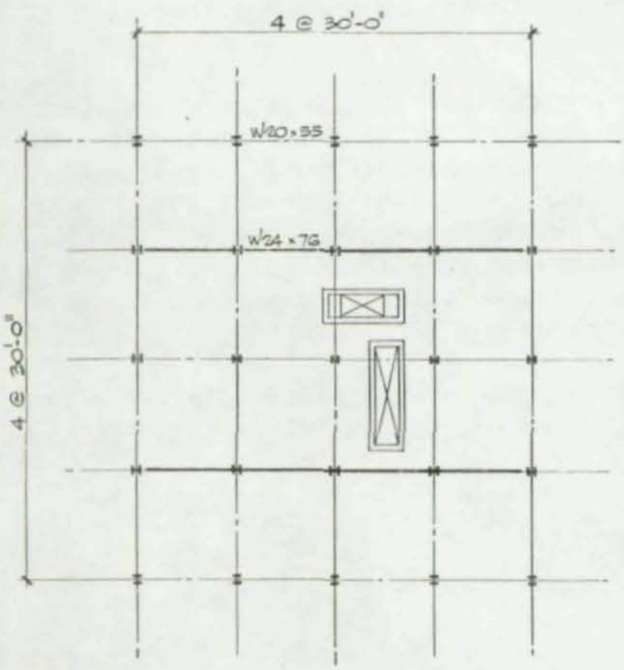


FIG. 1

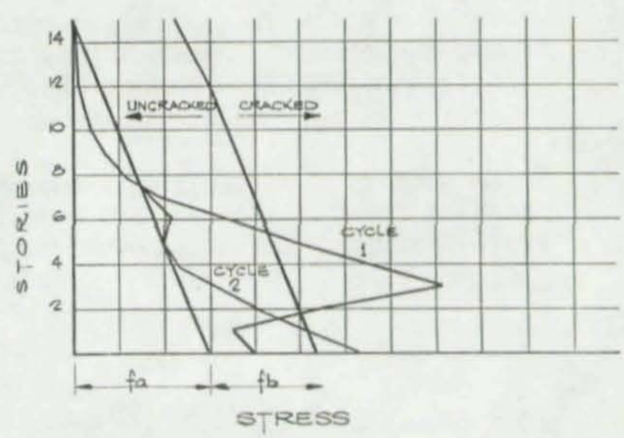


FIG. 2

TASK GROUP REPORTS

Joint Japan - United States Seminar on Composite Structures and Mixed Concrete Structural Systems - A Summary Report

R. W. Furlong, University of Texas at Austin

The seminar was sponsored by the U. S. National Science Foundation and its Japanese counterpart for the prupose of sharing new information and suggesting priorities for useful research in composite systems. Six participants from the United States and 8 from Japan were jointed by 3 experts from other countries and 10 observers from Japan in active and candid discussions of practices, problems and potential improvements.

All of the Japanese research includes considerations of earthquake resistant construction. In contrast, North American and European research rarely has included seismic applications in research directed primarily toward stiffness and strength assessment. When subjected to cyclic reversals of severe overload or displacement, appropriately constructed steel-concrete composite elements display desirable initial stiffness, strength, ductility, and energy dissipating hysteresis response not possible with structural steel or reinforced concrete acting separately. The essence of present research is directed toward definitions of appropriate construction.

Field construction practice in Japan employs a degree of precise fabrication control that would be prohibitively expensive if not altogether impossible in North America. Nevertheless, modified applications or alternate assemblies appear feasible for North American practice. Precast composite spandrel, wall panel, and floor units were described.

Recent studies of flexural members or systems have involved negative moment continuity, torsion, fatigue at shear connectors, partial shear connection, and deck reinforced slabs under both static and dynamic loading. Flexural members with column and wall units have required connection studies and improved techniques for estimating strength, stiffness, and inelastic response to flexure, shear, and thrust. Applications of composite elements in buildings, bridges and industrial structures almost always presented uncertainties regarding fatigue and seismic response at joints and at the steel-concrete interface.

A summary report with recommendations for research priorities is to be prepared by the Conference organizers, Prof. Le Wu Lu, Lehigh University, and Prof. Ben Kato, Tokyo University.

TASK GROUP REPORTS

TASK GROUP 1 - CENTRALLY LOADED COLUMNS

Chairman, R. Bjorhovde, The University of Alberta

Pretensioning of Single-Crossarm Stayed Columns

M. C. Temple and H. H. Hafez, University of Windsor

In the past few years research on stayed columns has dealt only with the value of the maximum buckling load. A procedure based on the finite element method has been developed to predict this maximum buckling load (1).

In all of the previous studies it was assumed that a small amount of tension existed in the stays just prior to buckling, which results in a maximum buckling load. The initial pretension required to ensure that this small amount of tension existed at the instant of buckling had not been determined. Only a few experiments had been performed and the tests indicate that the critical load is significantly affected by the variation of the initial pretension in the stays. No relationship, however, was derived to predict analytically the influence of the pretension force on the buckling load of stayed columns.

In this paper the effect of pretension on the buckling load of single-crossarm stayed columns is presented. In addition, the minimum effective pretension, the optimum pretension, and the maximum possible pretension are defined and determined by a geometric study of the stayed column. These relationships are applied to numerical examples to demonstrate the influence of the stayed column parameters on each of these pretension forces.

Finally, experimental results for the buckling load of a single-crossarm stayed column were obtained by varying the initial pretension. A sketch of the stayed column is shown in Fig. 1. The outer diameter of the cold-drawn seamless steel tube used for the column and crossarms is 1.50 in. (38.1 mm), and the inner diameter is 1.00 in. (25.4mm). The stays were made from 1/8 in. (3.18 mm) diameter steel rods.

The relationship between experimental and theoretical results is good at lower values of initial pretension force as shown in Fig. 2. At higher initial pretensions, however, the experimental buckling load is about 80% of that predicted. Two typical load-deflection curves are shown in Fig. 3. The tension in the stays is also plotted in the same Fig.

Research on the pretensioning of stayed columns is continuing at the University of Windsor.

References

1. Temple, M. C., "Buckling of Stayed Columns," Journal of the Structural Division, ASCE, Vol. 103, No. ST4, Proc. Paper 12894, April 1977, pp. 839-851.

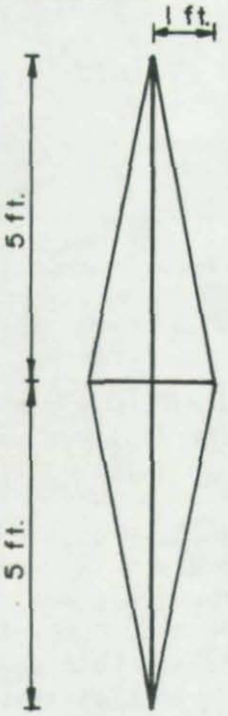


FIG. 1

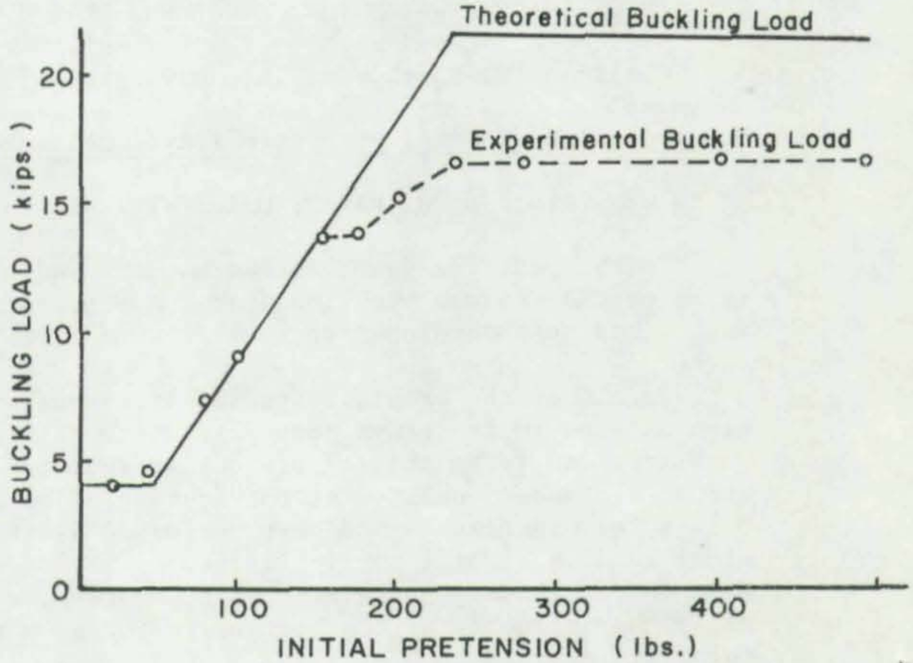
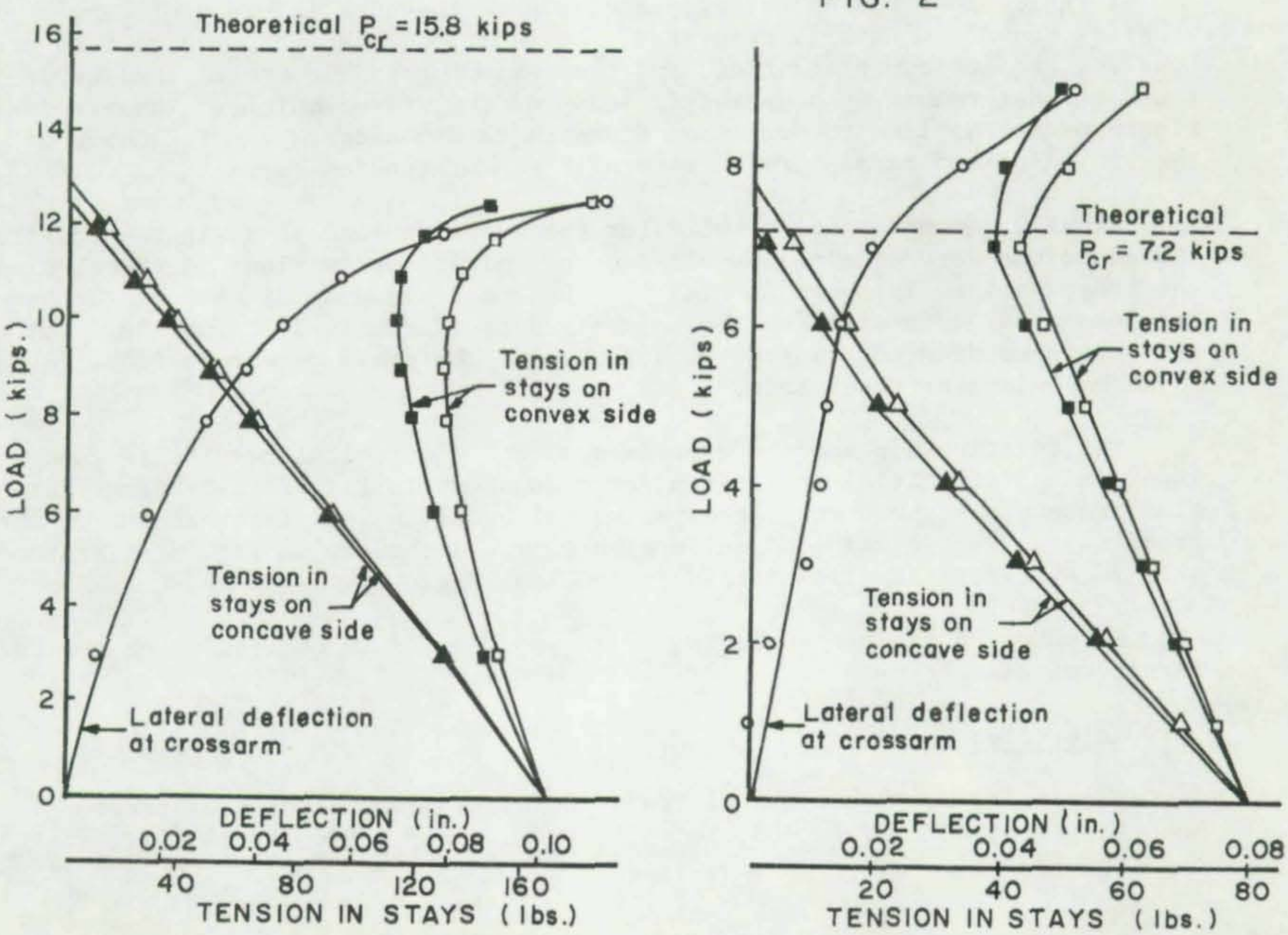


FIG. 2



(a) INITIAL PRETENSION OF 175 LB.

(b) INITIAL PRETENSION OF 80 LB.

FIG. 3

TASK GROUP REPORTS

Collapse of Space Trusses With Post-Buckling Unloading of Struts

C. Marsh, Concordia University

Introduction. Although the collapse analysis of trusses is usually applied to latticed space trusses, there are many types of triangulated three dimensional, highly redundant, trusses in which the first buckling of a strut need not lead to collapse. A number of research programs have been concerned with the post buckling behaviour and methods of collapse analysis for lattice structures have fallen into three groups (Fig. 1):

1) An assumption is made of elastic-plastic behaviour. The analysis involves the incrementing of the load to cause the next bar to buckle. The buckled bar is then removed and the next increment computed. This is continued until the structure is unstable, i.e. the matrix becomes singular. The total capacity is the sum of the increments. This is the most unconservative treatment.

2) In buckling, the strut is assumed to lose part or all of its capacity. The analysis finds successively the struts that buckle and replaces each with one of reduced capacity as a load, continuing until the total load begins to diminish. This is the most conservative treatment.

3) The correct load/shortening relationship for each strut is stored in the computer and analysis by an iterative procedure gives the exact theoretical solution. This is probably the most accurate method, but most expensive in programming and computer time.

This paper represents a compromise analysis.

Strut Behaviour. The theoretical load shortening curves for axially loaded struts, based on the assumption that first yield limits the capacity, are of the form in Fig. 2, for which the expression is:

$$(\varepsilon E/\sigma_y) = (\sigma/\sigma_y) + (\sigma_e/4\sigma_y)(\sigma_y/\sigma-1)^2(r/c)^2$$

Tests give more or less the same behaviour. In effect, struts vary in behaviour from brittle to plastic as the slenderness ratio increases.

These curves can be roughly approximated, in the immediately post buckling range, by the straight lines: (Fig. 3)

$$(\sigma/\sigma_y) = \left|(\sigma_y/\sigma_e) - (\varepsilon E/\sigma_y)\right| / \left|(\sigma_y/\sigma_e)^2 - 1\right|$$

This may be represented as a negative elastic modulus or more conveniently, by changing the area of the member to a negative area given by:

$$A' = -A/((\sigma_y/\sigma_e)^2 - 1)$$

Analysis now follows the first method in which the load is incremented to cause the next member to buckle. The area of this member is then replaced by the appropriate negative area, and the next increment computed. During the increments, the buckled member, by unloading, in effect, contributes to the applied loading. The analysis continues until the next increment is zero, i.e. when a displacement causes internal forces in exact internal balance, with no spare for external loads. The matrix never becomes singular.

The only extra information required by the computer program to provide this unloading feature is the formula for the negative area and the yield stress of the metal.

