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About the Council

The Structural Stability Research Council (formerly Column Research Council) was founded in 1944 to remove the confu-
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STRUCTURAL STABILITY RESEARCH COUNCIL

Established in 1944 by Engineering Foundation

Proceedings 1980

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

Conference supported by grants from
Federal Highway Administration/Urban Mass Transit Administration

Table of Contents

Foreword

Publication of this 1980 Proceedings of the Council commemorates completion of my second year as your Chairman. Thanks to the hard work and active participation of our members I believe that this has been a successful year.

The following five very important SSRC documents have been published or are scheduled to be published in the near future:

1. "Research Needs in Stability of Metal Structures", initially prepared by the Ad Hoc Committee on Research Priorities chaired by the writer with the final paper prepared by Reidar Bjorhovde and published in the ASCE Journal of the Structural Division, December, 1980.

2. Technical Memorandum No. 6, "Determination of Residual Stresses", prepared by Task Group 6 under the chairmanship of Teoman Pekoz and to be published in the Society of Experimental Stress Analysis Journal of Experimental Mechanics.

3. Technical Memorandum No. 5, "General Principles for the Stability Design of Metal Structures", prepared by the Ad Hoc Committee on Column Problems chaired by past SSRC Chairman Theodore V. Galambos and to be published in CIVIL ENGINEERING magazine.

4. "Stability of Metal Structures: A World View", prepared by Task Group 11 of SSRC in cooperation with European Convention for Constructional Steelwork, Column Research Committee of Japan and Council of Mutual Economic Assistance. This major document is the Comparison/Summary Report of the 2nd International Colloquium on Stability and will be published in four successive issues of the American Institute of Steel Construction's Engineering Journal in 1981. Thirty-five authors have contributed to this forerunner to an "International Guide". Regional Editors are Duiliu Sfintesco, Otto Halasz, Ben Kato and Theodore V. Galambos, representing four world regions; Western Europe, Eastern Europe, Japan and North America, respectively. This work also would not have been possible without the substantial assistance of Lynn S. Beedle, our Director, and Riccardo Zandonini, Sritawat Kitipornchai and M. Nuray Aydinoglu, our Technical Secretaries during 1978-1980 period.

5. "SSRC History", prepared by Bruce G. Johnston and scheduled for publication in the ASCE Journal of the Structural Division.

This year we had a successful Annual Technical Session and Meeting, concluding with a panel session arranged and chaired by Gerard F. Fox on the subject of "Bridge Stability Problems", thanks to the co-sponsorship of the Federal Highway Administration and the Urban Mass Transit Administration. The Council also cooperated in the 5th International Specialty Conference on Cold Formed Steel Structures and is cooperating in Council related sessions in both the Spring and Fall Annual ASCE Conferences in 1981.

Completed in 1980 or near completion are five research projects for which SSRC provided seed money support. These are:

1. "Influence of Imperfections on the Maximum Strength of Restrained Beam-Columns", by Sriramula Vinnakota.

2. "The Effect of the Material Damage on the Buckling of Structures", by Dusan Krajcinovic.

3. "Criteria, Analysis and Design of Braced & Unbraced Frames", by Mrinmay Biswas and Rodney Earwood.

4. "Investigation of Post-Buckling Behaviour", by Alexander Chajes.

5. "Preliminary Investigation of Partially Restrained Columns", by Zia Razzaq.

Next year we look forward to publication of the results of these investigations and to completion of the first draft of the 4th Edition of the "Guide" under the direction and through the efforts of the Editor. Theodore V. Galambos. 1981 may also see substantial progress in the work of the Bibliography Committee chaired by Jackson L. Durkee.

Finally, progress was made in 1980 in the increase in the number of Participating Firms supporting the Council. Last year the number of Participating Firms grew from 25 to 40. Our thanks are warmly given to [illegible] of them for their support.

As Chairman, my job has been made especially easy this past year by cooperation and assistance of our Headquarters staff, Lynn S. Beedle, Di[illegible] M. Nuray Aydinoglu, Technical Secretary and Lesleigh G. Federinic, Admini[illegible] Secretary. To them I express my appreciation and give them my sincere [illegible] for a successful second year.

Jerome S. B. Iffland

Jerome S.B. Iffland, Chairman
Structural Stability Research Council
New York, New York
1980

SSRC Executive Committee

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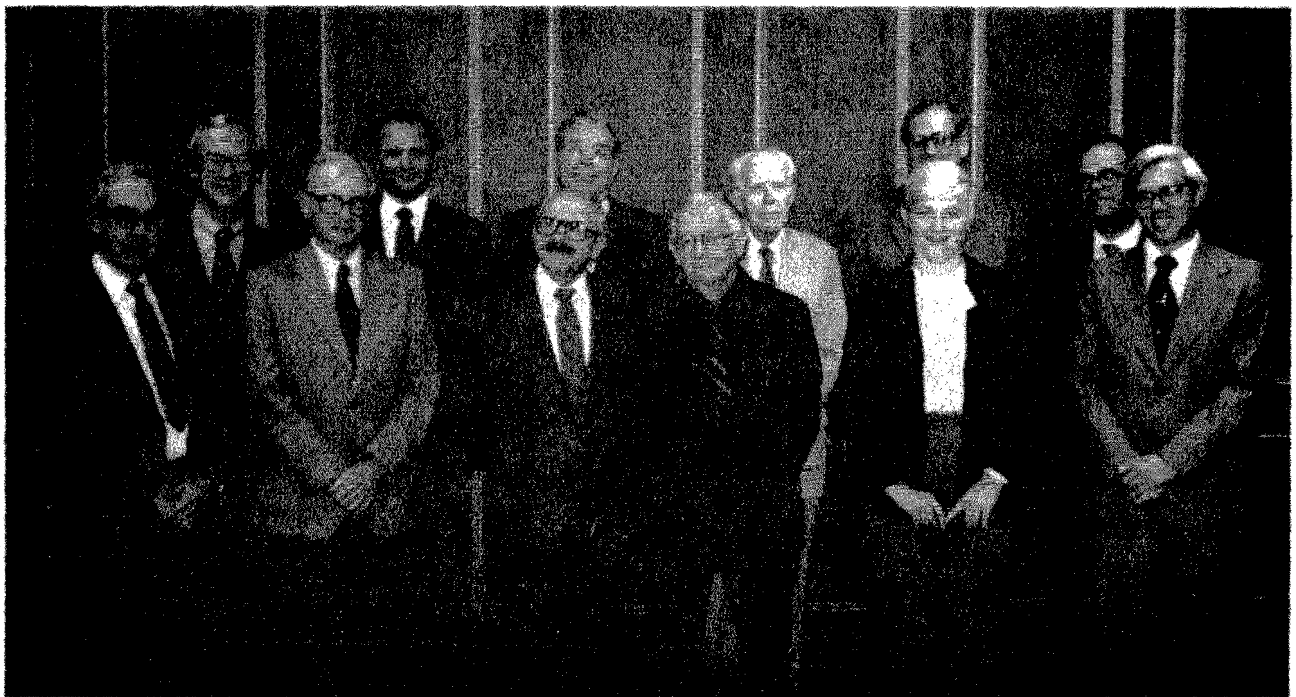


Back Row - J. Springfield, R. M. Meith, W. J. Austin, T. R. Higgins, J. S. B. Iffland, M. N. Aydinoglu
Front Row - R. R. Graham, G. Winter, T. V. Galambos, B. G. Johnston, G. F. Fox and S. J. Errera

Annual Technical Session

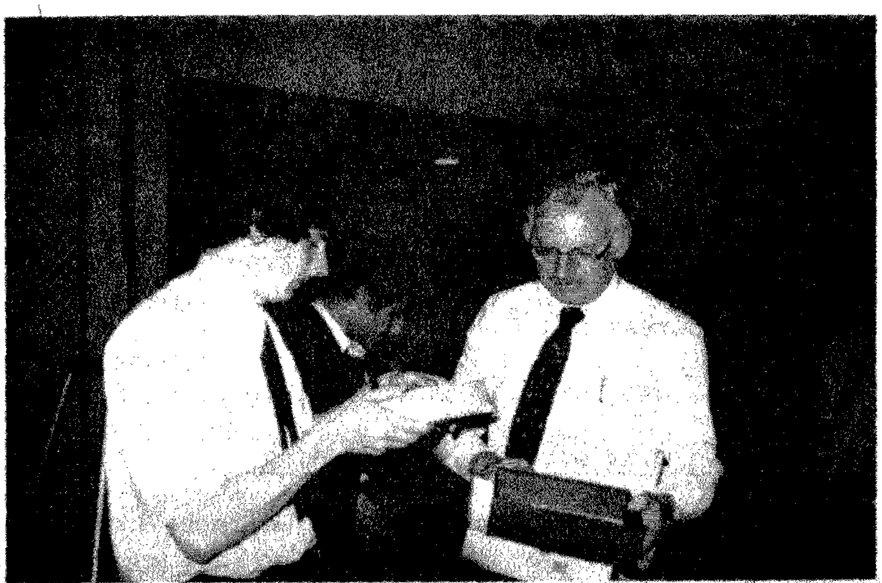
One of the purposes of the Council is to maintain a forum where structural stability aspects of the behavior of frames, columns and other compression-type elements in metal and compression structures can be presented for evaluation and discussion. The Annual Technical Session provides an opportunity to carry out this function.

The 1980 Annual Technical Session was held on April 29 and 30 at The New York Sheraton Hotel in New York, New York. Ninety-three persons attended the Session and forty papers were delivered.

A panel discussion on "Bridge Stability Problems" was held in the evening of April 29, 1980. The panelists were Bernard P. Wex, Heinze Noelke, and Herbert B. Rothman. The moderator was G. F. Fox.

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

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



PROGRAM OF TECHNICAL SESSION

Tuesday, April 29, 1980

7:30 a.m. - REGISTRATION

8:30 a.m. - MORNING SESSION

Presiding: J. L. Durkee, Consulting Structural Engineer

INTRODUCTION

J. S. B. Iffland, Chairman, SSRC

Task Group 1 - Centrally Loaded Columns

Chairman, R. Bjorhovde, The University of Alberta

Ultimate Displacement - Strain Design Approach for Compression Members with Imperfections

J. J. Melcher, Technical University of Brno

The Equations of Buckling for a Thin-Walled Column with a Concentric Axial Load

M. Ojalvo, The Ohio State University

Inter-Connection of Starred Angle Compression Members

M. C. Temple and A. J. Schepers, University of Windsor

Classification of Structural Steel Members Initial Imperfections

J. J. Melcher, Technical University of Brno

Multiple Column Curves and Codes

L. Finzi and R. Zandonini, Technical University of Milan

Task Group 3 - Columns with Biaxial Bending

Chairman, J. Springfield, Carruthers & Wallace, Ltd.

Reliability of Aluminum Beam-Columns

J. Chapuis and T. V. Galambos, Washington University

PROGRAM

Nonsway Columns with Biaxial Partial Restraints

C. H. Caro, H. N. Mohamed-Aly, and Z. Razzaq, University of Notre Dame

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

Chairman, R. B. Testa, Columbia University

10:15-10:35 a.m. - BREAK

Task Group 15 - Laterally Unsupported Beams

Chairman, J. A. Yura, University of Texas

Lateral Stability of Roof Truss Systems

G. Masoumy and T. V. Galambos, Washington University

Combined Stiffness in Beam and Column Braces

C. O'Connor, University of Queensland

Approximate Elastic Lateral-Torsional Buckling Formulas for Continuous Beams

J. Krpan and Z. Razzaq, University of Notre Dame

Task Group 6 - Test Methods for Compression Members

Chairman, T. Pekoz, Cornell University

Task Group 17 - Stability of Shell-Like Structures

Chairman, A, Chajes, University of Massachusetts

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

Chairman, W. F. Chen,Purdue University

End Restraint and Initial Imperfection in Aluminum Columns

T. V. Galambos and J. Chapuis, Washington University

Initially Crooked Columns with Partial Restraints

Z. Razzaq, J. G. Chang, and P. K. Krueger, University of Notre Dame

12:00 Noon - GROUP LUNCHEON

1:00 p. m. - AFTERNOON SESSION

Presiding: B. G. Johnston, Consultant

P R O G R A M

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

P.-Delta Effect in Column Design

T. J. Downs, Bakke Kopp Ballou & McFarlin and F. Cheong-Siat-Moy, University of Minnesota

Task Group 11 - International Cooperation on Stability Studies

Chairman, D. Sfintesco, Lamorlaye, France
Vice-Chairman, W. A. Milek, Jr., American Institute of Steel Construction

Task Group 8 - Dynamic Stability of Compression Elements

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

Parametric Excitation of Structures

D. Krajcinovic, University of Illinois at Chicago Circle

Dynamic Stability of Shallow Elastic Arches and Shells

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

Flow Induced Instabilities of Circular Cylindrical Structures

S. S. Chen, Argonne National Laboratory

Post Critical Behavior of the Pflüger Problem

M. S. El Naschie and S. AlAthel, University of Riyadh

Importance of Eccentric Valves on the Stability of Fluid Conveying Pipes

M. A. J. G. da Silva, University of Illinois at Chicago Circle

A Parametric Study of Instability Effect on Seismically Excitated Structure-Soil Systems

M. N. Aydinoglu and L. W. Lu, Lehigh University

Task Group 18 - Unstiffened Tubular Members

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

2:35-2:55 p. m. - BREAK

Task Group 14 - Horizontally Curved Girders

Chariman, M. Ojalvo, The Ohio State University

Stability of Curved Girders

C. H. Yoo and P. A. Pfeiffer, Marquette University

Task Group 22 - Stiffened Cylindrical Members

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

External Pressure Tests of Ring Stiffened Fabricated Cylinders

R. K. Kinra, Shell Oil Co., and C. D. Miller, Chicago Bridge and Iron Co.

Task Group 20 - Composite Members

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

Plastic Design with Noncompact Sections Including Composite Bridge Members

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

Task Group 16 - Plate Girders

Chairman, W. Hsiong, MTA Incorporated

Interaction Effect of Stresses on Plate Collapse

J. E. Harding and P. J. Dowling, Imperial College of Science & Technology

Shear Strength of Longitudinally Stiffened Plate Girders

A. Ostapenko, Lehigh University

Buckling Stresses of Web Panels in Girders

Y. S. Chen and B. T. Yen, Lehigh University

The Collapse of Stiffened Shear Webs

C. Marsh, Concordia University

4:35 p. m. - SOCIAL HOUR

6:00 p. m. - PANEL DISCUSSION : BRIDGE STABILITY PROBLEMS

Moderator: G. F. Fox, Howard Needles Tammen & Bergendoff

Panelists: Bernard P. Wex, Freeman Fox & Partners
Heinze Noelke, Technische Universitat Hannover
Herbert B. Rothman, Weidlinger Associates

8:00 p. m. - ADJOURN

Wednesday, April 30, 1980

8:30 a. m. - MORNING SESSION

Presiding: L. S. Beedle, Lehigh University

Task Group 4 - Frame Stability and Effective Column Length

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

Nonprincipal Axis Bracing of Centrally Loaded Columns

M. Ojalvo, T. A. Bolte and V. Faridani, The Ohio State University

Second-Order Elastic-Plastic Analysis of Steel Frames

C. K. Wang and C. P. Talaboc, University of Wisconsin

Computing and Using Effective Length in Portal Frame Design

P. Grundy, Monash University

Evaluation of Frame Systems Based on Optimality Criteria with Performance Constraints and P-Delta Effect

F. Y. Cheng, University of Missouri-Rolla

Task Group 21 - Box Girders

Chairman, R. C. Young, Frank E. Basil, Inc.

Stability of Cable Stayed Box Girders

M. C. Tang, DRC Consultants

Task Group 7 - Tapered Members

Chairman, A. Amirikian, Amirikian Engineering Co.

Design of Single-Story Rigid Frames Consisting of Tapered Members

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

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

Task Group 13 - Thin-Walled Metal Construction

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

Load and Resistance Factor Design of Cold-Formed Steel Structural Members

W. W. Yu and B. Supornsilaphachai, University of Missouri-Rolla and T. V. Galambos, Washington University

The Strength of Cold-Formed Steel Columns

D. T. Dat, Exxon Production Research and T. Pekoz, Cornell University

Task Reporter 14 - Fire Effects on Structural Stability

K. Klippstein, U. S. Steel Corporation

A Report on the Effects of Fire on the Stability of Steel Columns

Task Reporter 15 - Curved Compression Members

W. J. Austin, Rice University

Inplane Stability of Parabolic Arches

H. B. Harrison, University of Sydney

An Ultimate Strength Formula for Braced Twin Arches

T. Sakimoto and S. Komatsu, The Ohio State University

Other Research Reports

Buckling of Rail-Road Tracks

A. Chajes, University of Massachusetts

Lateral-Torsional Buckling of T-Section Steel Beam-Columns

Shao-Fan Chen, Xian Institute of Metallurgy and Constructional Engineering

11:37 a. m. - END SESSION

11:40 a. m. - SSRC ANNUAL BUSINESS METTING

ADJOURN

Robert F. Varney
Federal Highway Administration
Representative

Jerome S. B Iffland
SSRC Chairman

TASK GROUP REPORTS

TASK GROUP 1 - CENTRALLY LOADED COLUMNS

Chairman, R. Bjorhovde, The University of Alberta

Ultimate Displacement-Strain Design Approach for Compression Members with Imperfections

J. J. Melcher, Technical University of Brno

It is well-known that from the beginning of real member behaviour an increase in loading causes an increase in displacement. Looking for the answer to the question of specified design load determination in the form of LRFD approach, we have to realize that the limit state is not any fiction, but actually possible state of the structure behaviour. Consequently, it must be acceptable regarding both ultimate design load and the corresponding displacements (besides the separate serviceability condition).

From our tests performed during 1978 - 1979 follows, for example, that the maximum values of the real final ultimate strength deflections of compression steel pipes were as much as L/167 (welded pipes) and L/86 (seamless pipes). Such large displacements at the limit state of structure are entirely unacceptable. It was proved that if we demand small limit values of transverse deflections (for example under L/500) the corresponding limit load carrying capacity can be less than the specified buckling strength.

In general we cannot accept the ultimate strength as the only limit loading level for determination of safe design load. It should be prescribed by the ultimate displacement approach: for the acceptable ultimate displacement or strain we determine the corresponding ultimate specified load inversely using the load-displacement relationship.

The Equations of Buckling for a Thin-Walled Column with a Concentric Axial Load

M. Ojalvo, The Ohio State University

A review of the literature on concentrically loaded thin-walled columns discloses a general agreement as to the governing equations, yet results based on them give at least one unexpected and disquieting result.