Because the computer program gives the actual shortening and load for each strut at failure, a direct check on the probable contribution to the strength of the structure can be obtained from a comparison with the known load/shortening behaviour.

The difference in predicted capacity between a plastic analysis and an unloading analysis depends of the sequence of buckling. Ideally, the slender struts buckle first, the stocky ones last, in which case the collapse load is typically 1.5 times the load to cause first buckle with the assumption of plasticity, and 1.4 if unloading occurs.

A space truss with heavy column-line chords and lighter interior chords is typical of such structures. The strain, at collapse, in the first strut to buckle is of the order of 3 times the elastic strain at buckling.

In the structure of Fig. 4, which is carried on radially sliding supports, the ring beams, acting in compression at the centre and tension at the perimeter, provide the primary structural action.

All the ring beams are of equal size. Because the most highly loaded ring beam is the shortest, contributing most to supporting the main trusses, when it buckles it unloads rapidly. The result is that, where in plastic analysis all the beams would contribute their maximum load at failure, when the unloading method is used the buckling of only two ring beams is required to precipitate collapse. In fact, the collapse load was only 1.05 times the elastic design load.

This structure is at present under contruction in northern Brazil.

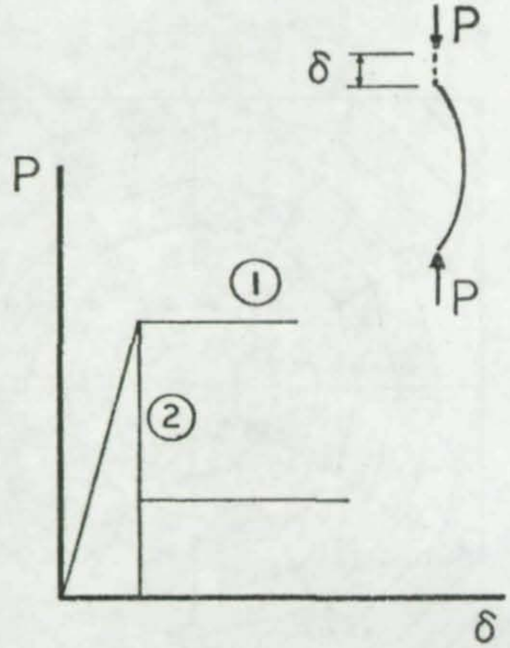


FIG. 1 - POST BUCKLING RELATIONSHIPS

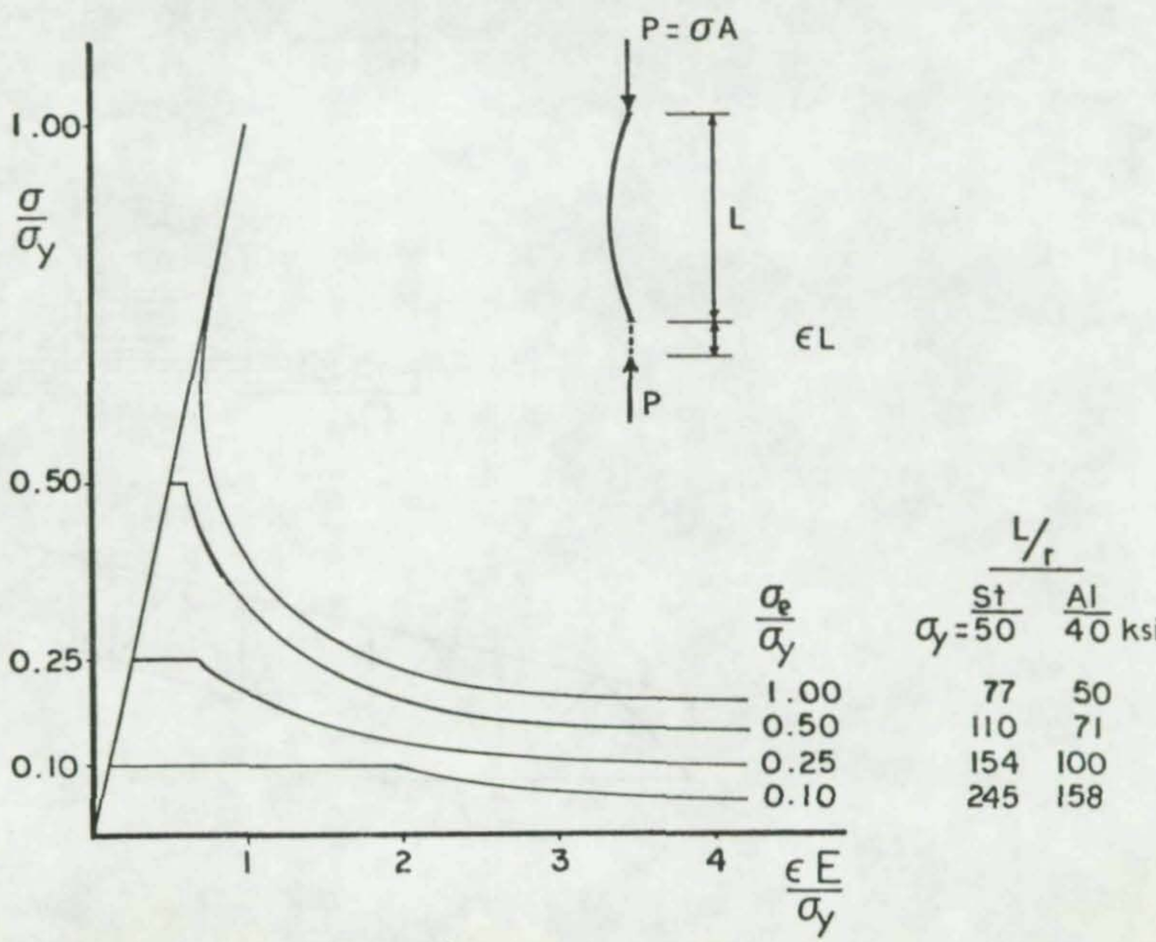


FIG. 2 - MEAN AXIAL STRESS VS EFFECTIVE STRAIN

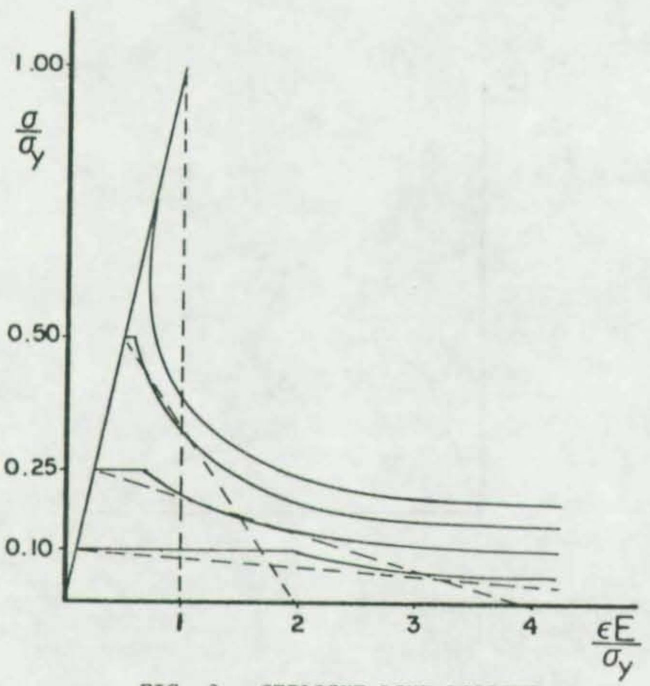


FIG. 3 - STRAIGHT-LINE LOADING

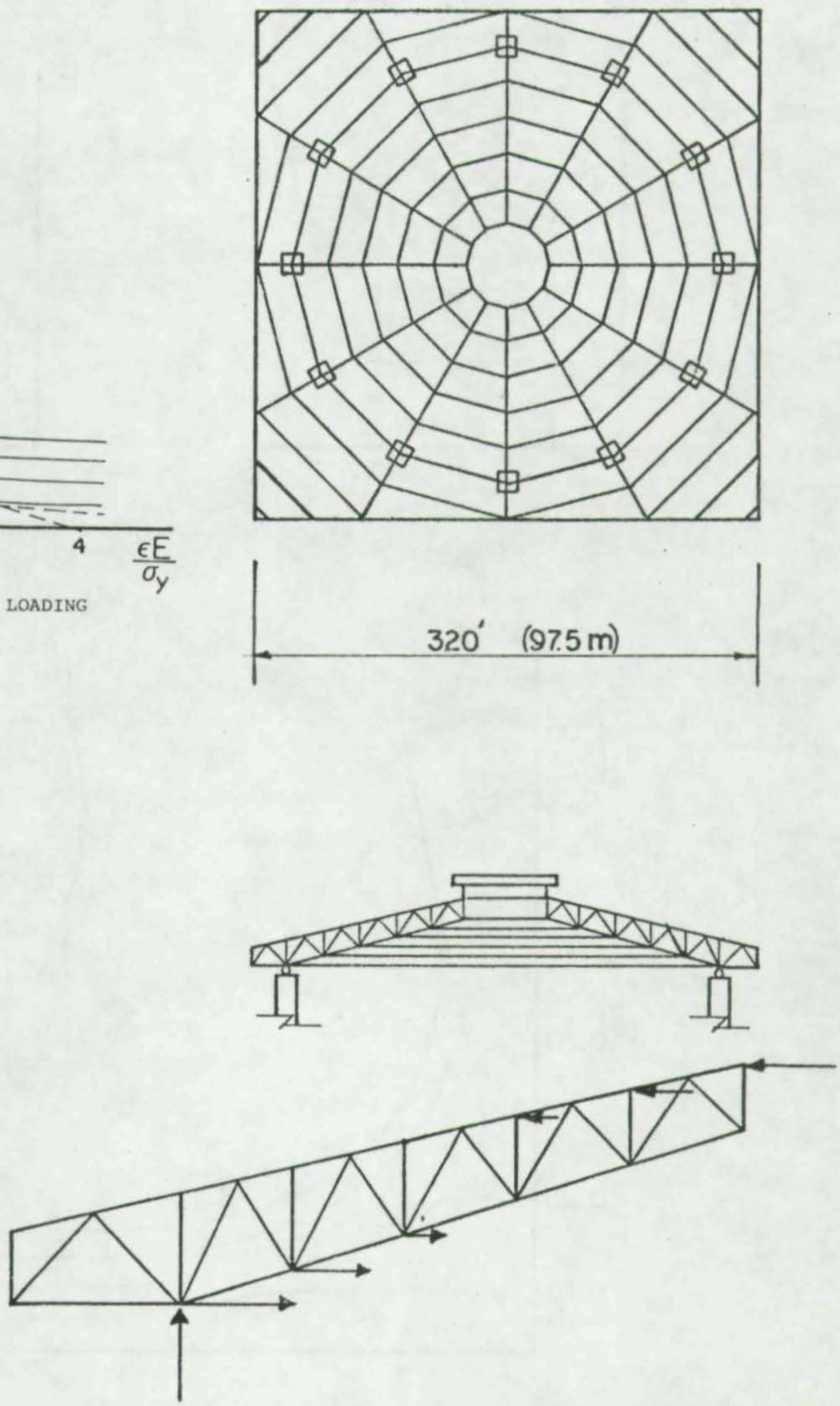


FIG. 4 - ARENA POOF

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 Columns with Unequal Flanges

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

The purpose of the study presented herein is to develop design information for the columns of single-span gable-type frames which are normally prefabricated from members with linearly tapered web. When such a column is subjected to both compression and bending at the top, the problem of lateral-torsional buckling of the unsupported compression flange may become severe. One approach commonly attempted in engineering practice is to use a larger compression (inner) flange area by increasing its width or both its width and thickness. Two supporting cases are considered in this study. The first case deals with a practical solution where the tension (outer) flange is braced laterally so that the column buckles torsionally about an enforced axis of rotation along the junction of the tension flange and the web. The second case covers the situation when no lateral support is provided to the tension flange so that the entire member fails by lateral torsional buckling. In all cases, both elastic and inelastic solutions are obtained. In obtaining the analytical results, the end conditions of the columns are assumed to be simply supported and the moment is applied at the deeper end of the column. All buckling solutions are obtained by using the finite element method. Ten prismatic elements for the column are found to provide sufficiently accurate answers.

Based on the analytical study, an interaction relationship of the form $\frac{f_a}{F_a} + C\frac{f_b}{F_b} = 1$ is developed in which the non-dimensional factor C is a function of the ratio of the unequal flanges areas, the tapering ratio of the column, the axial load, and the member length.

Design of Tapered Member Gabled Frames

C. J. Miller, Case Western Reserve University and T. G. Moll, Jr., Fluidyne, Inc.

A program for calculating member sizes to yield a minimum weight gabled frame with tapered members has been developed. The design produced satisfies in all respects the requirements of Supplement D to the 1969 AISC specification which governs the design of tapered members. The frame design is symmetrical about the vertical centerline, although neither loading nor support conditions need be symmetrical. Loading can be any combination of horizontal, vertical or normal loading on the rafter and horizontal loading on the column. The rafter can have no, one or two changes of taper within its length, at the designer's option. The matrix force method is used to do the analyses necessary in the design process.

The problem is solved by finding a design which minimizes frame weight subject to constraints imposed by the design specification, such as maximum stress and maximum width/thickness ratios, to minimum or maximum dimension constraints and to serviceability constraints, such as a limiting deflection. The minimization scheme used is a modified interior penalty function approach using the variable metric method of Davidon, Fletcher and Powell. The basic interior penalty function was modified so that an analysis did not have to be carried out every time a new design was made. This led to a significant reduction in running time without any effect on frame weights.

A number of example frame designs are given to show the versatility of the technique, as well as demonstrating the effect parameters such as purlin spacing, member lengths, rafter slope and member depth to width ratio have on resulting designs.

$$\text{MINIMIZE} \quad F(\vec{x}) = \text{frame weight}$$

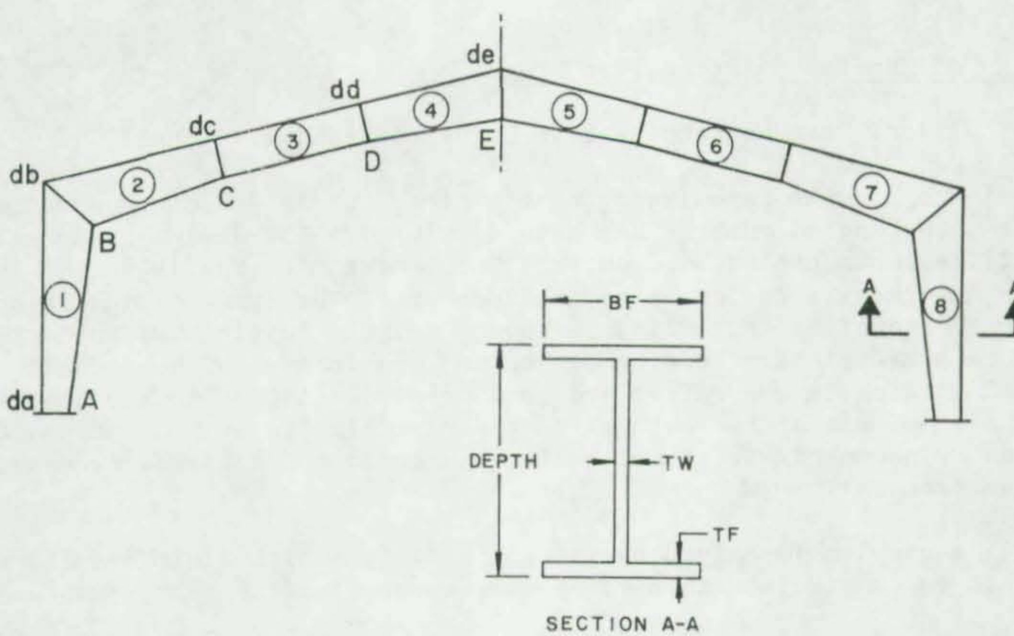
$$\text{SUBJECT TO} \quad G_j(\vec{x}) \leq 0 \;, \; j = 1, \ldots, m$$

$$\vec{x} = \text{vector of design variables}$$

$$G_j(\vec{x}) = j^{th} \text{ constraint equation}$$

$$m = \text{total number of constraints}$$

$$\text{MINIMIZE} \quad PF(\vec{x}) = F(\vec{x}) + r \cdot \sum_{j=1}^{m} \frac{1}{G_j(\vec{x})}$$



PROBLEM DEFINITION

TASK GROUP REPORTS

TASK GROUP 15 - LATERALLY UNSUPPORTED BEAMS

Chairman, T. V. Galambos, Washington University

Elastic Analysis and Design of Biaxially Loaded I-Section Beams

H. Yektai and Z. Razzaq, Southern Illinois University at Carbondale

This investigation is a theoretical study of biaxially loaded elastic beams with an I-shaped cross-section subjected to both equal and unequal end-moments. The three coupled differential equations of equilibrium governing the behavior of simply-supported beams subjected to nonuniform biaxial moments have been solved for the cases of free and restrained warping using the finite-difference as well as the energy methods. The total potential energy expression for the case of nonuniform biaxial moments has been derived and used in conjunction with Rayleigh-Ritz method for the analysis of the problem and to verify the numerical results obtained from the finite-difference method. For the case of the uniform biaxial moments, comparisons have also been made to the results found from the closed-form solution of the differential equations. Furthermore, interaction curves for the design of beams subjected to equal end-moments for the case of free warping have been presented and a procedure for design outlined together with a numerical example illustrating the use of these interaction curves. In conclusion, it has been found that the energy method is one of the most efficient approaches for the analysis and the development of moment-interaction relationships for biaxially loaded beams.

Basic Tests of Lateral Buckling of Beams

Y. Fukumoto, M. Kubo and Y. Ito, Nagoya University

This presentation summarized the results of an experimental investigation into the laterally unsupported rolled beams under a single concentrated load being applied at the compression flange of span center. End fixtures were specially made for this study to provide the laterally and torsionally simply supported conditions and a concentrated load was applied vertically at the compression flange through the Lehigh-type gravity load simulator.

Nominally identical twenty-five 7-m long members with 200x100x5.5x8 mm cross sectional dimensions have been prepared. From each 7-m member, beam specimens having three different span lengths of 2.6 m, 2 m and 1.5 m, a tensile coupon and a short beam for residual stress measurements have been cut out.

Included in the report were the statistical data on residual stresses in a beam-type cross section, the initial-out-of-straightness about the strong and weak axes and twisting, and the results of the buckling behavior of the nominally identical twenty-five beams with the specified slenderness parameter of $\overline{\lambda} = \sqrt{M_p/M_E} = 1.0$, 0.86 and 0.69, respectively. The experimental results obtained in this study were compared with available probabilistic models.

TASK GROUP REPORTS

TASK GROUP 16 - PLATE GIRDERS

Chairman, F. D. Sears, Federal Highway Administration

Effect of Flange Thickness on Web Capacity Under Direct In-Plane Loading

M. Elgaaly, Bechtel Associates Professional Corporation

The stability of the web of a plate girder loaded by in-plane discrete edge loading through the flange as affected by the flange thickness was examined. Results from finite element analysis demonstrate the effect of the flange rigidity on the stress distribution in the web as well as the web critical buckling load. Five girders were tested to determine their behavior up to failure. The web dimensions were kept identical in all five girders (aspect ratio = 1). The width of the flange was kept constant, however, its thickness varied from equal web thickness to 6.25 times web thickness. Strain rosettes were attached to the web (both faces front and back, nine rosettes to each face) and strain gages to the top and bottom surface of the flange (13 gages to the top surface and 6 gages to the bottom). Strain readings from the 73 gages were recorded at each load increment during testing and the readings from the rosettes were transferred to principal strains. The load was increased in increments up to failure. Failure loads and mode are given in the paper and the effect of the flange rigidity on the ultimate capacity is discussed. The requirements by the AISC specification to avoid web crippling under this type of loading are discussed in the light of the results from this study.

Interaction Between Shear Lag and Buckling in Plates at Collapse

P. J. Dowling and A. R. G. Lamas, Imperial College of Science and Technology

The paper examines the effect of the non-uniform compression associated with shear lag on the ultimate strength of a thin plate loaded in compression. Because use is made of the effective width concept to deal with both phenomena, treatment of plate elements which are liable to buckle under a shear lag type distribution of inplane stresses presents problems to designers. Can the non-uniform distribution of compressive stresses be neglected at collapse, and ultimate effective widths relating to uniformly compressed plates be utilised, or is there some interaction?

An exploratory numerical study was carried out on the box section loaded over the depth of each web at mid-span to simulate the effects of point loading (see Fig. 1). The webs were assumed to be made of high yield steel and so remain elastic in order to isolate the effects of plasticity in the flange. The numerical analysis modelled the box using dynamic relaxation to solve the finite difference equations describing the box behaviour. The behaviour of the webs and tension flange was described by the plane stress equations, while that of the compression flange was given by the large deflection equations of von Karman as modified by Marguerre to incorporate the effects of initial imperfections. A study was conducted in the normal way to establish a mesh size which gave results of acceptable accuracy. Some typical results are illustrated in Fig. 2. These relate to the effect of slenderness, b/t, on the load-end shortening curves for a flange of fixed initial imperfection and aspect ratio, b/ℓ. This latter parameter is, of course, the most important parameter influencing shear lag, while the slenderness parameter is the critical one for buckling.

Two failure criteria were seen to have an important influence on the results. For plates of small aspect ratio an upper bound to the load carrying capacity of the plate is often the shear capacity of the edges of the plate adjacent to the webs. In such cases plasticity does not spread across the entire width before the maximum load is attained. For flanges of larger aspect ratio the full plate width becomes plastic while portions of the plate edge removed from the centre remain elastic. These elastic areas provide a route for the shear stresses to mobilise the full inelastic capacity of the plate in compression under increasing load. Both of these failure modes are illustrated for a plate with $b/t = 60$, and $\omega_o = b/200$ in Fig. 3.

The results can be summarised in the form shown in Fig. 4 where the effects of shear lag on buckling strength of mild steel plates in compression are illustrated for plates of $b/t = 60$ and 100. It can be seen that in certain situations (i.e. for plates of large b/ℓ) the effect of shear lag is to reduce the collapse effective breadths to values below those associated with uniformly applied compression. In many practical situations, however, the aspect ratios of the panels concerned will be large enough to permit designers to ignore the effects of shear lag, and strength is conservatively predicted by expressions for buckling effective widths such as that proposed by Winter. The procedure of using the product of the shear lag effective breadth ratio and the buckling effective breadth ratio in design, as proposed by Maquoi and Massonnet, leads to over-conservative results. Nonetheless, it is advisable to place limits on plate geometries for which uniform applied compression can be assumed to occur. This may be of special practical importance for short span wide box construction as used to support decks of offshore steel jackets, or indeed may occur over the supports between points of contraflexure in bridge construction. The concept of a limiting b/ℓ for which uniform compression can be assumed has been used in the new draft British Steel Bridge Code.

Further theoretical and experimental work on this problem is in progress at Imperial College with a view to clarifying the situation in relation to stiffened flanges and wide flanged beams over continuous supports.

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2. Lamas, A. R. G., "Combined effect of shear lag and buckling on flange efficiency," Discussion, Final Report, 2nd Int. Coll. on Stability Steel Structures, Liege, 1977.

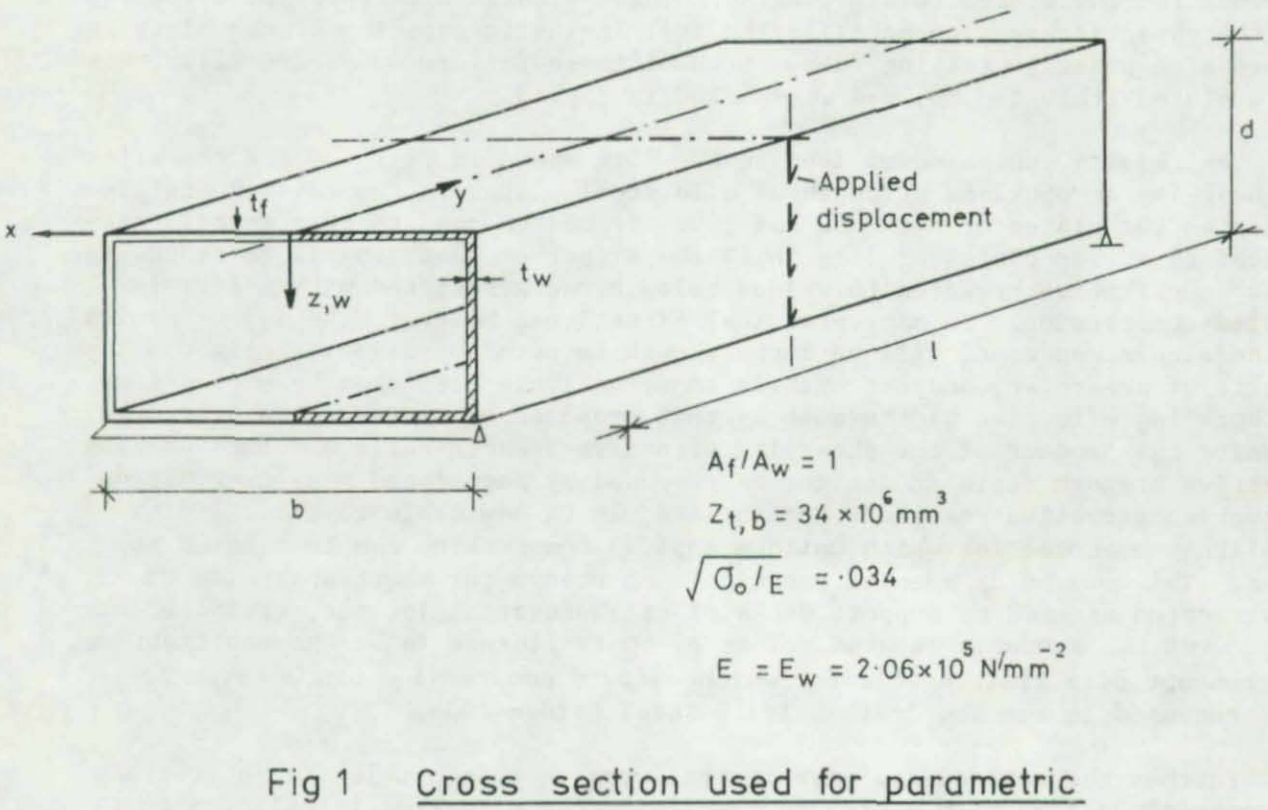


Fig 1 Cross section used for parametric study (webs remain elastic)