When the centroid and the shear center coincide, the theory may be used to yield two distinct results for the buckling stress. Both involve a condition of torsional buckling without flexure.

$$\sigma = \frac{GJ}{I_p} \tag{1}$$

For the first and lower value the variation of the longitudinal rotation, β, is arbitrary, while for the second, β has a sine wave longitudinal variation. It has been argued that the higher buckling load is in fact the only one possible because of the presence of unavoidable initial imperfection. This explanation runs counter to normal expectations.

The three equations have been rederived using two distinct methods - the method of stationary potential energy employing the calculus of variations and the strength of materials method. Both lead to the same equations, but the equations differ from the accepted in one term appearing in what is considered to be the third of the three equations. The new equation is:

$$EC_w \beta^{iv} + (P r_o^2 - GJ) \beta'' + Py_o u'' - Px_o v'' = 0 \qquad (3)$$

The term r_o is the distance between the centroid and the shear center of the cross section. In the generally accepted form of these equations, $P \frac{I_p}{A}$ appears for $P r_o^2$ in Eq. (3). The difference works significant changes on predicted buckling loads.

Pure torsional buckling is not possible under the new theory, and the importance of coupled torsional-flexural buckling is diminished.

The new derivation differs from the old in that δ of the $-P\delta$ term in the expression for the potential energy is determined for the longitudinal element which is the locus of centroids. In the usual derivation $-\iint \sigma \, \delta \, dA$ is used where δ is for a longitudinal element associated with dA.

The approach used in the writer's derivation is shown to be based on a sounder analysis of the behavior of beams and columns during buckling.

The development reported here should significantly impact on the theories of beam-columns and beams where, in recent times, refinements embodying the so-called Wagner effect of the longitudinal stresses have been incorporated.

Inter-Connection of Starred Angle Compression Members

M. C. Temple and A. J. Schepers, University of Windsor

At the SSRC Annual Technical Session held in Pittsburgh in April 1979 a paper was presented outlining the great variety of specifications with regard to the inter-connection of starred angle compression members (see p. 17 of SSRC Proceedings 1979). The results of preliminary tests were also given.

This report summarizes the theoretical and experimental results obtained since that time.

The starred angle compression member has been modelled for a finite element analysis. This model is used to determine the critical load by an eigenvalue approach and for studying the effects of out-of-straightness by a nonlinear incremental approach. Where applicable, the results were compared to those obtained by Johnston for Spaced Steel Columns (ASCE, J. Struct. Div., Vol. 97, No. ST5, May 1971, pp. 1465-1479).

The Canadian Code and American Specifications for the inter-connection of starred angle compression members require the use of only one inter-connector. It has been found that with one inter-connector the individual angles buckle about their weak axis with some end fixity. With two or more inter-connectors the angles buckle about the weak axis of the entire cross-section. The latter results in a higher buckling load. The loads given in the Limit States Design Steel Manual (CISC, 1977) are based on a mode of buckling that results in this higher buckling load. To obtain this higher load, two inter-connectors are required, as opposed to the one required by the Canadian and American Codes.

Classification of Structural Steel Members Initial Imperfections

J. J. Melcher, Technical University of Brno

The initial imperfections can be divided into three primary groups:

1. Geometrical imperfections,

2. Internal structural imperfections:

   - dispersion of mechanical properties,

   - initial stress state,

3. Construction imperfections: deviations and imperfections in the fabrication of connections, joints, splices, gussets, column bases and caps, bearing supports and other constructional details that result in the behaviour deviation of real structure in comparison with the ideal boundary assumption used in practical design procedure.

At this time much is known about the compression member behaviour from the point of view just of the 1st and 2nd group of imperfections. It is necessary to emphasize that all these results refer to an isolated member (lab-member) separated from the constructional system.

It is evident that the tests on real structural system with compression members should give another picture about the significance of imperfections from the results for lab-member.

Moreover, there are further problems in the direct application of the multiple column curve concept in practical designing: the well-known proposal is based on the only first loading cycle; there is no test data for flexural-torsional buckling and the beam-column problems, also for built-up and non-prismatic members and also for a large number of grades of steels.

The absolute values of imperfections substantially depand on the character, behaviour and type of the constructional system and without consideration of these aspects it is not proper to derive the results for practical designing of real compression members (if we decide to accept the multiple column curve proposal, in general).

Multiple Column Curves and Codes

L. Finzi and R. Zandonini, *Technical University* of Milan, Italy

In the last decade, multiple column curves (up to five) have been adopted or recommended in Europe (ECCS) as well as in the U.S.A. (SSRC). However, there has been a certain resistance to the introduction of this approach in National Codes, mainly because it leads to a more complex, boring and sometimes uncertain procedure.

We think that this approach is a useful foundation for anyone involved in drafting National Codes, but that modifications are needed in order to reduce and simplify the work of practitioners.

With reference to a previous paper presented at the Second Stability Colloquium - Washington 1977, new ideas and proposals to overcome the complexity of the multiple column curve approach are presented here.

The basic idea is still the one to refer all the column curves to a single one, chosen as reference curve; this curve would be the only one to be introduced in a code.

As reference curves the ECCS A_0 and the SSRC curve 1 have been adopted respectively; this because these curves practically depend only on the initial standard out-of-straightness, while the influence of residual stresses and yield point scatter are disregarded.

The values of the increase $\Delta\bar{\lambda}$ of the nondimensional slenderness which, for a given value of the buckling stress σ_K, leads from a buckling curve to the reference curve have been plotted first. The range of $\bar{\lambda}$ between 0.5 and 1.6 has been considered, because it covers all the practical cases of compressed members. It is possible to fit well these curves with polynomial expressions, but it would lead to a too-complex design procedure; a bilinear approximation has then been selected as indicated in fig. 1.

Considering the second line, $\Delta\bar{\lambda}_2$, as equal to the difference $\Delta\bar{\lambda}_1 - \Delta\bar{\lambda}_3$, the formulas presented in fig. 2 can be obtained. That means that, for designing purposes, it needs only a modified radius of inertia r* if $\bar{\lambda}$ is less than 0.5 and, additionally, the coefficient K_λ of the linear correction, term when $\bar{\lambda}$ ranges between 0.5 and 1.6. The values of r* and K_λ , defined with a least square procedure, are presented in the same figure, together with the maximum relative error ε which would be made in evaluating a curve through the suggested approach.

The values of r* and K_λ can be easily implemented in a usual table giving the "Properties for Designing" making the approach "ready to use" for practitioners.

It has also been studied an alternative approach which consider directly curves giving the value of the modified radius r* as function of $\bar{\lambda}$.

Several approaches, less or more sophisticated, have been set up starting from these curves; in the simplest one the modified radius is taken constant to $\bar{\lambda}$ = 0.5, then varies linearly to $\bar{\lambda}$ = 1.2 and is again constant from $\bar{\lambda}$ = 1.2 to $\bar{\lambda}$ = 1.6 (see fig. 3).

TG-1 cont'd

The values 0.5 and 1.2 have been selected, not only on the basis of an optimization process, but because their practical meaning: in fact members of multistory frames have frequently slenderness less than 0.5 so many truss web members have slenderness greater than 1.2.

Figure 4 gives the design formula and the values of $r^*_{0.5}$, $r^*_{1.2}$ and of the maximum relative error.

It is evident that this method gives less accurate results, but it seems preferable for its simplicity and immediateness of its physical meaning. Also in this case the easy implementation in "Properties for Designing" table makes the approach "ready" for practical purposes.

We hope that this proposal can lead to codes oriented toward safe and economic design without sacrificing simplicity.

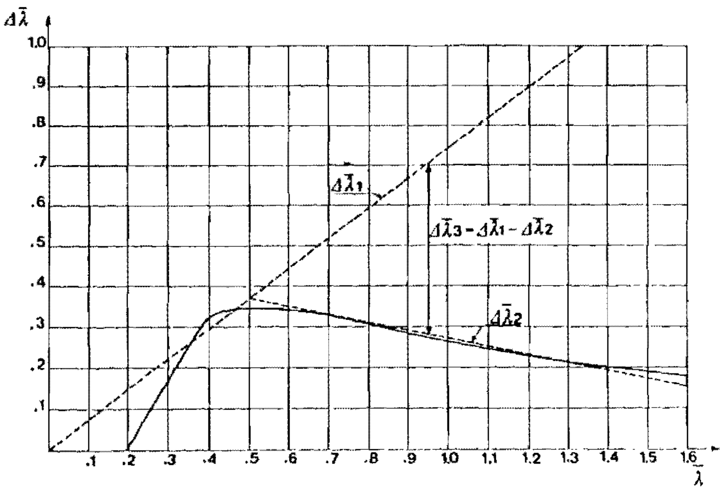


Figure 1

DESIGN FORMULAS

$\lambda^* = L/r^*$ for $\bar{\lambda} \leq 0.5$

$\lambda^* = L/r^* - K_\lambda \left(\frac{L - L_{0.5}\sqrt{235/\sigma_r}}{r^*} \right)$ for $0.5 \leq \bar{\lambda} \leq 1.6$

$r^* = \alpha r$; $L_{0.5} = 0.5\, r \pi \sqrt{E/235}$

CURVE		r^*	K_λ	$\lvert\varepsilon\ \max\rvert$ %
ECCS	A	0.763 r	0.336	2.5
	B	0.654 r	0.445	3.3
	C	0.578 r	0.532	2.8
	D	0.508 r	0.599	3.8
SSRC	2	0.655 r	0.435	1.6
	3	0.514 r	0.584	3.7

Figure 2

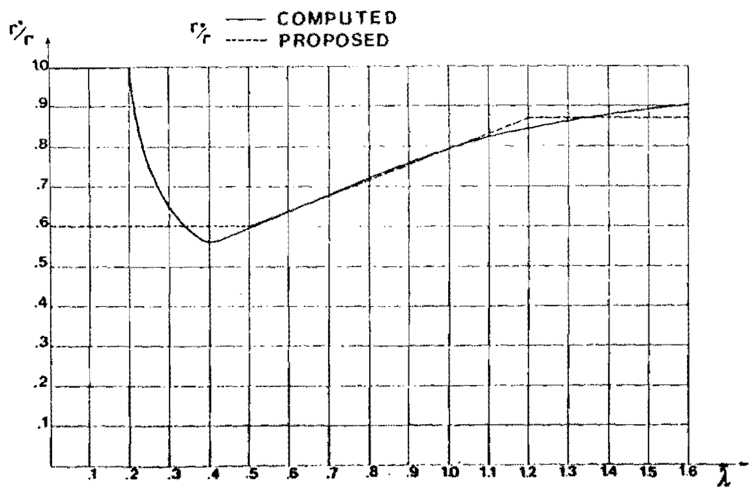


Figure 3

DESIGN FORMULAS

$\lambda^* = L/r^*$; $r^* = \alpha r$

$$r^* \begin{cases} = r^*_{0.5} & \text{for } \bar{\lambda} \leq 0.5 \\ = r^*_{0.5} + (\bar{\lambda} - 0.5)\dfrac{r^*_{1.2} - r^*_{0.5}}{0.7} & \text{for } 0.5 \leq \bar{\lambda} \leq 1.2 \\ = r^*_{1.2} & \text{for } 1.2 \leq \bar{\lambda} \leq 1.6 \end{cases}$$

CURVE		$r^*_{0.5}$	$r^*_{1.2}$	$\lvert\varepsilon\ \max\rvert$ %
ECCS	A	0.784 r	0.967 r	2.50
	B	0.677 r	0.908 r	4.06
	C	0.601 r	0.869 r	6.15
	D	0.530 r	0.815 r	6.21
SSRC	2	0.684 r	0.901 r	4.99
	3	0.537 r	0.811 r	8.64

Figure 4

TASK GROUP 3 - COLUMNS WITH BIAXIAL BENDING

Chairman, J. Springfield, Carruthers & Wallace, Ltd.

Reliability of Aluminum Beam-Columns

J. Chapuis and T. V. Galambos, Washington University

This paper first briefly describes various First-Order Second Moment approaches used in determining the reliability index β of non-linear design criteria such as interaction curves for beam-columns. Next the ultimate strength of aluminum beam-columns is discussed as it is affected by the type of limit state, the material non-linearity, the type of section, the attitude of flexure and the initial imperfection. Finally the reliability index β is determined for various types of interaction equations, and some comments are made on the use of this information in the development of Load and Resistance Factor Design criteria for aluminum beam-columns.

Nonsway Columns with Biaxial Partial Restraints

C. H. Caro, H. N. Mohamed-Aly, and Z. Razzaq, University of Notre Dame

Using the Rayleigh-Ritz approach and some suitable deflection functions, approximate formulas for computing the buckling load of a perfect elastic column with biaxial partial end-restraints are developed. Only nonsway columns are considered. The rotational restraints are linear but unequal at both ends of the column. Approximate load versus deflection curves for crooked columns with similar restraints are being developed. The results will also be used to check the solutions obtained from the finite-difference analysis of the differential equations of equilibrium to be used eventually in an inelastic analysis of the column.

The problem of columns with biaxial partial restraints seems to encompass the subject matters of both Task Groups 3 and 23; however, due to the "biaxiality" of the problem, it probably would better fit under the former group.

TASK GROUP 15 - LATERALLY UNSUPPORTED BEAMS

Chairman, J. A. Yura, University of Texas

Lateral Stability of Roof Truss Systems

Gholam Masoumy and T. V. Galambos, Washington University

This paper deals with the lateral stability of steel joist roof truss systems during construction when the bridging lines are in place but before the permanent roofing provides full lateral stability. The paper first discusses the lateral stability of a single truss, treating the truss not as an equivalent beam but as a spatial assembly of laterally and torsionally restrained prismatic members. The results are compared with experimental data and previous analyses. Next, the stability of a roof with multiple trusses connected by bridging lines is discussed. Various configurations are studied and general conclusions are given. Finally, practical design recommendations based on the stability analysis are given.

Combined Stiffness in Beam and Column Braces

Colin O'Connor, University of Queensland

For many simple braced columns and beams, the critical load increases with brace stiffness until a particular stiffness is reached, and then remains constant. The member is said to be fully braced. The brace stiffness for full bracing - although often readily supplied by any practical bracing member - is of major importance to the designer.

A brace generally has three stiffnesses - its axial stiffness and two bending stiffnesses. Published analyses which allow for all three stiffnesses are rare. Many authors have presented analyses which allow for axial and out-of-plane bending stiffness. Although these papers present typical results, it is difficult to provide sufficient design information for the two-stiffness case in numerical form.

Consider a bi-symmetric I-beam with a central brace. A single brace without bending stiffness may be insufficient to achieve full bracing even with an infinite axial stiffness - for example, for the case of an axially loaded member with a brace at one flange. In many cases of practical interest, particularly when the member is subject to a number of load cases, it is necessary to brace the member either by a brace to each flange, or by a brace with both axial and bending stiffness. The designer requires acceptable combination of axial and bending stiffness for full bracing.

A desire for closed form, rather than numerical, solutions has led to the development of an approximate analysis. The I-beam is replaced by two members, one for each flange, joined by a system of rigid transverse links which ensure that differential transverse deflections of the flanges lead to twist rotations, and St. Venant torques.

TG-15 cont'd

Typical requirements for full bracing are given below and shown in Fig. 1 - for a member with an eccentric axial load. The terminology is shown below the figure.

Brace at centroid

$$\frac{k_R}{h^2 k_0} = 0.25 - \frac{1}{3}\ (2a_1 + B) \tag{1}$$

Brace located midway between centroid and flange

$$\frac{k_R}{h^2 k_0} = 0.3125 - \frac{1}{3}\ (2a_1 + B) + \frac{1}{16(\frac{k_L}{k_0} - 1)} \tag{2}$$

Brace at flange

$$\frac{k_R}{h^2 k_0} = 0.5 - \frac{1}{3}\ (2a_1 + B) + \frac{1}{4(\frac{k_L}{k_0} - 1)} \tag{3}$$

The upper two curves are asymptotic to points A, and B_2 or B_3. The coefficients, 16 or 4, control the sharpness of the curves between the asymptotes.

Point A represents the value of k_L which is required to be provided in association with infinite k_R.

Points B_1, B_2 represent the values of k_R required in association with infinite k_L.

B_1, B_2 and B_3 have vertical coordinates which may be positive or negative. If positive, then k_R is required for all k_L.

These curves of k_L and k_R for full bracing are fully defined by three parameters -

(a) the horizontal coordinate of A;

(b) the vertical coordinate of B; and

(c) the parameter - e.g., 16 or 4 - controlling the sharpness of the curves.

It appears that this conclusion is generally valid for uniformly loaded members. Parameters (a) and (b) have a clear physical significance. Parameter (c) needs to be determined for particular cases.