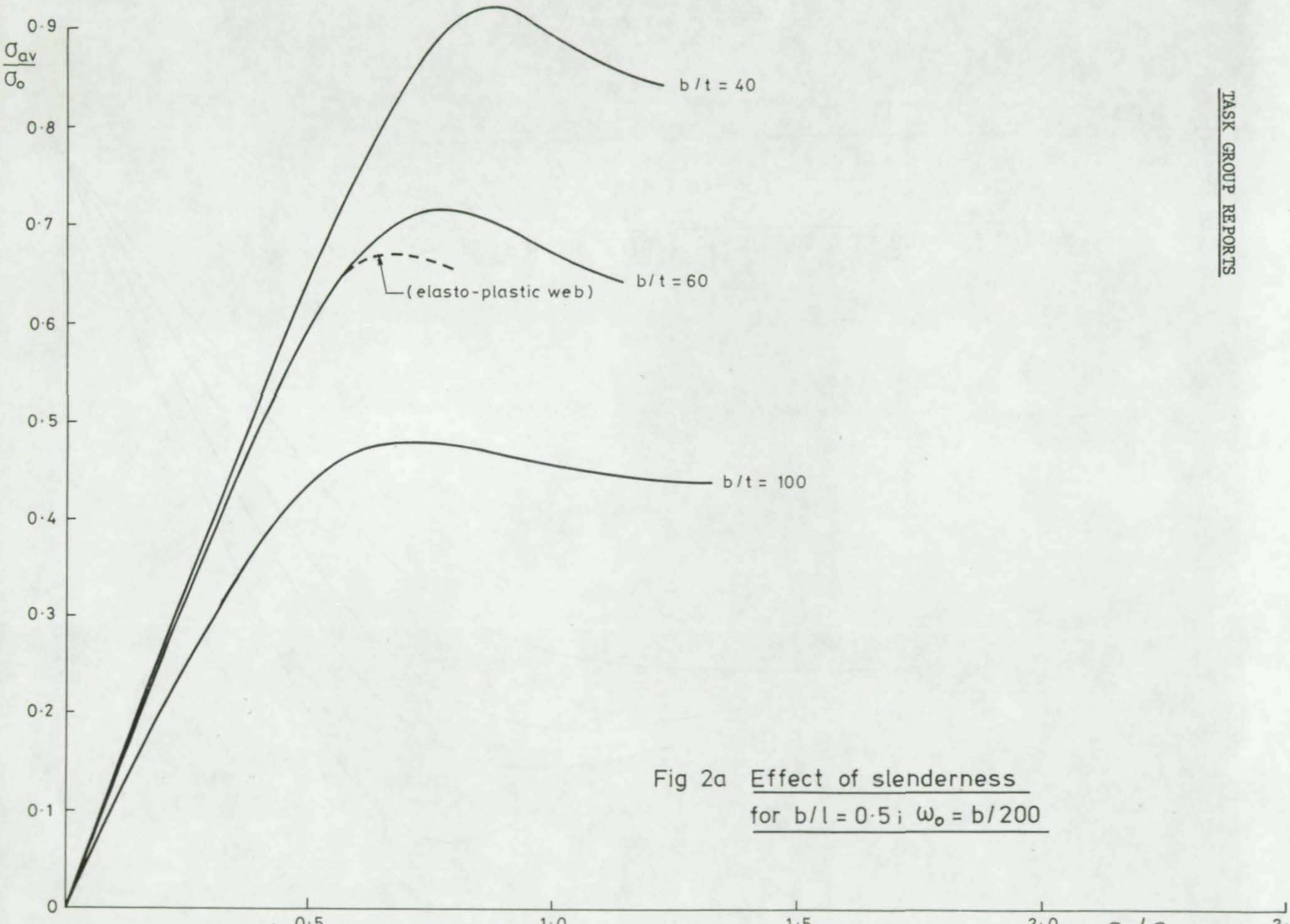


Fig 2a Effect of slenderness for $b/l = 0{\cdot}5$; $\omega_o = b/200$

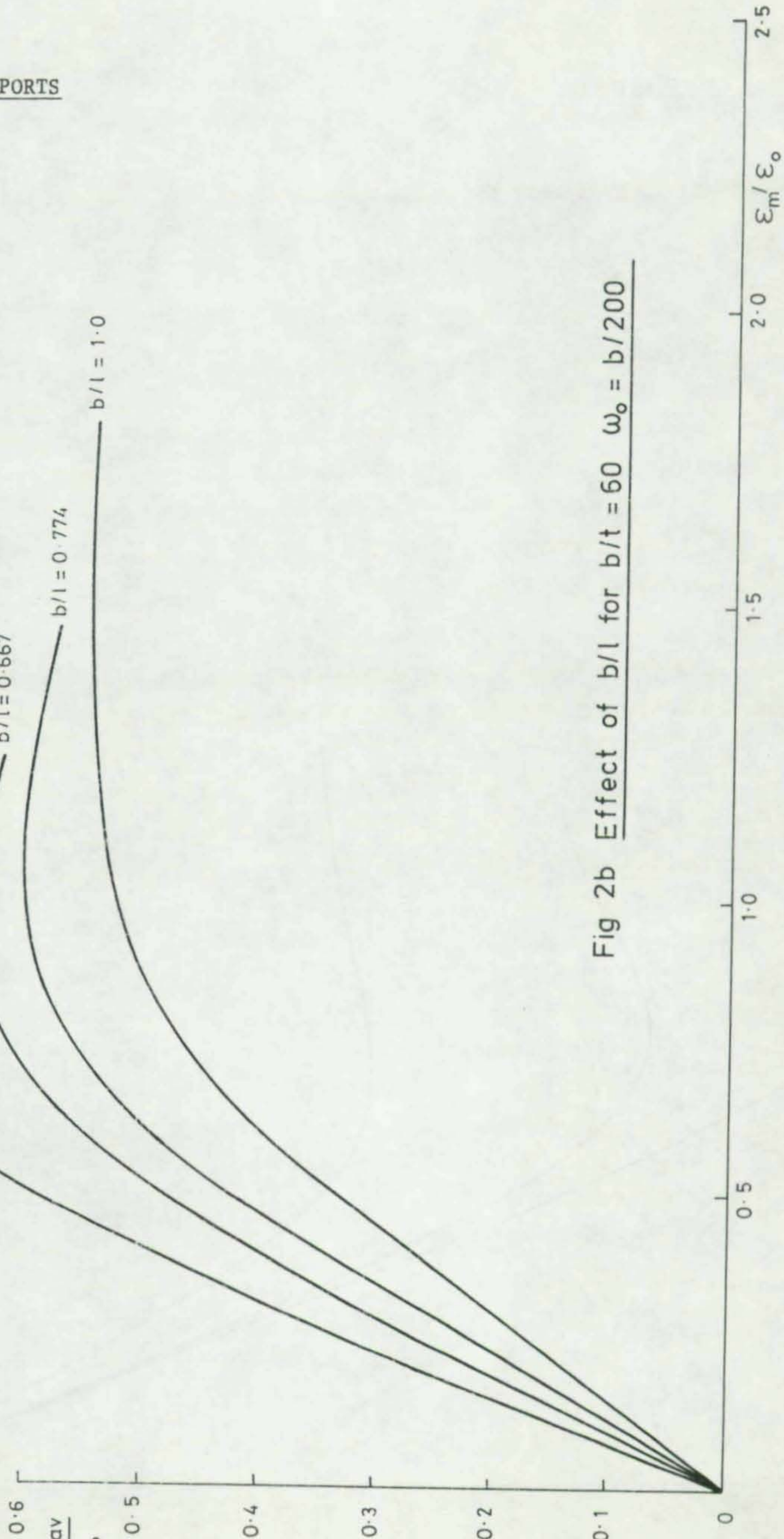
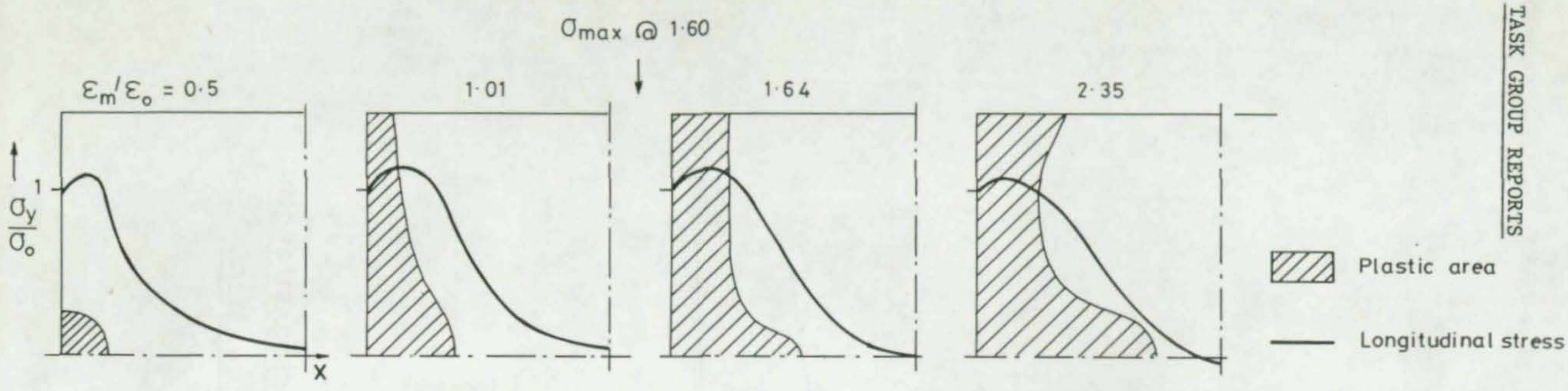
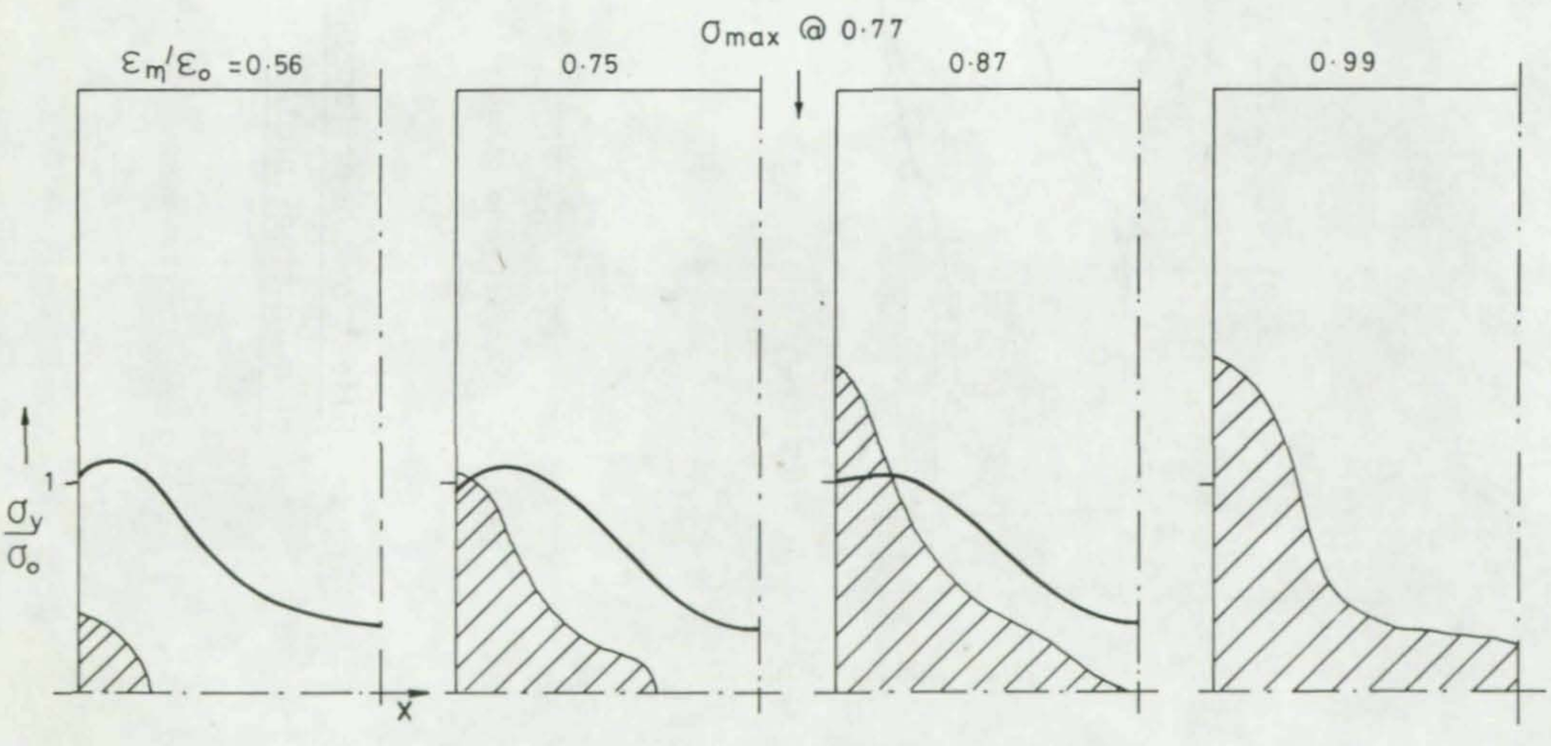


Fig 2b Effect of b/l for b/t = 60 ω_o = b/200

Fig 3 Spread of plasticity



CASE 1. SHORT PLATE $b/l = 1{\cdot}0$; $b/t = 60$; $\omega_o = b/200$



CASE 2. LONGER PLATE $b/l = 0{\cdot}5$; $b/t = 60$; $\omega_o = b/200$

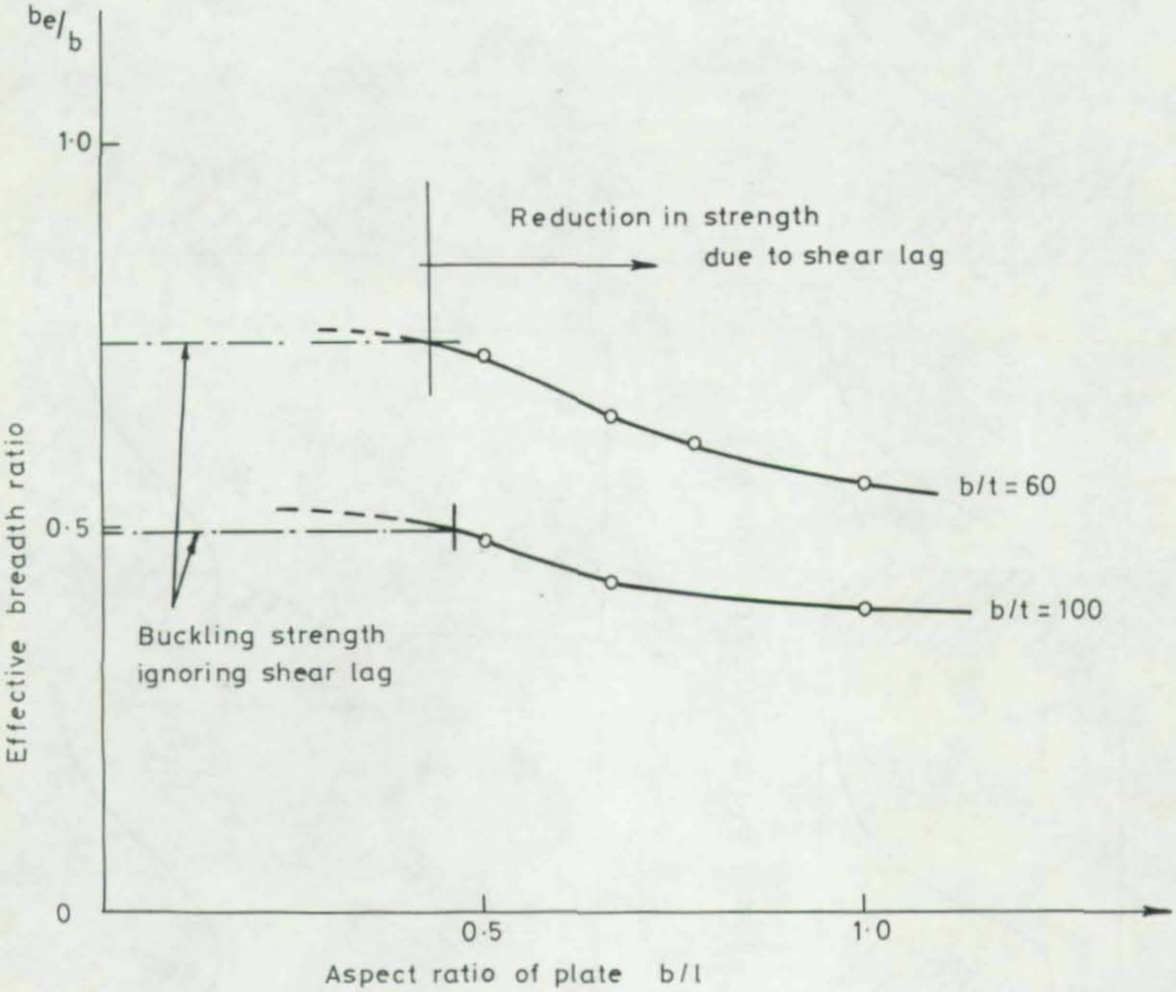


Fig 4 Effect of shear lag on buckling strength of mild steel plates in compression.

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

MIXED STEEL-CONCRETE SYSTEMS

MODERATOR: S. H. Iyengar, Skidmore, Owings and Merrill

PANELISTS: I. M. Hooper, Seelye Stevenson Value & Knecht
R. W. Furlong, University of Texas at Austin
W. J. Le Messurier, Le Messurier Associates/SCI

NOTE: Since Messrs. Hooper and Le Messurier spoke without a written text, only Furlong's full presentation is recorded herein.

Mixed Steel-Concrete Construction - S. H. Iyengar

Mixed steel-concrete construction basically, as implied here, consists of structural steel and concrete or reinforced concrete elements forming a total system which resists the forces imposed on the structure by their integral or composite action.

The degree of composite action depends on the particular combination or mixture of structural steel or concrete components.

In a true sense, all structures involve interaction between steel and concrete.

However, concrete has played a very secondary role in steel construction and most often, structural steel has played no role at all in concrete construction.

Some of the earliest applications of mixed forms were therefore isolated to just members, like composite beams and composite columns.

Composite beams involve composite action with the concrete floor slab and composite columns may take the form of concrete encased or concrete filled steel columns.

Over the last decade, composite systems have been viewed on a much broader scope as part of a systems concept.

This has entailed evolving systems which not only involve composite members, but also structural steel or concrete subsystems providing the necessary ingredients for strength and stability of the system. This inter-mixture of structural steel and reinforced concrete subsystems is what we would call mixed systems.

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I would like to present briefly some recognizable forms of mixed systems that have been in use in recent years.

In the formation of any of these systems, member stability and overall system stability are inherent considerations in terms of design.

I would like to present a few slides of some of these systems including some recent systems used in Japan.