TG-15 cont'd

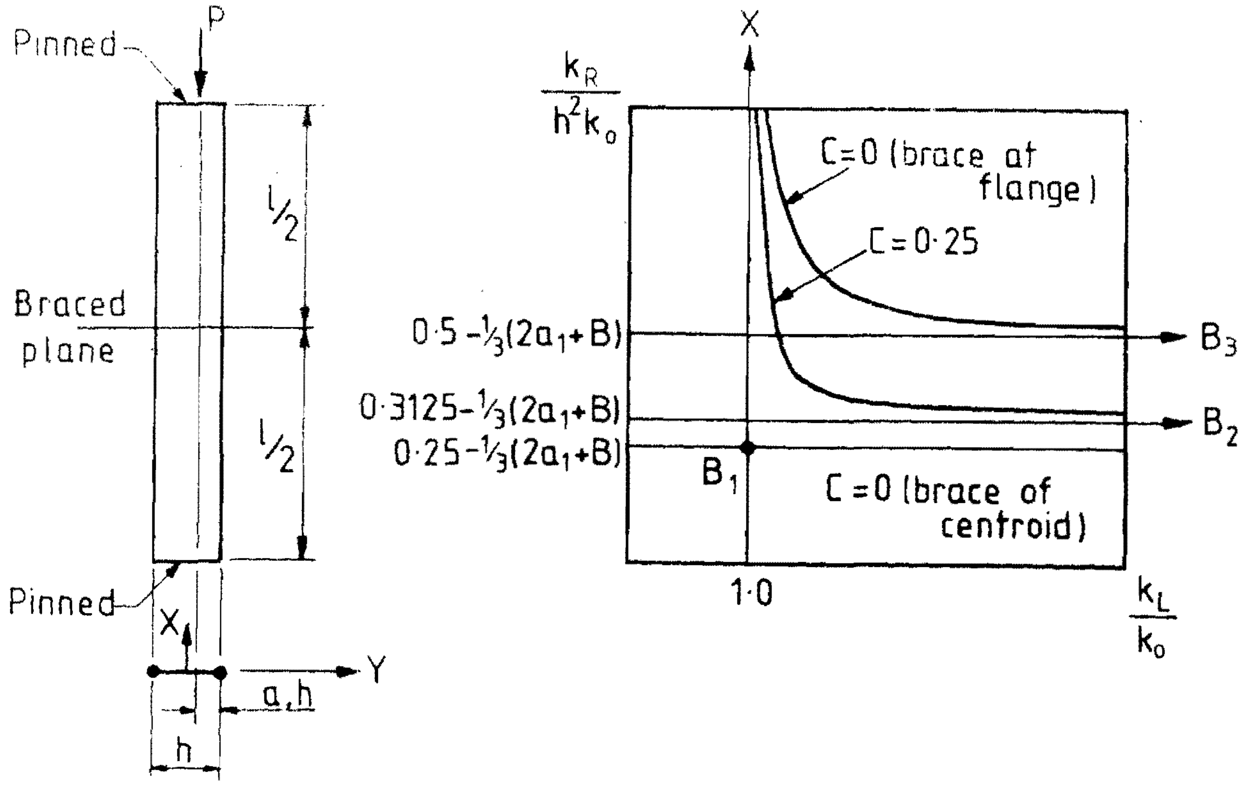


FIGURE 1 Stiffness for full bracing – member with eccentric axial load

Approximate Elastic Lateral-Torsional Buckling Formulas for Continuous Beams

J. Krpan and Z. Razzaq, University of Notre Dame

A theoretical study of the lateral-torsional buckling behavior of continuous beams is conducted. Based on suitable assumed deflection functions, it is shown that it is possible to arrive at approximate buckling load formulas for some special cases of loading and member geometry using the Rayleigh-Ritz technique. For the general case, a finite element program is written based on suitable interpolation functions and the virtual work approach including the effects of elastic foundation and axial load. The results obtained are compared to limited experimental results available in the literature. With the aid of the finite element program, a study is conducted of the various parameters influencing the lateral-torsional buckling of elastic continuous members. The advantages and disadvantages of the analytical tools employed are pointed out.

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

Chairman, W. F. Chen, Purdue University

End Restraint and Initial Imperfection in Aluminum Columns

T. V. Galambos and J. Chapuis, Washington University

This paper reviews first the ultimate strength of axially loaded aluminum columns as this is affected by material non-linearity, type of section, initial imperfection and end restraint. Conclusions are then drawn with regard to the design of end-restrained initially crooked aluminum columns. The role of the applicability of the effective length concept in design is discussed.

Initially Crooked Columns with Partial Restraints

Z. Razzaq, J. G. Chang, and P. K. Krueger, University of Notre Dame

The results of a theoretical investigation of perfect as well as initially crooked statically loaded planar elastic nonsway columns with unequal linear or elastic-plastic partial rotational end-restraints are presented. Approximate buckling load formulas accurate only for certain ranges of input parameters are developed for a perfect elastic nonsway column using the Galerkin technique and assuming linear moment-rotation characteristics for the end-restraints. The elastic stability of columns having an initial half sine wave imperfection and unequal elastic-plastic rotational restraints is also studied; computer programs which generate load versus deflection curves for these columns are developed using both the exact and finite difference solutions of the governing differential equation. The influence of the nature of the end-restraints and of the initial crookedness on the column instability load is investigated. Several conclusions relative to these parameters and to the accuracy of the approximate buckling load formulas are drawn. Included is a comparison to results obtained using the buckling load formulas to those based on the well-known K-factor alignment charts.

Work sponsored by SSRC on the inelastic behavior of partially restrained imperfect columns is in progress. Finite difference approach is being used to study the behavior of inelastic columns with nonlinear equal end-restraints. Once this problem is analyzed, the program will be extended to include unequal nonlinear end-restraints.

P-Delta Effect in Column Design

T. J. Downs, Bakke Kopp Ballou & McFarlin and F. Cheong-Siat-Moy, University of Minnesota

A summary of this paper is not available.

TASK GROUP 8 - DYNAMIC STABILITY OF COMPRESSION ELEMENTS

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

Parametric Excitation of Structures

D. Krajcinovic, University of Illinois at Chicago Circle

The intent of this presentation is to review the most important aspects of the parametric excitation phenomenon with a specific emphasis on the practical problems common to structural engineering. In fact the presentation should serve as an initial attempt to identify the part of the theory which is of definite interest in practical design. The entire process of casting the theory into a form directly usable in design is a novel undertaking which will take a long time and a combined effort of the entire Task Group.

For convenience, the physical phenomenon is initially discussed on a single degree-of-freedom system. The determination of the boundaries of the unstable regions in the frequency space is briefly reviewed for several different types of the load-time histories. According to the linear theory, the amplitude of a system in parametric resonance grows exponentially. The linear theory is sufficiently accurate in modelling the motion while the amplitudes are small. In resonance conditions, though, it becomes necessary to introduce the nonlinear terms which in general contribute to the modification (usually reduction) of the unstable regions and limit the amplitude within the unstable region. The influence of the nonlinear damping, stiffness and inertia is mentioned.

The additional insight into the phenomenon is gained studying the multi degree-of-freedom systems. In particular, the multi degree-of-freedom systems are necessary to study the combination and autoparametric resonance, as well as the flow of energy between the different modes of vibration.

Subsequently, the presentation reviews some of the investigations of the columns and beams modelled as continuous elastic systems. In particular, the review focuses on the influence of:

- the various boundary (end) conditions (specifically on the occurrence of the combination resonance),
- the torsional-bending instability in case of the thin-walled structures,
- the rotatory inertia,
- the imperfections, etc.

The presentation is concluded with a brief recital of the most important aspects of the nonstationary and stochastic problems.

Dynamic Stability of Shallow Elastic Arches and Shells

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

Recent work on the snap-through instability of shallow elastic structures under dynamic loading is reviewed. Most of the investigations cited involve either circular arches or spherical shells. The loads are usually applied uniformly over the structure, although some papers examine the effect of asymmetric components in the loading distribution. Timewise, a variety of loading applications are considered: impulse loads, step loads with infinite duration, triangular pulse loads, N-shaped pulse loads, and sine-wave pulse loads. A number of techniques are utilized to discretize the problems over space and time. Various definitions of critical loading are employed in the literature, including the Budiansky-Roth criterion and Hsu's sufficient condition.

The influence of damping (external and internal) on the critical load is examined by several authors. Attention is also given to the effect of imperfections (such as initial displacements). A comparison of critical loads for various boundary conditions is presented in one study. Several papers compare results obtained by different computer programs. Finally, some problems involving multiple, independent, dynamic loads are described, in which the results are presented as dynamic interaction curves.

Flow-Induced Instabilities of Circular Cylindrical Structures

S. S. Chen, Argonne National Laboratory

Many structural and mechanical components are containing fluid, subjected to external flow, or conveying fluid; therefore, they are susceptible to flow-induced instability. In fact, many flow-induced instability problems have been encountered in important system components. Recently, extensive studies of these problems have been made; this is brought about by the development of advanced nuclear power reactors, the use of high strength materials, and the intrinsically interesting characteristics of structural/fluid interaction.

The objective of this paper is to review the instability of circular cylindrical structures subjected to fluid flow. Included in this review are:

1. Stability of a single rod or multiple rods subjected to axial flow;
2. Buckling, flutter, and parametric resonance of tubes conveying fluid;
3. Vortex excited oscillations of a single cylinder in cross flow;
4. Wake-induced flutter of twin tubes;
5. Fluidelastic instability of tube array in cross flow; and
6. Dynamic instabilities of circular cylindrical shells subjected to fluid flow.

In each case, available analytical results and experimental data will be reviewed. Methods to predict the critical flow velocity will be presented. A brief design guide to avoid detrimental flow-induced instability, including simple formulas, figures and tables, will be discussed.

Post Critical Behavior of the Pflüger Problem

M. S. El Naschie and S. Al Athel, University of Riyadh

A global analysis of post critical behaviour of non-conservative systems (including that under follower forces) and the associated Hopf bifurcation was recently undertaken in Ref. 1. The present paper gives a quantative perturbation analysis for a particular problem. The work is concerned with extending the linear study of Pflüger to the nonlinear case and thus exploring the post critical behavior of a strut loaded with uniform tangential follower forces. A few peculiarities of the post critical regime are pointed out, such as certain independence of the large deflection. Effort has been made to give realistic applications of follower forces (Trans-Arabian pipeline, nuclear piping) which is very often only loosely hinted at or even misinterpreted. This is especially so with regard to fluid conveying pipes at steady flow. There the friction does not enter into the governing differential equation. Finally, the role of stability control through concentrated masses[2] is also discussed.

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Importance of Eccentric Valves on the Stability of Fluid Conveying Pipes

M. A. J. G. da Silva, University of Illinois at Chicago Circle

The dynamic behavior of pipes conveying fluids has been extensively examined in the past decade with a great number of publications devoted to examine the several parameters that affect the stability of the system or simply its response.

Despite the wealth of available information, the important problem created by the presence of eccentric valves on piping systems, a fact of widespread occurrence in industrial installations, has not been considered.

TG=8 cont'd

The preliminary study presented herein aims at describing possible mechanical models to represent such systems, establishing the pertinent governing conditions and analyzing the stability of some typical configurations. The relative eccentricity and mass of the valves are shown to affect significantly the frequencies of the vibrating pipe and the stability of the mechanical system.

A Parametric Study of Instability Effect on Seismically Excitated Structure-Soil Systems

M. N. Aydinoglu and L. W. Lu, Lehigh University

The effect of soil-structure interaction on the seismic response of structures has been intensively investigated during the past decade. In defining the parameters which control soil-structure interaction, the instability effect of gravity loads is generally neglected. It is generally believed, but not confirmed, that these two effects tend to off-set each other with the resulting response essentially unchanged. Soil-structure interaction is more pronounced in stiff but tall structures on relatively soft soils. Introducing the gravity effect into the analysis, the structure will become softer, which leads to a decreasing tendency in soil-structure interaction. However, the overturning moment due to gravity loads tends to produce a pseudo-softening effect in the soil. Consequently, the simultaneous softening both in structure and the soil could lead to changes in the dynamic response of the system.

An extensive parametric study was performed to clarify the significance of the gravity effect on the dynamic response of soil-structure systems. The results show that the gravity effect may cause considerable frequency shifts and changes in the response spectra values for certain combinations of the non-dimensional parameters. Details of the parametric study can be found in Ref. 1.

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TASK GROUP 14 - HORIZONTALLY CURVED GIRDERS

Chairman, M. Ojalvo, The Ohio State University

Stability of Curved Girders

C. H. Yoo and P. A. Pfeiffer, Marquette University

A finite element formulation for three dimensional curved girders based on the minimum potential energy principle is presented. The element stiffness and stability matrices were derived, including degrees of freedom associated with warping torsion. The bimoment contributions toward stability of unsymmetrical sections is included. The choice of displacement field functions is based on the assumption that the static deformation modes are similar to the buckling shapes. Thus, based on that assumption, the static analysis as the prerequisite of the buckling analysis yields exact solutions regardless of the grid refinement and the eigenvalues are extremely fast converging upper bounds. The formulation has been programmed for use in digital computers and several appropriate examples are analyzed. The obtained eigenvectors for translations and rotations are indeed trigonometric waves as used in the classical solution technique.

The examination of a wide variety of examples reveals fairly significant results in thatthe buckling mode shapes of horizontally curved girders subjected to gravitational loads change depending on the relative stiffnesses of flexure and torsion and the subtended angles. In order to characterize and quantify the results obtained, a series of design charts has been obtained.

TASK GROUP 22 - STIFFENED CYLINDRICAL MEMBERS

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

External Pressure Tests of Ring Stiffened Fabricated Cylinders

R. K. Kinra, Shell Oil Co., and C. D. Miller, Chicago Bridge & Iron Co.

This report summarizes the results of 20 hydrostatic external pressure tests of ring stiffened fabricated cylinders. The tests were carried out at Southwest Research Institute. The objective of the tests was to check the accuracy of the current American Petroleum Institute design rules for fixed offshore platforms.[1] These rules were developed based on theoretical considerations and available test data on seamless pipe.

The test specimen parameters, shown in Table 1, were selected to be representative of:

(1) Typical offshore platform member sizes (D/t from 32 to 128),
(2) Commonly used platform steels (A36 and A572), and
(3) Routine platform fabrication procedures.

The specimens were designed to fail at different stress levels ranging from 0.25 to 0.95 F_y, with failure pressures ranging from 90 to 2700 psi. Specimen numbers 1 to 14 were designed to fail by local buckling of the shell between rings, and specimens 15 to 20 were designed to fail by ring buckling (general instability).

Two tensile coupon tests per specimen were performed to determine the material yield strength and stress-strain properties. The average yield strength value for each test specimen is given in Table 1.

Out-of-roundness measurements were taken at the middle of each bay and adjacent to each ring. As shown in Table 1, the maximum out-of-roundness values for the local buckling specimens ranged from 0.4 to 3.0% vs. the API Spec 2B[2] allowable tolerance of 1.0%. For the general instability specimens, the maximum out-of-roundness values ranged from 1.4 to 2.5%. It is likely that the geometric imperfections were greater than normally expected, because of the relatively small diameter (16 in.) of most specimens.

In an attempt to quantify the effect of geometric imperfections, the test results were plotted against the measured out-of-roundness values. The following relationships were found to fit the elastic buckling test results:

For local buckling:

$$\alpha = 1 - 0.2\sqrt{\frac{D_{max} - D_{min}}{.01D}} \leq 0.8 \qquad \ldots (1)$$

For general instability:

$$\alpha = 1 - 0.22\left(\frac{D_{max} - D_{min}}{.01D}\right) \leq 0.78 \qquad \ldots (2)$$

where α is the imperfection reduction factor, D_{max} and D_{min} are the measured maximum and minimum diameters, and D is the nominal diameter. These relationships are based on only a limited number of tests of cylinders with out-of-roundness values up to 3%. They must, therefore, be investigated further before being generally accepted.

The test results for each specimen were normalized, based on the above relationships, to correspond to an equivalent out-of-roundness of 1%. These adjusted values are plotted in Fig. 1. A comparison with the existing API design curve shows that there is good correlation between test and design values in the elastic buckling range. In the inelastic range, however, the test results generally fall 10 to 15% below the design curve. The results suggest that the effect of residual stresses is much more severe than anticipated. It is possible that the relative effect of residual stresses is heightened by the relatively small diameter of most of the test specimens. More tests with larger specimens and tests on stress-relieved specimens are required to verify this observation.

TG-22 cont'd

A new API design curve, based on the test results in the inelastic range, is proposed. The inelastic buckling equations describing the proposed curve, shown in Fig. 1, are:

$$\left.\begin{aligned} F_{hc} &= 0.45\,F_y + 0.18\,F_{he} & \quad 0.55\,F_y < F_{he} \le 1.6\,F_y \\ F_{hc} &= \frac{1.31\,F_y}{1.15 + (F_y/F_{he})} & \quad 1.6\,F_y < F_{he} \le 6.2\,F_y \\ F_{hc} &= F_y & \quad F_{he} > 6.2\,F_y \end{aligned}\right\} \quad \ldots (3)$$

where F_{he} and F_{hc} are the elastic and inelastic buckling stresses, and F_y is the yield stress. The proposed design curve is a reasonable lower bound for the test results.

An attempt was made to compare the test results with available theoretical methods[3-8] to estimate the buckling pressure, based on measured geometric imperfection values. All the methods considered, except the method proposed by De Hart[8] provide very conservative results. DeHart's method yields unconservative results for this test series. In general, it is found that the difference between theoretical and test results increases with increasing out-of-roundness for local buckling, whereas for general instability the difference decreases with increasing out-of-roundness.

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TABLE 1

HYDROSTATIC COLLAPSE TEST RESULTS

Specimen Number	Diameter D, in.	Thickness t, in.	D/t	Length ft.	Ring Spacing	Ring Size in. x in.	Out-of-Roundness $(D_{max}-D_{min})/.01D$	Yield Stress F_y,ksi	Buckling Pressure, psi
1	16	0.5	32	16	2D	3x.5	2.3	43.5	2200
2	16	0.5	32	16	2D	3x.5	2.2	62.6	2700
3	16	0.5	32	16	3D	3x.5	2.1	38.7	2120
4	16	0.5	32	16	4D	3x.5	1.3	40.0	1790
5	16	0.4375	36.6	16	4D	3x.4375	1.7	42.2	1350
6	16	0.375	42.7	16	1.5D	3x.375	0.5	46.1	1850
7	16	0.375	42.7	16	4D	3x.375	1.0	38.3	1080
8	16	0.3125	51.2	16	4D	3x.3125	1.4	46.5	870
9	16	0.25	64	16	D	2.5x.25	1.6	44.9	1140
10	16	0.25	64	16	D	2.5x.25	2.6	59.4	1205
11	16	0.25	64	16	1.5D	2.5x.25	1.8	56.6	1200
12	16	0.25	64	16	4D	2.5x.25	3.0	45.5	450
13	48	0.375	128	8	0.5D	4x.375	0.7	41.4	425
14	24	0.1875	128	16	2D	2.5x.1875	0.4	45.4	209
15	16	0.5	32	16	3D	1.5x.5	2.2	43.7	1650
16	16	0.3125	51.2	16	2D	1.25x.3125	1.6	45.7	780
17	16	0.25	64	16	2D	1.25x.25	2.5	45.7	545
18	16	0.25	64	16	3D	1.25x.25	2.4	44.6	402
19	24	0.1875	128	16	2D	1.25x.1875	1.4	45.2	99
20	24	0.1875	128	16	2D	1.25x.1875	2.3	44.1	90