Slides were presented for:

a. Composite Tubular Systems: Which involve a reinforced concrete exterior equivalent framed tube in combination with structural steel framing on the interior. The floor framing members are composite beams.

b. Concrete Core Braced Systems: Which involve cast-in-place concrete core shear walls with floor framing on the exterior made of structural steel.

c. Composite Claddings: Which involve steel plates or pre-cast concrete integral cladding.

d. Composite Frames.

e. Composite Connections.

f. Several Prefabricated Composite Systems used in Japan for apartment buildings.

Composite Framing Systems - I. M. Hooper

Mr. Hooper gave a brief description of work on composite frames particularly with respect to the role of the composite floor system. Also included were general observations on stability considerations in mixed systems from a designer's point of view.

A Recommendation For Composite Column Design Rules Consistent With Specifications Of The American Institute Of Steel Construction - R. W. Furlong

Introduction

Composite steel and concrete compression members can be employed in some applications more advantageously than can ordinary reinforced concrete or plain structural steel columns. The rapid field erection, great strength per unit area, and factory fabrication control of structural steel can be coordinated with the formability, stabilizing influence, low strength-cost ratio, and fire resistance of concrete through the use of composite columns. Since concrete is recognized as useful primarily for its compressive strength, design regulations for composite compression members have been relegated traditionally to the Building Code of the American Concrete Institute. Constraints that are appropriate for reinforced concrete design have led ACI to composite column design regulations that are not consistent with the design philosophy of the American Institute of Steel Construction when applied to columns in structures that are basically structural steel frames.

Design regulations for composite columns can be expressed in a form that is consistent with the AISC Specification. In a comparison with 180 reported test results, an earlier proposal for design rules was shown to give allowable loads 27 to 78 percent lower than those observed in tests. That proposal has been reviewed and revised in order to satisfy two fundamental considerations that seem to be necessary before any standard is acceptable in North America.

1. The allowable load on a composite column must be greater than the load allowed by AISC Specifications on the structural steel component of the column acting alone.

2. The allowable load on the reinforced concrete component of a composite column must be approximately the same as the load that would be permitted by ACI regulations on the reinforced concrete acting alone.

The compression behavior of concrete that is biaxially confined within the walls of steel tubing differs significantly from that of concrete which is placed as reinforced concrete outside structural steel shapes. The most significant difference in behavior occurs as the concrete resists compression strains in excess of 0.15 percent. The biaxially confined concrete cannot spall and fracture until the confining tube yields longitudinally, whereas outside a structural shape surface concrete can begin to fracture and spall at strains near 0.20 percent. Consequently it was considered necessary to employ separate design regulations for the 2 different types of composite member.

Proposed Design Specification for Composite Columns

Nomenclature (in addition to AISC 1973)

A_c = Actual area of effective concrete in composite design

A_{sr} = Area of longitudinal reinforcing steel for concrete providing composite action

h_1 = Concrete encasement thickness perpendicular to the plane of bending of an encased column

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h_2 = Concrete encasement thickness in the plane of bending of an encased column

E_m = Modified modulus of elasticity for composite section

F_{my} = Modified value of yield stress for composite section

F_{yr} = Yield stress of longitudinal reinforcement for composite concrete

r_m = Effective radius of gyration of a composite section

r_s = Radius of gyration of structural shape, pipe or tube in the assumed plane of bending or buckling

S_m = Modified section modulus of a composite section

S_s = Section modulus of structural shape, pipe or tube about assumed axis of bending

General Requirements

A composite column shall consist of structural steel rolled shapes, pipe or tubing acting with load bearing concrete together to resist compression and/or compression plus bending. In order to qualify as components of a composite column, individual structural steel shapes in the same cross section must be connected to one another with lacing, tie plates or batten plates adequate to satisfy Sections 1.18.2

Concrete encasement of structural steel shapes must be tied laterally and longitudinally with reinforcement spaced not more than $\frac{2}{3}$ the least dimension of the encasement exterior surface and containing both transverse and vertically a cross section area not less than 0.007 in^2 per inch of spacing between each bar. Concrete encasement of structural steel shapes must be thick enough to provide at least 1.5 inches of clear cover over lateral and longitudinal reinforcement.

The specified yield strength of structural steel in composite columns may be taken no greater than 55 ksi.

Concrete filled structural steel pipe or tube shall have a wall thickness t greater than

$$t \geq b \sqrt{\frac{F_y}{3Es}} \quad \text{for each face of width b}$$

or

$$t \geq h \sqrt{\frac{F_y}{8Es}} \quad \text{for circular sections of diameter h}$$

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Allowable Stresses

The allowable axial stress on the structural steel area of a composite cross section shall be determined with Eqs. 1.5-1 or 1.5-2 after a modified composite yield stress F_{my} is substituted for F_y, a modified composite modulus of elasticity E_m is substituted for E, and a radius of gyration r_m is substituted for r. The allowable axial force on the composite cross section shall be taken as the product of the area of structural shape A_s and F_a.

For concrete filled pipe or tube

$$F_{my} = F_y + F_{yr} \frac{A_{sr}}{A_s} + 0.85\ f'_c \frac{A_c}{A_s} \text{ with } F_{yr} \leq F_y \tag{A}$$

$$E_m = 29000 + 0.4\ E_c \frac{A_c}{A_s} \tag{B}$$

$$r_m = r_s$$

For concrete encased structural steel

$$F_{my} = F_y + 0.7\ F_{yr} \frac{A_{sr}}{A_s} + 0.6\ f'_c \frac{A_c}{A_s} \text{ with } F_{yr} \leq F_y \tag{C}$$

$$E_m = 29000 + 0.2\ E_c \frac{A_c}{A_s} \tag{D}$$

$$r_m = r_s \text{ but not less than } 0.3h_2$$

Combined Axial and Bending Force

An index of axial stress shall be taken as $f_a = \frac{P}{A_s}$.

An index of bending stress on composite sections shall be computed on the basis of a modified section modulus S_m.

$$S_m = S_s + \frac{1}{3} A_{sr}\ (h_2-5) + A_w \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7\ f'_c\ h_1} \right) \tag{E}$$

The value of A_w is zero for pipe or tubing.

Composite members subjected to both axial compression and bending stresses shall be proportioned to satisfy the expression

$$\left(\frac{f_a}{F_a}\right)^2 + \frac{C_{mx}}{\left(1-\frac{f_a}{F'_{ex}}\right)}\frac{f_{bx}}{F_{bx}} + \frac{C_{my}}{\left(1-\frac{f_a}{F'_{ey}}\right)}\frac{f_{by}}{F_{by}} = 1 \qquad \text{(F)}$$

when neither $\dfrac{C_{mx}}{\left(1-\frac{f_a}{F'_{ex}}\right)}$ or $\dfrac{C_{my}}{\left(1-\frac{f_a}{F'_{ey}}\right)}$ are taken to be less than 1.

and $$F'_e = \frac{12\,\Pi^2 E_m}{23\,(kl/r_m)^2} \qquad \text{(G)}$$

Values of F_{bx} and F_{by} shall be 0.75 F_y for tubes and 0.60 F_y for shapes.

Commentary on Proposed Specifications

General requirements and regulations for minimum supplementary deformed bar reinforcement are very similar to such requirements in the ACI Building Code, and the minimum wall thickness ratios for steel tube are the same as those of the ACI Building Code.

Allowable axial stresses for composite columns are to be determined from an index of yield stress equivalent to the reliable amount of axial force necessary to squash or yield a short length of the composite cross section divided by the area of structural steel in the section. Due to lateral confinement provided by tubing the contained concrete and any longitudinal reinforcement contained within the tube can be expected to develop all of their nominal capacity. In the absence of effective lateral confinement, the concrete and the longitudinal bar reinforcement placed outside of structural steel are assigned strength reduction factors consistent with ACI recommendations for reinforced concrete columns. The overall stability of composite columns is so dependent on the elastic behavior of the structural steel that the effective yield strength of longitudinal reinforcement is limited to that of the structural steel. For the same reason, the effective stiffness index E_m does not include any influence of longitudinal reinforcement.

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Concrete cannot be relied upon to provide lateral stability to tubes or to encased shapes when strains exceed 0.2 percent, which is near the stress level of 55 ksi in steel. Thus 55 ksi was taken as an upper limit to effective yield strength for composite columns. (Possibly future research will demonstrate that the longitudinal reinforcement can be more effective both in strength and in stiffness.)

The modified section modulus, when multiplied by F_y approximates the pure bending strength of a cross section. The first term reflects the strength of structural steel flanges or tubing. Generally there will be more than 4 longitudinal bars in cross sections that require longitudinal reinforcing steel, and approximately $\frac{1}{3}$ of the bars will be located about 2.5 inches from the edge of the cross section. The last component of the S_m equation is an estimate of the helpful moment capacity of a concrete section reinforced by the web of the structural steel component.

Comparison With Building Code Requirements ACI 318-77.

The most recent edition of the ACI Building Code permits column designs for compression members essentially without flexure. The maximum service load axial force P_{ser} can be expessed

$$P_{ser} = \frac{\alpha\ \phi}{L.F.} (A_s F_y + 0.85\ f'_c\ A_c) \qquad \text{(H)}$$

with α = upper limit coefficient for axial force

ϕ = strength reduction factor

L.F. = net load factor

α = 0.85 and ϕ = 0.75 for concrete filled steel tubes

α = 0.85 and ϕ = 0.70 for concrete encased steel shapes

L.F. will vary between 1.4 and 1.7.

Thus according to the ACI Building Code with L.F. = 1.5,

$$P_{ser} = 0.425\ A_s f_y + 0.361\ f'_c\ A_c \qquad \text{for filled steel tubes}$$

$$P_{ser} = 0.397\ A_s f_y + 0.397\ A_{sr} F_{yr} + 0.337\ f'_c\ A_c \text{ for encased shapes}$$

and these values would pertain for most columns with slenderness ratios l/r less than 34.

The proposed regulations would allow axial loads as high as

$P_a = 0.600\ A_s F_y + 0.510\ f'_c\ A_c$ for filled steel tubes and

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$P_a = 0.600\ A_sF_y + 0.42\ A_{sr}F_{yr} + 0.360\ f'_c A_c$ for encased shapes when the slenderness ratio l/r is zero. When the length of composite column is near 34r, values of permissible service load become

$$P_a = 0.54\ A_sF_y + 0.46\ f'_c A_c \text{ for filled tubes}$$

$$P_a = 0.53\ A_sF_y + 0.38\ A_{sr}F_{yr} + 0.33\ f'_c A_c.$$

It is apparent that the proposed regulations for concrete filled steel tubes will allow service loads 27 to 33 percent greater than those now permitted by ACI rules. The component of service load permitted on the reinforced concrete portion of encased shapes is practically equal to that permitted by ACI rules, and the service load to be allowed on concrete encased structural steel columns will be greater than that permitted by ACI regulations only if the 33 percent higher load on the structural shape is larger than the reduced load allowed by the upper limit of F_y for an effective stress on longitudinal reinforcement.

Moment capacities and beam column interaction functions that are proposed will not allow highly eccentric column loads to be quite as high as values precisely determined according to regulations of the ACI Building Code. The apparent accuracy of precision in calculation does not appear to be justified since the simplified estimates of M_o lead to reasonable consistent comparisons with test results.

Slenderness effects are reflected in the proposed regulations by calculations of the quantity E_m and by the procedures for determining the effective radius of gyration r. The values of E_m influence the column length l_c at which long column behavior is assumed to commence. Contained concrete inside steel tubing is considered to be twice as stiff as surface concrete in the encasement outside steel shapes, and until laboratory tests demonstrate its effectiveness the stiffening effect of longitudinal reinforcement of the encasement is to be ignored. The radius of gyration of tubing or structural steel will be the major influence on resistance to buckling unless large amounts of encasement concrete can stiffen the entire cross section effectively. Values of r are to be taken as the greater of r_s for steel alone or one-third the overall thickness of an encased shape.

The interaction equation for composite columns includes, somewhat conservatively, the beam column interaction relationships of Section 1.6.1 in just one equation for which the beam column magnification of moments are not to be taken as less than unity.

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Laboratory Test Data Compared with Proposed Regulations

Tables 1 through 4 contain tabulations of load test results from axially loaded filled tubes, axially loaded encased shapes, eccentrically loaded filled tubes, and eccentrically loaded encased shapes. Values of loads allowed in accordance with the proposed design rules are shown as values P_a in each table, and ratios between actual test loads and allowable P_a are tabulated in the right-hand column of each table. The computed allowable eccentric loads include the effects of slenderness according to Eq. (F).

The average of ratios between test loads and allowable loads on filled tube specimens were 2.26 for 77 axially loaded columns and 2.50 for 32 eccentrically loaded columns. Somewhat lower averages were obtained for concrete encased shape specimens. The average ratio was 2.04 for 30 columns under axial load and 2.01 for 44 eccentrically loaded columns. Coefficients of variation were 15 to 20 percent of the average ratios. Except for the calculations that involved axially loaded steel tubes filled with concrete, a maximum yield stress of 55 ksi was used in the calculations of allowable loads.

On the basis of average values and coefficients of variation for ratios between test loads and loads that would be permitted according to the recommended design equations, there is adequate safety for general application. It can be observed that among the 176 specimens in the data sample, 8 ratios are lower than the nominal safety factor of 1.67. The specimens for which test loads were less than 1.67 times the allowable load involved intermediate slenderness ratios and reported concrete cylinder strengths above 4 ksi. Possibly some tests of supplementary specimens should be observed in order to review the validity of the reported results. Similar specimens with similar properties of materials and slenderness ratios show test loads well above the 1.67 multiple of proposed allowable loads.

Concrete filled tube composite columns showed generally higher ratios between test load and allowable load. Some liberalization of the proposed design rules might be anticipated as research, experience, and better estimating formulas reveal higher confidence levels.

The proposed rules fulfill the 2 basic goals set forth for a composite column design specification. It permits on reinforced concrete components of cross sections loads no greater than those permitted by the ACI building Code, and the allowable loads on composite columns will be greater than those now permitted on the steel alone according to ACI regulations.