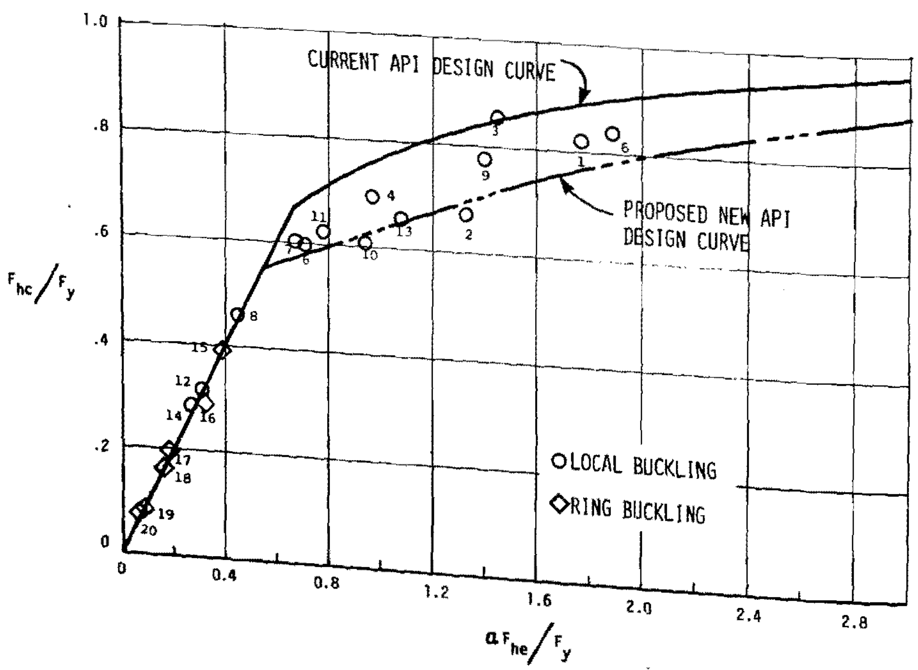

Fig. 1 - Comparison of Test Results with Current and Proposed API Design Curves

TASK GROUP 20 - COMPOSITE MEMBERS

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

Plastic Design with Noncompact Sections Including Composite Bridge Members

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

Fig. 1 shows the moment-versus-rotation curve obtained in a model-bridge test conducted as part of AISI Project 188 on Autostress Design. The flange- and web-slenderness ratios corresponded to the lightest weight available rolled shapes used in bridges. Because these ratios exceed the limiting values for compact shapes with a yield point of 50 ksi, effective yield points for the flange and web are defined as follows:

$$FYFE = 9{,}800 * (TF/BF)^2 \leq FYF \qquad (1)$$

$$FYWE = 38{,}300 * (TW/DWCP)^2 \leq FYF \qquad (2)$$

where FYFE and FYWE are the effective yield points in ksi of the compression flange and web, respectively; TF is the thickness and BF is the total width of the compression flange; TW is the thickness of the web and DWCP is the depth of the web in compression for plastic bending; and FYF is the yield point of the flange material.

An effective plastic moment is then defined as

$$MPE = MP*(FYFE + FYWE)/(2*FYF) \qquad (3)$$

It is suggested that MPE be used in place of the plastic moment, MP, in conventional plastic design. For the model-bridge section the effective negative plastic moment, MPNE, was 49 percent of the negative plastic moment, MPN, of the composite section including the reinforcing bars and excluding the concrete. At MPNE, the available plastic rotation was 140 mrads (Fig. 1).

To place this amount of rotation in perspective, Fig. 2 shows a moment versus plastic rotation curve for a nonsymmetrical section that was approximately compact for a yield point of 50 ksi. The effective plastic moment was 0.95*MP. The plastic rotation at this level was 71 mrads, which may be consideredsufficient for bridges and buildings. The other 48 date points in Fig. 2 represent the effective plastic moment and the corresponding plastic rotation from test results published by several teams of investigators. These data cover a range of BF/(2*TF) from 5.0 to 14.7 and 2*DWCP/TW from 29.9 to 138. In all instances, the available plastic rotation at the effective plastic moment is greater than that provided by the compact section with maximum slenderness ratios for the compression flange and web. Thus, the proposed procedure is conservative.

TG-20 cont'd

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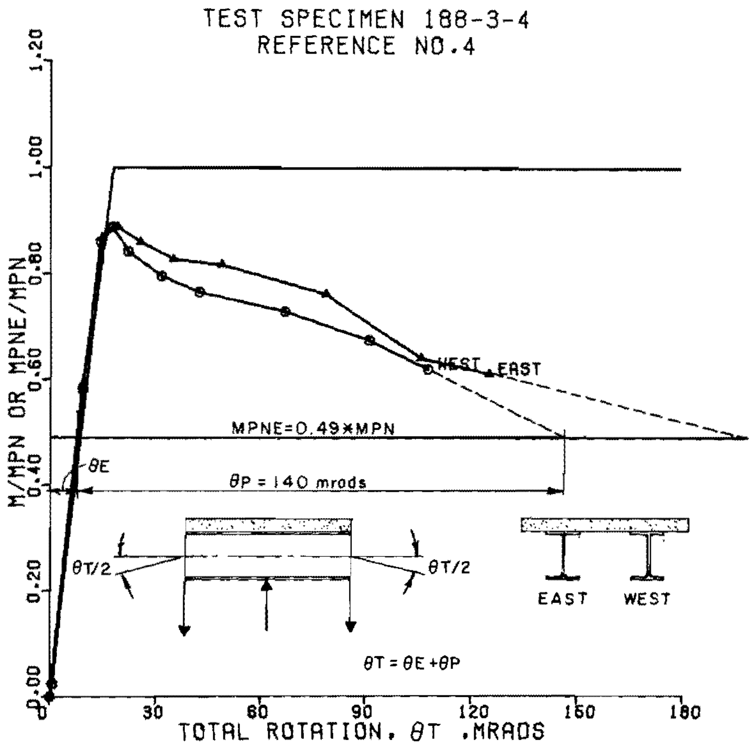
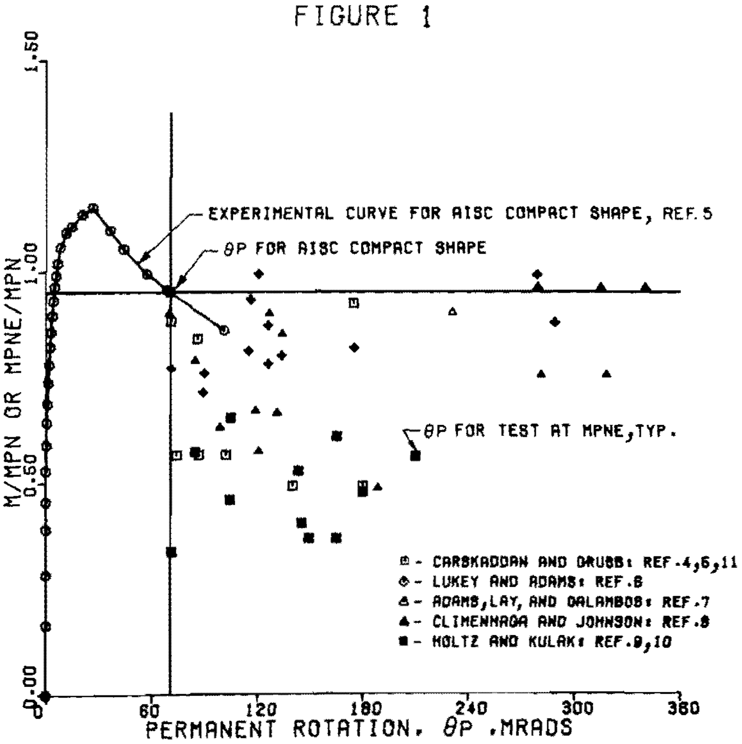


MOMENT VS. TOTAL ROTATION

FIGURE 1



MOMENT VS. ROTATION TEST DATA

FIGURE 2

TASK GROUP 16 - PLATE GIRDERS

Chairman, W. Hsiong, MTA Incorporated

Interaction Effect of Stresses on Plate Collapse

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

In recent years much progress has been made at Imperial College into the buckling behaviour of plate panels subjected to complex forms of in-plane and lateral loading. This progress has, in the main, been made using finite difference computer techniques[1] allowing for both large deflection and plasticity behaviour. Studies have been conducted on plates in combined compression, tension, shear and in-plane bending[2], plates in biaxial compression[3], and plates in biaxial compression and lateral pressure loading[4]. Much of this work was done with the design of box girder bridge elements in mind, although the last study was undertaken for the analysis of ship structures.

The studies of references 2 and 3 were used as the basis for the design rules for panels under complex loading in the new steel bridge code in the U. K. (BS 5400)[5] which has now been released in draft form[6,7]. In formulating these parts of the code it was decided that a direct presentation of the data was too complex, and a simple interaction formula, having its origins in elastic buckling interaction, was formulated to represent the elastic-plastic buckling.

Interaction Formula

Using the results of the study on the interaction of direct and shear stresses[2], it was possible to derive a simple interaction equation representing the combination of direct in-plane and shear stress components. The equation

$$\left[\left(\frac{\sigma_x}{S_{c1}\sigma_o}\right)^2 + \left(\frac{\sigma_y}{S_{c2}\sigma_o}\right)^2\right]^{\frac{1}{2}} + \left(\frac{\sigma_b}{S_b\sigma_o}\right)^2 + \left(\frac{\tau}{S_s\tau_o}\right)^2 = 1$$

represents this interaction. This equation in combination with a limit on panel yield describes the ultimate stress state of the panel. σ_x and σ_y are the longitudinal and transverse compressive stresses in the plate, σ_b is the maximum longitudinal in-plane bending stress, and τ is the coincident level of shear stress. S_{c1}, S_{c2}, S_b and S_s are numerical factors on the yield stresses σ_o and τ_o providing the best fit between the interaction formula and the analytical curves. These factors hence allow for the effect of elasto-plastic buckling. Figures 1 and 2 show a comparison between analysis results and BS 5400 code values using the above interaction formula presented in a paper by Horne[8]. The latter comparison is conservative partly because analytical curves have been chosen which do not include the effect of residual stress and partly because the code formulation largely ignores the effect of aspect ratio for direct stresses and uses a lower bound curve.

It has also been shown that the same numerical factors can be used for the case of biaxial compression. Comparison between predicted values and analysis results[3] are again reasonable.

Interaction of Biaxial Compression and Lateral Pressure

An analytical study has recently been completed[4] into the interaction between biaxial compression and lateral pressure. Using the interaction curves produced the authors of this paper have derived S_c functions which are dependent on lateral pressure level and boundary condition.