TABLE 1: AXIALLY LOADED CONCRETE FILLED TUBES

O.D.	A_s	A_c	F_y	f'_c	I_s	$k\ell$	P_{test}	P_a	$\frac{P_{test}}{P_a}$
in.	in.^{2}	in.^{2}	ksi	ksi	in.^{4}	in.	k	k	
3.74	5.07	5.92	39.9	2.94	6.79	33.9	229.	120.	1.91
						55.9	209.	110.	1.90
						78.0	203.	98.	2.07
	1.63	9.36	50.7	3.62	2.63	53.9	150.	61.2	2.45
						55.9	131.	55.8	2.85
						78.0	119.	49.2	2.42
8.50	4.22	52.5	42.3	3.32	36.7	87.4	371.	178.	2.09
				4.32			509.	201.	2.53
	6.13	50.6	56.8	3.32	52.3		549.	264.	2.08
			50.8	4.32			645.	268.	2.41
3.74	1.63	9.36	49.0	3.49	2.63	80.	104.	47.3	2.20
4.76	2.14	15.6	45.2	3.06	5.71	41.3	162.	76.2	2.13
				3.51			192.	79.4	2.42
				3.06		91.	143.	69.7	2.21
				3.51			163.	67.2	2.43
	3.11	14.7	49.8	3.06	8.04	41.3	227.	107.	2.13
				3.51			245.	110.	2.23
				3.06		91.	180.	89.4	2.01
				3.51			195.	91.7	2.13
1.00	0.11	0.68	76.0	4.04	.0124	42.	3.52	1.96	1.80
1.50	0.48	1.29			.116		24.7	11.1	2.22
2.00	0.40	2.74			.185		27.1	15.9	1.70
3.00	0.59	6.47		3.95	.646		72.0	32.7	2.20
14.0	18.7	135.	51.5	5.52	769.	22.	2576.	950.	2.71
				4.76			2408.	898.	2.69
	13.5	140.		3.40	431.	21.1	1671.	654.	2.55
	8.07	146.	40.1	3.04	316.	21.5	791.	417.	1.90
5.01	0.99	18.7	53.8	9.60	3.04	19.7	289.	119.	2.43
			47.7				289.	115.	2.51
5.00	1.78	17.9	53.8		5.31	20.0	293.	140.	2.10
			47.7				293.	134.	2.19
4.00	1.49	11.1	87.8	4.95	2.78	60.	184.	88.5	2.23
							180.		2.18
4.76	2.33	15.5	65.5	4.99	6.16	413.	260.	119.	2.19
				4.29			246.	114.	2.16
				3.76			214.	110.	1.96
6.00	2.29	26.0	60.2	3.03	9.88	89.4	211.	91.1	2.32
							198.		2.17
3.01	0.63	6.5	52.7	3.62	0.88	55.	55.	83.4	2.25
				5.93		24.	95.2	36.2	2.63
				3.76			74.2	29.9	2.48
4.50	1.72	14.2	60.0	4.20	4.11	33.	165.	85.5	1.93
5.00	1.46	18.2	42.0	5.10	4.40	59.	143.	73.2	1.95
6.00	1.14	27.1	48.0	3.05	5.02		153.	67.5	2.27
				3.75			163.	75.8	2.15
5.51	6.14	17.7	38.5	4.66	20.0	16.	663.	180.	3.68
			39.0				663.	182.	3.64
5.53	3.25	20.8	41.9	4.74	11.6		410.	129.	3.17
			43.2				410.	132.	3.12
6.62	3.62	30.6		4.56	18.7	32.	451.	159.	2.84
				6.26			489.	184.	2.66
				3.34			392.	141.	2.79
3.50	2.36	7.26	58.0	5.81	3.17	68.	138.	74.6	1.85
				5.75		56.	160.	81.4	1.97
				5.65		44.	161.	87.3	1.84
				6.06		32.	206.	94.0	2.19
				5.92		20.	223.	98.3	2.27
3.25	0.55	7.74	70.0	6.00	0.705	68.	50.5	30.1	1.68
				5.36		56.	66.2	32.6	2.03
				5.92		44.	80.0	37.5	2.13
						32.	90.0	40.7	2.21
						20.	110.0	43.3	2.54
						10.	119.2	45.1	2.64
6.64	2.13	32.5	43.2	2.60	11.4	12.	298.	97.1	3.07
						78.	185.	87.1	2.12
				4.95		12.	274.	135.	2.02
						78.	206.	119.	1.73
			46.0	5.30		12.	294.	145.	2.03
						78.	170.	127.	1.34
				4.87		12.	299.	136.	2.17
						78.	155.	121.	1.28
	3.98	30.6	32.1	4.72	20.9	90.	236.	131.	1.80
	2.89	31.5	37.8	4.75	15.3	90.	254.	121.	2.10
		STD. DEV. 0.45 - 20%						Ave.	2.26

TABLE 2: AXIALLY LOADED ENCASED SHAPES

Steel Shape	h_1	h_2	A_s	A_c	f'_c	F_y	$k\ell$	P_{test}	P_a	$\frac{P_{test}}{P_a}$
	in.	in.	$in.^2$	$in.^2$	ksi	ksi	in.	k	k	
3 x 1½	5	3.5	1.18	16.32	2.60	36.0	46.	81.4	36.5	2.23
							64.	71.5	33.3	2.15
							82.	63.0	29.3	2.15
							100.	43.6	24.5	1.78
							118.	50.6	18.9	2.68
							136.	36.1	14.2	2.54
							154.	33.9	11.1	3.06
5 x 4½	7	6.5	5.88	39.6	2.60	36.0	9.	352.	163.	2.15
							46.	308.	158.	1.95
							82.	307.	150.	2.05
							118.	288.	138.	2.09
							153.	231.	123.	1.88
8 x 6	10	8.	10.3	69.7	2.60	36.0	84.	572.	269.	2.13
	12	10.		110.			84.	726.	310.	2.35
	14	12.		158.			84.	856.	356.	2.41
	16	12.	19.1	173.			36.	1051.	568.	1.85
							72.	990.	558.	1.77
							108.	926.	544.	1.70
							144.	937.	526.	1.78
							180.	933.	504.	1.85
5½ x 5½	9.5	9.5	6.66	82.6	4.66	41.5	169.	482.	240.	2.01
					4.28	42.7	137.	526.	253.	2.08
					4.77	40.2	98.	590.	276.	2.13
					4.29	40.0	50.	572.	279.	2.05
					4.24	55.0	137.	528.	287.	1.84
					4.24	72.6	168.	528.	298.	1.77
					4.27	70.8	137.	554.	329.	1.68
					4.77	72.5	98.	545.	382.	1.43
					4.39	41.5	136.	513.	253.	2.03
					4.30	70.7	136.	517.	331.	1.56
									Ave.	2.04
									Std. Dev.	0.344
										16.8%

TABLE 3: ECCENTRICALLY LOADED CONCRETE FILLED STEEL TUBES

O.D.	A_s	A_c	f_y	f'_c	r_s	S_s	$k\ell$	P_{TEST}	θ	P_a	M_o	P_{ALL}	$\frac{P_{TEST}}{P_{ALL}}$
in.	in.^{2}	in.^{2}	ksi	ksi	in.	in.^{3}	in.	k	in.	k	in-k	k	
4.50	1.72	14.2	55.0	4.20	1.55	1.83	30	100	1.00	75.6	75.5	46.7	2.14
								90	1.18			43.2	2.09
								75	1.75			34.3	2.19
								30	2.82			24.1	2.08
								25	5.76			12.7	1.96
6.00	1.14	27.1	48.0	3.75	2.10	1.67	40	128	0.69	77.4	60.1	50.3	2.54
								95	1.66			30.6	3.11
								64	2.39			22.9	2.79
				3.05				30	4.77	68.6	60.1	12.2	2.46
								30	4.43			13.1	2.29
5.00	1.40	18.2	42.0	5.10	1.77	1.76	42	128	0.61	71.6	55.4	48.8	2.63
								120	0.93			40.5	2.96
								90	1.57			29.4	3.06
								79	1.77			26.9	2.94
								79	1.59			29.1	2.71
								78	1.81			26.5	2.95
								69	2.19			22.8	3.03
								60	2.60			19.7	3.04
								39	3.74			14.2	2.74
								20	7.05			7.8	2.57
								10	13.0			4.25	2.35
5.00	1.85	23.2	55.0	6.50	1.67	5.60	42	250	1.24	116.0	231	85.4	2.93
								150	2.43			65.1	2.30
								150	2.87			59.4	2.53
								100	4.50			44.0	2.29
4.00	1.31	14.7	48.0	3.40	1.60	1.68	42	84	0.52	52.5	60.5	48.0	2.00
								54	1.70			26.5	2.04
								20	5.48			10.6	1.89
4.00	1.94	14.1	48.0	4.18	1.58	2.42	42	98	1.21	60.5	87.1	43.3	2.27
								68	2.38			29.7	2.29
								59	3.22			23.8	2.48
								29	6.93			12.2	2.38
												Ave.	2.50
												Std. Dev.	0.375
													15.0%

TABLE 4: ECCENTRICALLY LOADED CONCRETE ENCASED STEEL SHAPES

h_1	h_2	A_s	A_c	A_w	S_s	$k\ell$	f'_c	f_y	P_{TEST}	e	P_a	M_o	P_{ALL}	$\frac{P_{TEST}}{P_{ALL}}$
in.	in.	$in.^2$	$in.^2$	$in.^2$	$in.^3$	in.	ksi	ksi	k	in.	k	in-k	k	
9.45	9.45	6.66	82.6	1.52	4.7	135.9	4.80	41.5	251	1.57	248	265	117	2.15
							4.63		265		245	264	116	2.29
							4.03		240		232	259	112	2.14
							4.50	55.	265		277	334	138	1.92
							4.36		251		274	332	137	1.83
							4.03		223		267	327	135	1.66
12.60	8.27	5.18	99.0	2.01	2.2	96.5	4.64	39.5	269		244	211	103	2.62
							4.36		234		236	209	101	2.32
							4.28		229		234	208	100	2.28
16.0	12.0	19.1	172.9	5.16	16.3	120.	2.52	32.3	672	1.00	478	673	331	2.03
							2.36		486	2.00	470	656	235	2.06
							3.92		515	2.00	553	760	273	1.88
							2.68		361	3.00	487	687	188	1.92
							2.68		296	4.00	487	687	151	1.96
							2.80		262	5.00	493	697	127	2.06
							2.72		231	6.00	489	691	107	2.15
							3.08		199	7.00	514	726	98.3	2.02
							3.00		168	8.00	510	721	86.2	1.95
7.0	6.5	5.88	39.6	1.45	2.93	82.	2.80	33.6	161	0.75	138	111	85.0	1.89
									168	0.80			82.6	2.03
						28.6			202	0.75	153		92.9	2.18
									228	0.80			90.1	2.53
						28.6			166	1.00			80.1	2.07
						45.5			224	0.50	14.9		106	2.11
									164	1.00			78.6	2.09
						82.			141	1.00	138		73.9	1.91
						118.			161	0.50	126		89.5	1.80
									119	1.00			67.8	1.75
						153.			99	1.00	111		60.8	1.63
									78	1.50			49.1	1.59
									74	2.00			41.0	1.81
8.0	7.0	2.94	53.1	0.96	0.88	84.	3.71	40.7	195	0.40	121	85.4	88.2	2.21
							3.28	45.6	108	0.80	121	90.2	68.9	1.57
							4.20	39.3	88	1.50	127	85.0	46.1	1.91
						120	4.58	39.5	201	0.20	117	86.6	97.2	2.07
						120	4.31	39.5	135	0.40	114	85.8	80.8	1.67
						120	3.25	42.7	88	0.80	106	85.8	59.4	1.48
						120	4.28	39.5	68	1.50	113	85.7	42.8	1.59
							4.28	42.4	211	0.40	132	90.8	95.1	2.22
							3.91		130	0.80	126	89.2	69.9	1.86
		1.47	54.5	0.68	0.37	84	2.89	43.0	116	0.40	79.8	57.9	58.4	1.99
							3.81		108	0.80	93.6	61.0	49.3	2.19
7.0	8.0			0.96	2.15		3.81	39.5	214	0.40	91.4	113.9	75.7	2.83
							3.46		175	0.80	86.1	112.3	61.0	2.87
												Ave.		2.02
												Std. Dev.		0.31
														15.3%

REFERENCES

1. Building Code Requirements for Reinforced Concrete (ACI 318-77) American Concrete Institute, Detroit, 1977.

2. Manual of Steel Construction, 7th Edition, American Institute of Steel Construction, New York, 1970.

3. Light Gage Cold Formed Steel Design Manual, American Iron and Steel Institute, New York, 1972.

4. Stevens, R. F., "Encased Stanchions", The Structural Engineer, Vol. 43, No. 2, February 1965.

5. Janss, J., "Le Calcul des Charges Ultimes des Colonnes Metalliques Enrobees de Beton MT 89", Industrial Center of Scientific and Technical Research for Fabricated Metal, Brussels, April 1974 (in French).

6. Gardner, N. J., "Use of Spiral Steel Tubes in Pipe Columns", Journal of the American Concrete Institute, Vol. 65, No. 11, November 1968

7. Knowles, R. B. and R. Park, "Strength of Concrete Filled Steel Tubular Columns", Journal of the Structural Division, ASCE, Vol. 95, No. ST12, December 1969.

8. Gardner, N. J. and E. R. Jacobsen, "Structural Behavior of Concrete Filled Steel Tubes", Journal of the American Concrete Institute, Vol. 64, No. 7,July 1967.

9. Furlong, Richard W., "Strength of Steel Encased Concrete Beam Columns", Journal of the Structural Division, ASCE, Vol. 94, No. ST1, Proc. Paper 5761, January 1968.

10. Stevens, R. F., "Encased Stanchions", The Structural Engineer, Vol. 43, No. 2, February 1965.

11. Loke, Y. O., "The Behavior of Composite Steel-Concrete Columns", Ph.D. Thesis University of Sidney, December 1968.

12. Furlong, R. W., "AISC Column Design Logic Makes Sense for Composite Columns Too", Engineering Journal, American Institute of Steel Construction, First Quarter, New York, January 1976.

The objectives of Task Group 13 are ~~is~~ 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.

Currently, we have 10 members in our Task Group. The membership consists of ~~they are~~

J. W. Clark
S. J. Errera
A. L. Johnson
C. Marsh
T. M. Murray
A. Ostapenko
T. Pekoz
W. P. Vann
G. Winter
W. W. Yu

~~Prof. S. T.~~ Jerry Wang of the Univ. of Kentucky will be a new member of ~~the~~ our Task Group. ~~In view of the fact that~~ Because 8 of the 11 members are ~~college or~~ university faculty members, we plan to invite 2 or 3 practical engineers to be new members of our Committee to carry out our new assignments.

During the last year, members of our Task Group have been actively involved with various research projects ~~on cold-formed steel structures and aluminum structures.~~ sponsored by ~~steel~~ the industry and governmental agencies, and ~~they~~ have ~~also been engaged~~ participated in the development of new and revised design criteria. We have maintained a close ~~relationship~~ contact with the ASCE Committee on Cold-Formed Members. ~~In addition,~~ In June 1978, our Task Group cosponsored the 4th International Specialty Conference on Cold-Formed Steel Structures held in St. Louis, Mo.

~~Our~~ In the future, our activity will include the revision of Chapters 4 and Chapter 9 of the SSRC Guide. ~~and~~ Because ~~the~~ AISI plans to issue a revised Specification on cold-formed steel design in 1980 or 1981, we will ~~revise~~ update ~~Chap~~ Ch. 9 ~~the~~ according to the recent research findings and the revised AISI Spec. ~~The~~ In addition, ~~because~~ our Committee will ~~the~~ cosponsor ~~ship of~~ the 5th International Specialty Conference on Cold-Formed Steel Structures ~~to be~~ scheduled for November 18-19, 1980. Copies of ~~the~~ Call for papers for the 5th Conf. are now available at the information desk.

Chladny.

PANEL DISCUSSION

General Comments on Composite Steel Systems - W. J. Le Messurier

Mr. Le Messurier reviewed various cases of steel framing where composite steel beam design can be utilized beyond the simple cases that are usually used. One example involved a continuous frame where the beam was proportioned to form plastic hinges at beam ends at certain load levels and composite action was utilized in positive moment areas.

1978 ANNUAL BUSINESS MEETING

The Structural Stability Research Council holds an annual meeting for the purpose of reporting activities, election of officers, and presentation of the budget for the following year. The 1978 Annual Meeting was held on May 17, 1978, in conjunction with the Annual Technical Session at the Copley Plaza Hotel, Boston, Massachusetts.

The minutes of the 1978 Annual Meeting follow:

CALL TO ORDER

The meeting was called to order to 11:35 a.m. by the Chairman, Dr. George Winter. Approximately 45 persons were present.

The Chairman introduced the Vice Chairman, Dr. John W. Clark, and the Director, Dr. Lynn S. Beedle, and welcomed the members and friends.

The Chairman acknowledged the support given by the National Science Foundation and thanked Emile W. J. Troup, AISC Regional Engineer in Boston for his very efficient handling of the local arrangements and publicity.

ELECTION OF OFFICERS AND EXECUTIVE COMMITTEE MEMBERS

The Nominating Committee, chaired by A. L. Johnson, submitted the following nominations:

Chairman: John W. Clark

Vice Chairman: Jerome S. B. Iffland

Executive Committee: Reidar Bjorhovde, Gerard F. Fox, Roland R. Graham, Bruce G. Johnston, Donald R. Sherman, Joseph A. Yura.

Voting for all nominees was conducted by letter ballot to the membership. Results of the balloting were announced:

Chairman: John W. Clark (3 year term starting 1 October 1978)

Vice Chairman: Jerome S. B. Iffland (3 year term starting 1 October 1978)

Executive Committee: Gerard F. Fox, Roland R. Graham, Bruce G. Johnston (3 year terms effective immediately)

MEMBERS-AT-LARGE

The following persons were nominated by the Executive Committee for election to Member-at-Large:

M. J. Abrahams, Parsons, Brinkerhoff, Quade & Douglas

F. Y. Cheng, University of Missouri-Rolla

P. J. Dowling, Imperial College of Science & Technology

D. Krajcinovic, University of Illinois at Chicago Circle

F. W. Stockwell, American Institute of Steel Construction

R. Wolchuk, Wolchuk and Maylbaurl

The motion that all moninees be elected as Members-at-Large was carried unanimously.

LIFE MEMBERSHIP

The Life Membership Nominating Committee, chaired by R. R. Graham, submitted the following person for life membership:

A. Amirikian

The motion that A. Amirikian become a Life Member was carried unanimously.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director including the proposed budget for the fiscal year 1978-79. He pointed out that $5,000 had been added for "Research Support."

Budget 1978-79:

Expected balance, Oct. 1, 1978	$20,400
Income	25,700
Expenditures	35,400
Expected balance, Sep 30, 1979	10,700

The budget was approved.

SUMMARY OF ACTIVITIES

The Director gave a few highlights of the work of the Council over the past year. The extent of the work and the progress of the various task groups is illustrated by the presentations at the Technical Session.

The 2nd Edition of the Guide, of which 3,297 copies have been sold, is now out of print. Over 2,100 copies of the 3rd Edition have been sold. Plans for the 4th Edition are underway with T. V. Galambos as Editor; estimated publication date -- 1985. Serious consideration is being given to the assembly of interim state-of-art reports, as presented at the Annual Technical Session. These could be published thereafter as "Guide Supplements."

A paper on Research Priorities is being prepared for publication by ASCE. Pre-prints should be available at the ASCE Boston meeting in April 1979.

Technical Memorandum No. 5 "General Principles for the Stability Design of Metal Structures" has been drafted by an Ad-Hoc Committee chaired by T. V. Galambos. Upon approval of the membership, TM 5 will be submitted for publication.

A publications brochure, featuring the publications pertaining to and resulting from the International Colloquium, has been distributed by Headquarters. Work on the Colloquium Summary Report will commence; Task Group 11, in cooperation with ECCS Committee 8, has been charged with the task.

Mrs. Lesleigh G. Federinic has been appointed SSRC Administrative Secretary. Dr. Riccardo Zandonini will join the staff shortly as Technical Secretary, and will devote much of his effort to the Colloquium Summary Report.

NEXT ANNUAL TECHNICAL SESSION AND MEETING

The Chairman announced that the next Annual Technical Session and Meeting will be held at the William Penn Hotel in Pittsburgh; dates will be 23-25 April 1979. The title of the Panel Discussion will be "Space Frame Stability."

ADJOURNMENT

The meeting was adjourned at 12:10 p.m.

ANNUAL TECHNICAL SESSION & MEETING ATTENDANCE

Participant	Affiliation
Abrahams, M. J.	Parsons, Brinkerhoff, Quade & Douglas, New York
Anderson, P.E.	Combustion Engineering, Inc., Windsor, Connecticut
Austin, W. J.	Rice University, Houston, Texas
Beauregard, D. Z.	Combustion Engineering, Windsor, Connecticut
Beauregard, R. L.	Combustion Engineering, Windsor, Connecticut
Beedle, L. S.	Lehigh University, Bethlehem, Pennsylvania
Bernstein, M. D.	Foster Wheeler Energy Corp., Livingston, New Jersey
Bjorhovde, R.	University of Windsor, Edmonton, Alberta
Born, J.	John Born Associates, Cambridge, Massachusetts
Bua, D. T.	Bayside Engineering Assoc., Inc., Boston, Mass.
Capanoglu, C.	Earl & Wright Consulting Engineers, San Francisco, Cal.
Chajes, A.	University of Massachusetts, Amherst, Massachusetts
Chalabi, A. F.	Worcester Polytechnic Institute, Worcester, Mass.
Chen, K.	Frederick R. Harris, Inc., Boston, Massachusetts
Chen, S. S.	Argonne National Laboratory, Argonne, Illinois
Chen, W. F.	Purdue University, West Lafayette, Indiana
Cheng, F. Y.	University of Missouri-Rolla, Rolla, Missouri
Chin, S. L.	I.T.T. Grinnell, Providence, Rhode Island
Clark, J. W.	Alcoa Laboratories, Alcoa Center, Pennsylvania
Cox, J. W.	McDermott Hudson Engineering, Houston, Texas
Dabrila, L.	Northeastern University (student), Boston, Mass.
Dalwadi, N. B.	Combustion Engineering, Inc., Windsor, Connecticut
Davis, F. B.	West End Iron Works, Cambridge, Massachusetts
de Clercq, H.	South African Institute of Steel Construction, Johannesburg, South Africa
Desai, N. T.	B-P Technical Services, Inc., Braintree, Mass.
Dickson, W.	Northeastern University (student), Boston, Mass.
Di Julio Jr., R. M.	Northridge, California
Di Martino, J. M.	Zaldastani Associates, Inc., Boston, Massachusetts
Dowling, P. J.	Imperial College, London, England
Durkee, J. L.	Modjeski & Masters, Harrisburg, Pennsylvania
Dwight, J. B.	University Engineering Laboratory, Cambridge, England
Egan, K. K.	Allen & Demurjian, Inc., Boston, Massachusetts
Elgaaly, M.	Bechtel Associates Professional Corp., Ann Arbor, Mich.
Errera, S. J.	Bethlehem Steel Corporation, Bethlehem, Pennsylvania
Federinic, L. G.	SSRC, Lehigh University, Bethlehem, Pennsylvania
Foutch, D. A.	University of Illinois, Urbana, Illinois
Fox, G. F.	Howard, Needles, Tammen & Bergendoff, New York
Furlong, R. W.	University of Texas, Austin, Texas
Galambos, T. V.	Washington University, St. Louis, Missouri
George, E. D., Jr.	I.T.T. Grinnell Corp., Providence, Rhode Island
Gilmor, M. I.	Canadian Institute of Steel Construction, Willowdale, Ontario
Gladstone, D. D.	United Engineers & Constructors, Boston, Mass.
Govalet, G. E.	ALNASCO, Pittsfield, Massachusetts

Hagen, H. W.	LeMessurier Associates/SCI, Cambridge, Mass.
Hall, D. H.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Hall, F. X.	Howard, Needles, Tammen & Bergendoff, Boston, Mass.
Hansell, W. C.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Hartley, W. S.	Linenthal Eisenberg Anderson Engineering, Boston, Mass.
Hebel, R. E.	Stone & Webster Engineering Corp., Boston, Mass.
Higgins, T. R.	American Inst. of Steel Construction, Darien, Conn.
Hooper, I. M.	Seelye Stevenson Value & Knecht, New York
Iffland, J. S. B.	Iffland Kavanagh Waterbury, New York
Iyengar, S. H.	Skidmore, Owings & Merrill, Chicago, Illinois
Jayachandran, P.	Worcester Polytechnic Inst., Worcester, Massachusetts
Johnson, A. L.	American Iron & Steel Inst., Washington, DC
Johnson, D. L.	Butler Manufacturing Co., Grandview, Missouri
Johnston, B. G.	Consultant, Tucson, Arizona
Klabbers, L.	G.F.C. & C, Inc., Boston, Massachusetts
Konicki, W. P.	Badger America, Cambridge, Massachusetts
Krajcinovic, D.	University of Illinois at Chicago Circle, Chicago, Ill.
Lee, C. K.	Parsons Brinckeroff Quade & Douglas, Boston, Mass.
Leet, K.	Northeastern University, Boston, Massachusetts
Leitch, D. G.	University of Lowell, Lowell, Massachusetts
LeMessurier, W. J.	LeMessurier Associates/SCI, Cambridge, Massachusetts
Lelinto, J.	Northeastern University (student), Boston, Mass.
Lele, A. D.	Zaldastani Associates, Inc., Boston, Massachusetts
Maibach, L.	M.B.T.A. Engr. Dept., Charlestown, Massachusetts
Marsh, C.	Concordia University, Montreal Quebec
Mazur, S.	Howard, Needles, Tammen & Bergendoff, Boston, Mass.
McCarthy, C. M.	M.D.C. Engineering Div., Boston, Massachusetts
McGrath, F. X.	Bayside Engineering Assoc., Inc., Boston, Mass.
McNamara, R. J.	Gillum-Colaco, Cambridge, Massachusetts
Milek, Jr. W. A.	American Inst. of Steel Constr., New York
Miller, C. D.	Chicago Bridge & Iron Co., Plainfield, Illinois
Miller, C. J.	Case Western Reserve Univ., Cleveland, Ohio
Mitchell, J.	Combustion Engineering, Inc., Windsor, Connecticut
Mulcahy, J.	Mulcahy Engineers, Cranston, Rhode Island
Murray, R. W.	Riley Stoker Corp., Worcester, Massachusetts
Nash, K.	ALNASCO, Pittsfield, Massachusetts
Newton, E. H.	United Engineers and Constructors, Boston, Mass.
Norton, R.	Northeastern University (student), Boston, Mass.
Oktay, Y.	Badger America, Cambridge, Massachusetts
O'Neill, D.	Maurice A. Reidy Engineers, Boston, Massachusetts
Pattison, S.	United Engineers & Constructors, Boston, Mass.
Pekoz, T.	Cornell University, Ithaca, New York
Pineau, D.	Northeastern University (student), Boston, Mass.
Pinkham, C. W.	S.B. Barnes & Assoc., Los Angeles, California
Pretzer, C. A.	C.A. Pretzer Assoc., Inc., N. Scituate, Rhode Island

Razzaq, Z.	Southern Illinois University, Carbondale, Illinois
Reidy, M. A.	Maurice A. Reidy Engineers, Boston, Massachusetts
Reiniger, L. G.	Badger America, Cambridge, Massachusetts
Rivard, J.	M.D.C. Engineering Div., Boston, Massachusetts
Rouillard, B.	Rene Mugnier Assoc., Cambridge, Massachusetts
Rust, W. D.	General Services Administration, Washington, DC
Sen, S.	Storch Engineers, Boston, Massachusetts
Sfintesco, D.	C.T.I.C.M., Puteaux, France
Sherman, D. R.	University of Wisconsin-Milwaukee, Milwaukee, Wisc.
Simitses, G. J.	Georgia Inst. of Technology, Atlanta, Georgia
Snow, I.	Howard, Needles, Tammen & Bergendoff, Boston, Mass.
Sonstroem, K.	Riley Stoker Corp., Worcester, Massachusetts
Springfield, J.	C.D. Carruthers & Wallace, Rexdale, Ontario
Stockwell, F. W.	American Inst. of Steel Construction, New York
Stori, J.	ALNASCO, Pittsfield, Massachusetts
Szubelick, F. P.	Combustion Engineering, Inc., Windsor, Connecticut
Temple, M. C.	University of Windsor, Windsor, Ontario
Terry, R.	Northeastern University (student), Boston, Mass.
Testa, R. B.	Columbia University, New York
Thomaides, S. S.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Troup, E. W. J.	American Inst. of Steel Constr., Boston, Mass.
Troupe, J. P.	Allen & Demurjian, Inc., Boston, Massachusetts
Varney, R. F.	Federal Highway Administration, Washington, DC
Wang, C. K.	University of Wisconsin, Madison, Wisconsin
Weaver, W. L.	Wayne L. Weaver Assoc., Boston, Massachusetts
Weis, V. M.	United Engineers & Constructors, Boston, Mass.
Wiesner, K.	LeMessurier Assoc./SCI, Cambridge, Massachusetts
Winter, G.	Cornell University, Ithaca, New York
Woolf, A.	Abraham Woolf & Assoc., Boston, Massachusetts
Woolf, S.	Abraham Woolf & Assoc., Boston, Massachusetts
Yektai, H.	Southern Illinois University, Carbondale, Illinois
Yu, W. W.	University of Missouri-Rolla, Rolla, Missouri

List of Publications

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

Chen, W. F., Suzuki, H., Chang, T. Y.
ANALYSIS OF CONCRETE CYLINDER STRUCTURES UNDER HYDROSTATIC LOADING, Purdue University Technical Report No. CE-STR-78-2, April 1978

ECCS/IABSE
STABILITY OF STEEL STRUCTURES, FINAL REPORT, Second International Colloquium, Liege, 13-15 April 1977

Halasz, O. and Ivanyi, M., Editors
PROCEEDINGS, REGIONAL COLLOQUIUM ON STABILITY OF STEEL STRUCTURES, HUNGARY, Budapest, 19-21 October 1977

Holmgren, J.
DEFORMATIONER HOS INGJUTNA ARMERINGSSTANGER, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 114, Stockholm 1975

Ingvarsson, H.
BETONGPLATTORS HALLFASTHET OCH ARMERINGSUTFORMNING VID HORNPELARE, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 122, Stockholm 1977

Ingvarsson, H.
CONCRETE SLABS SUPPORTED ON CORNER COLUMNS - SUMMARY, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 129, Stockholm 1977

Ingvarsson, L.
COLD-FORMING RESIDUAL STRESSES and BOX COLUMNS BUILT UP BY TWO COLD-FORMED CHANNEL SECTIONS WELDED TOGETHER, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 121, Stockholm 1977

Ingvarsson, L.
RESIDUAL STRESSES IN WELDED BOX COLUMNS, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 119, Stockholm 1977.

Ingvarsson, L.
WELDED BOX COLUMNS OF HIGH STRENGTH STEEL, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 120, Stockholm 1977

Ingvarsson, L.
WELDING AND COLD-FORMING RESIDUAL STRESSES AND THEIR EFFECT ON BUCKLING OF BOX COLUMNS OF HIGH STRENGTH STEEL - SUMMARY, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 130, Stockholm 1977

Nylander, H. and Sundquist, H.
GENOMSTANSNING AV PELARUNDERSTODD PLATTBRO AV BETONG MED OSPAND ARMERING, Institutionen for Byggnadsstatik, Kungl. Tekniska Hogskolan, Maddelande No. 104, Stockholm 1972

Nylander H. and Kinnunen, S.
GENOMSTANSNING AV BETONGPLATTA VID INNERPELARE BROTTSTADIEBERAKNING, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 118, Stockholm 1976

Nylander, H., Kinnunen, S., Ingvarsson, H.
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Nylander, H., Mattsson, J., Nylander, B., Lindstrom, G.
BUCKLING AV TRYCKT TUNN FLANS VID I-BALK, Institutionen for Byggnadsstatik Kungl. Tekniska Hogskolan, Maddelande No. 128, Stockholm 1977

SSRC
PROCEEDINGS, INTERNATIONAL COLLOQUIUM ON STABILITY OF STRUCTURES UNDER STATIC AND DYNAMIC LOADS, Washington, D.C., 17-19 May 1977

Yu, W. W. and Senne, J. H., Editors
RECENT RESEARCH AND DEVELOPMENTS IN COLD-FORMED STEEL STRUCTURES, Fourth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri, 1-2 June 1978, Vol. I & II

SSRC Chronology

19-21 Oct 77	- Regional Colloquium on Stability of Steel Structures, Budapest, Hungary
1-2 Nov 77	- Executive Committee Meeting, Bethlehem, Pa.
28 Jan 78	- Ad-Hoc Committee on Column Problems Meeting, St. Louis, Missouri
31 Jan 78	- SSRC Secretary - T. Kanchanalai - resigned to return to Thailand
21 Feb 78	- Chairman's Meeting, New York City
15-17 May 78	- Annual Technical Session and Meeting, Executive Committee Meeting, Boston, Mass.
17 May 78	- L. G. Federinic appointed SSRC Administrative Secretary
17 May 78	- New task group formed - TG 23 "Effect of End Restraint on Initially Crooked Columns" (W. F. Chen, Chairman)
1-2 Jun 78	- Fourth International Specialty Conference on Cold-Formed Steel Structures, St. Louis, Missouri
1 Jul 78	- R. Zandonini assumed duties of SSRC Technical Secretary
1 Oct 78	- New Chairman - J. W. Clark - took office
1 Oct 78	- New Vice Chairman - J. S. B. Iffland - took office

Finance

	Fiscal Year 10/77 - 9/78		Fiscal Year 10-78 - 9/79	
	Budget (approved 5/16/77)	Cash Statement 10/1/77-9/30/78	Budget (approved 5/17/78)	
BALANCE at Beginning of Period	$ 6,400.00	$19,478.50 (a)	$20,400.00	
INCOME				
Contributions				
Sponsoring Organizations				
AISC	4,000.00	4,000.00	4,000.00	
AISI	5,000.00	5,000.00	5,000.00	
CISC	1,000.00	1,000.00	1,000.00	
NSF	7,000.00	9,000.00	9,000.00	(b)
Participating Organizations	1,000.00	1,300.00 (c)	1,500.00	
Participating Firms	2,000.00	100.00 (d)	2,000.00	
Total Contributions	$20,000.00	$20,400.00	$22,500.00	
Registration Fees	2,000.00	2,450.00 (e)	2,000.00	
MAL Subscription Fees	---	1,447.00	---	
Guide Royalties	1,000.00	2,235.75	1,000.00	
Sale of Publications	---	575.24 (f)	---	
Interest	200.00	188.92	200.00	
TOTAL INCOME	$23,200.00	$27,296.91	$25,700.00	
EXPENDITURES				
Technical Services (Hqtrs)				
Director	1,000.00	2,500.04 (g) (h)	1,000.00	
Tech/Admin. Secretary	7,000.00	8,214.28 (g) (h)	10,400.00	
Secretarial/Clerical Services	3,500.00	1,558.09 (g) (h)	1,000.00	
Supply, Phone, Mailing	750.00	1,310.13	1,400.00	
Travel	750.00	149.95	1,000.00	
Total Technical Services	$13,000.00	$13,732.49	$14,800.00	
Research Support	---	---	5,000.00	
Annual Meeting and Proceedings				
Annual Proceedings	1,300.00	714.00	1,300.00	
Colloquium Proceedings	---	465.00	---	
Colloquium Summary	---	---	3,800.00	
Expenses and Services	7,700.00	3,446.96	7,700.00	
Travel	2,000.00	1,761.19	2,000.00	
Total AM & Proceedings	$11,000.00	$ 6,387.15	$14,800.00	
United Engineering Trustees	100.00	100.00	100.00	
Travel	500.00	1,050.38 (i)	500.00	
Contingencies	400.00	320.00 (j)	200.00	
TOTAL EXPENDITURES	$25,000.00	$21,590.02	$35,400.00	
BALANCE at End of Period	$ 4,600.00	$25,185.39 (k)	$10,700.00	

EXPLANATORY NOTES

(a) Depositories (as of 10/1/77)

General Account (UET)	\$18,919.30
Technical Services (Lehigh Univ.)	(4,994.58)
NSF Grant (Colloquium)	5,553.78
	\$19,478.50

(b) An application has been made to NSF to support the 1979 Annual Technical Session.

(c) Aluminum Association(\$500), ECCS(\$100), FHWA(\$200), NFEC(\$100), NSR&D(\$100), SESA(\$100), SJI(\$200).

(d) Canvass of Participating Firms not yet conducted.

(e) Included \$450 paid luncheons.

(f) Included 16 ECCS Manuals for which payment was made to D. Sfintesco. See note (k).

(g)

	Technical Services	NSF Colloquium	NSF Boston ATS&M
Director	\$2,500.04		
Tech Secretary	2,971.51		
Admin Secretary	3,015.58	\$ 972.20	\$1,254.99
Secretarial/Clerical (includes Employees Benefits)	475.98	639.64	442.47
	\$8,963.14	\$1,611.84	\$1,697.46

(h) During the period February to July 1978, the Council had no Technical Secretary, necessitating increased time by the Director and Administrative Secretary. This also required additional clerical help. Administrative Secretary originally budgeted under "Secy./Clerical Services."

(i) At November 1977 meeting, EC increased travel to \$1,000 to cover support of Ad Hoc Committee on Column Problems meeting in St. Louis, January, 1978.

(j) Payment to D. Sfintesco for ECCS Manuals sold.

(k) Depositories (as of 9/30/78)

General Account (UET)	\$17,326.94
Technical Services (Lehigh Univ.)	337.95
NSF Grant (Colloquium)	3,144.55
NSF Grant (Boston ATS&M)	4,375.95
	\$25,185.39

Register

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Vice Chairman: J. S. B. Iffland*

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S. J. Errera (80)
G. F. Fox (81)
T. V. Galambos (79)
R. R. Graham (81)
T. R. Higgins (Technical Consultant)
B. G. Johnston (81)
W. A. Milek, Jr. (79)
J. Springfield (80)
G. Winter**

---

* Past Vice Chairman
** Past Chairman

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

B. G. Johnston, Chairman
T. V. Galambos, Editor
L. S. Beedle
J. S. B. Iffland
G. Winter

B. Committee on Finance

G. F. Fox, Chairman
J. S. B. Iffland
L. S. Beedle

C. Ad Hoc Committee on Research Priorities

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

D. Ad Hoc Committee on Column Problems

T. V. Galambos, Chairman
R. Bjorhovde
W. F. Chen
E. H. Gaylord
J. S. B. Iffland
J. Springfield
J. A. Yura

Secretaries: R. Zandonini, Technical
L. G. Federinic, Administrative

* Present Chairman - J. S. B. Iffland

TASK GROUPS

Task Group 1 - Centrally Loaded Columns

R. Bjorhovde, Chairman	J. W. Clark	D. H. Hall
L. S. Beedle	J. L. Durkee	T. Pekoz
W. F. Chen	R. R. Graham*	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	Z. Razzaq
M. J. Abrahams	P. W. Marshall	B. C. Ringo
W. F. Chen	D. A. Nethercot	S. Vinnakota
S. L. Chin	S. U. Pillai	

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*	J. H. Daniels	L. W. Lu
P. F. Adams	W. E. Edwards	W. A. Milek, Jr.
C. Birnstiel	T. R. Higgins	C. K. Wang
F. Y. Cheng	I. M. Hooper	J. A. Yura
H. de Clercq		

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.

Memorandum #6.

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* Executive Committee Contact Member

Task Group 7 - Tapered Members (Joint Task Group with Welding Research Council)

A. Amirikian, Chairman	T. R. Higgins*	L. W. Lu
D. J. Butler	D. L. Johnson	C. J. Miller
C. R. Femley, Jr.	K. H. Koopman	F. J. Palmer
D. S. Ellifritt	G. C. Lee	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	S. M. Holzer	G. J. Simitses
J. Amazigo	B. G. Johnston*	J. C. Simonis
S. S. Chen		

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	A. Carpena	B. Kato
W. A. Milek, Jr., V. Chairman*	M. Crainicescu	P. Marek
G. A. Alpsten	T. V. Galambos	G. W. Schulz
L. S. Beedle	M. P. Gaus	J. Strating
	J. S. Iffland	L. Tall

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	A. L. Johnson	E. P. Popov
G. A. Alpsten	B. G. Johnston	M. Shinozuka
G. F. Fox*	L. W. Lu	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

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

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 and the fabrication processes.

Task Group 14 - Horizontally Curved Girders

, Chairman	C. G. Culver	W. A. Milek, Jr.
R. Behling	J. L. Durkee*	M. Ojalvo
H. R. Brannon	E. R. Latham	S. Shore
A. P. Cole	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 literally unsupported beams, including those in framed structures; and to determine bracing requirements for such beams.

Task Group 16 - Plate Girders

W. Hsiong, Chairman 熊瑋	R. S. Fountain	C. Massonnet
K. Basler	K. L. Heilman	A. Ostapenko
P. B. Cooper	B. G. Johnston	F. D. Sears
J. L. Durkee*	H. S. Lew	B. T. Yen
		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	M. Crainicescu	E. P. Popov
W. J. Austin*	A. L. Johnson	D. R. Sherman
L. O. Bass	A. Kalnins	J. C. Simonis
J. Bruegging	D. Krajcinovic	D. T. Wright
K. P. Buchert	C. Libove	

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	T. G. Johns
B. O. Almroth	S. L. Chin	J. N. MacAdams
M. D. Bernstein	J. W. Cox	P. W. Marshall
P. C. Birkemoe	E. D. George, Jr.	R. M. Meith
C. Capanoglu	R. R. Graham*	C. D. Miller

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

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

Task Group 21 - Box Girders

R. C. Young, Chairman	G. F. Fox*	D. Tung
J. H. Daniels	F. D. Sears	

Scope: To review, organize and interpret available information on the behavior of box girders, cooperating with other groups working on this subject; and to develop stability criteria as needed.

* Executive Committee Contact Member

Task Group 22 - Stiffened Tubular Members

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

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 Group 23 - Effect of End Restraint on Initially Crooked Columns 15-members

W. F. Chen, Chairman	T. V. Galambos	G. Winter *
R. Bjorhovde	J. S. B. Iffland	R. Zandonini
	J. Springfield	

Scope: To study the effect of end restraint on these isolated, hinged-end, initially crooked w-shape columns for which residual stress patterns are generally known.

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

* Executive Committee Contact Member

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L. S. Beedle	1976
E. L. Erickson	1976
E. H. Gaylord	1976
J. A. Gilligan	1976
J. E. Goldberg	1976
T. R. Higgins	1976
N. J. Hoff	1976
S. C. Hollister	1976
L. K. Irwin	1976
B. G. Johnston	1976
R. L. Ketter	1976
N. M. Newmark	1976
B. Thurlimann	1976
G. Winter	1976

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By-Laws*

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

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.

MEETINGS OF THE COUNCIL

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.

FISCAL YEAR

The fiscal year shall begin on October 1.

DUTIES OF THE COUNCIL

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.

OFFICERS OF THE COUNCIL

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.

C O N T R A C T S

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.

S T A N D I N G A N D S P E C I A L C O M M I T T E E S

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".

R E S E A R C H C O M M I T T E E S A N D T A S K G R O U P S

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 , May 16, 1977

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.

V. WORKING RELATIONSHIP BETWEEN EXECUTIVE COMMITTEE & TASK GROUPS

1. Executive Committee defines scope of task group assignment, selects task group chairman, and appoints Executive Committee contact member. SSRC Chairman sends letter of appointment to task group chairman and furnishes him with Statement of Scope, name of contact member, and procedural guidelines as appropriate.

2. Task group chairman can recommend changes to scope if he so desires.

3. Executive Committee recommends possible task group members, but task group chairman assembles his own list of prospects and determines their willingness to serve, and furnishes names to contact member.

4. Executive Committee approves task group members and SSRC Chairman notifies them of their appointment.

5. Task group should meet at least once a year to remain in good standing. SSRC Chairman shall make this point clear to task group chairman when he is appointed.

6. Suitably in advance of Annual Technical Session, SSRC Secretary shall send instructions to each task group chairman regarding expected participation of his task group.

7. Suitably in advance of each Executive Committee meeting, SSRC Secretary shall send Executive Committee agenda (and relevant EC meeting minutes as necessary) to each task group chairman, requesting him to send one-page report to his contact member covering the following matters (and others as appropriate):

   a. Task group progress.

   b. Status of research projects being supervised or advised by task group.

c. Task group meeting minutes.

d. Comments on relevant matters on EC agenda.

e. Membership status and recommended changes.

f. (*Prior to spring meeting of Executive Committee*) Task group plans for SSRC Annual Technical Session.

8. It is contact member's responsibility to check regularly with task group chairman regarding task group progress, and particularly with respect to his duties and plans in connection with: (a) holding of task group meetings; (b) reports to Executive Committee; and (c) planning for and participation in Annual Technical Session.

9. In the event task group chairman will not be present at Executive Committee meeting or at Annual Technical Session, contact member will present task group report, or (if he is unable to attend) he shall arrange for an alternate to report, consulting in advance with SSRC Chairman or Secretary as appropriate.

10. In general, SSRC Chairman commissions and furnishes all necessary instructions to task group, and contact member renders follow-up services. Thus, task group chairman is ultimately responsible to Executive Committee, not to contact member.

VI. GUIDELINES FOR SSRC TASK GROUP CHAIRMEN

1. Scope of Task Group Activities

Review the scope as approved by the Executive Committee and recommend changes if needed.

2. Task Group Membership

a. At the time the task group is formed, recommend task group membership to the Executive Committee. Task group members will be approved by the Executive Committee and notified by the SSRC Chairman.

b. Review the task group membership at least once each year (before the annual meeting) and recommend new members or changes in the membership to the Executive Committee.

c. Endeavor to insure that members are active participants in the task group activities.

3. Conduct of Business

a. Direct the activities of the task group in the work required to carry out the assignment defined in the task group scope.

b. Carry out other tasks as may be assigned by the Executive Committee.

c. Hold a meeting of the task group at least once each year.

4. Reporting of Task Group Activities

Submit a written report of task group activities to the Executive Committee before each Executive Committee meeting. The deadlines for the reports will be indicated to the task group chairman by correspondence from the SSRC secretary. Reports should cover:

1. Task group meeting minutes.

2. Status of research projects being supervised or advised by task group.

3. Membership status and recommended changes (before the annual meeting).

4. Other items of task group progress.

5. Comments on other SSRC activities, as appropriate.