These functions can then be used in the same interaction formula to obtain design collapse loads. The effect of aspect ratio has again been ignored for simplicity but could be included without difficulty at the expense of a larger number of design curves. The S_c curves derived in this way are reproduced in Fig. 3. The Q_0, Q_{10}, Q_{20} levels correspond to uniform lateral pressure levels of 0, 10 and 20 metres. Fig. 4 shows a comparison between the analytical results and those predicted by the interaction formula.

Conclusions

Analytical studies have now reached the stage where results are available for the buckling behaviour of isolated plate panels subjected to many variations of applied stresses. It has been possible to represent this behaviour by means of a simple interaction formula and appropriate simplified strength curves. This interaction formula already forms the basis for the behaviour of plate panels of the new UK steel bridge code (BS 5400).

References

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TG-16 cont'd

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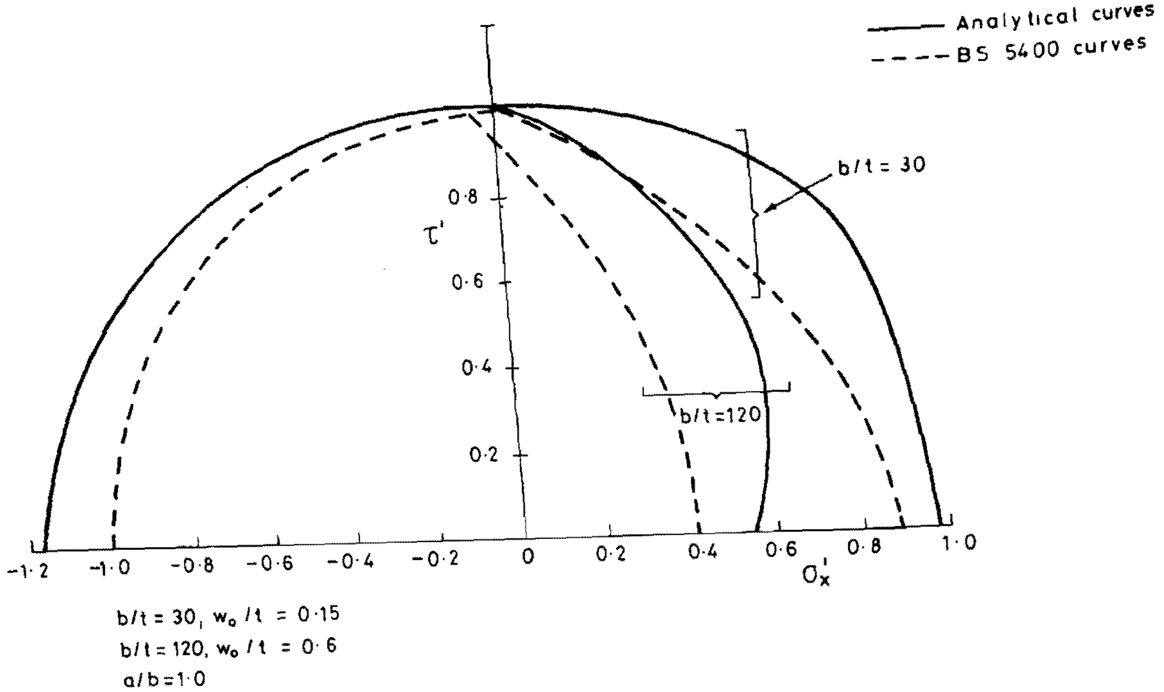

Fig 1 Direct compression / shear interaction curves, top and bottom edges restrained

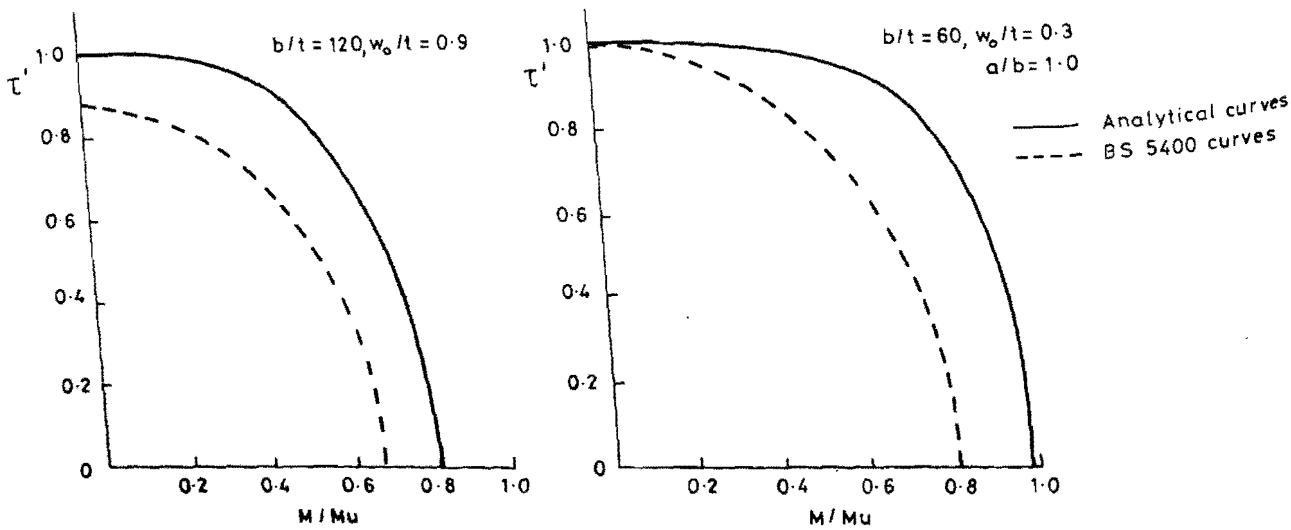

Fig 2. Direct in-plane bending/ shear interaction curves, top and bottom edges restrained

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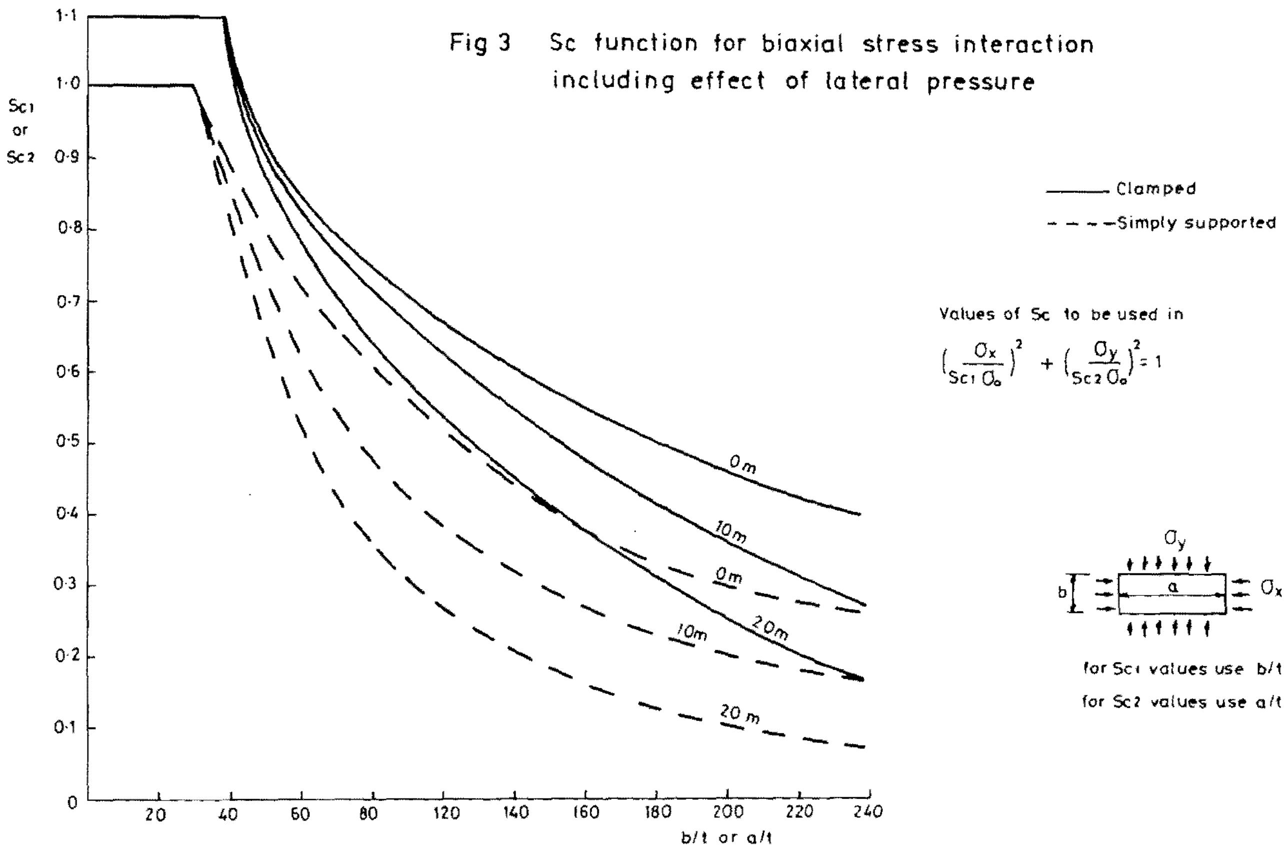

Fig 3 Sc function for biaxial stress interaction including effect of lateral pressure

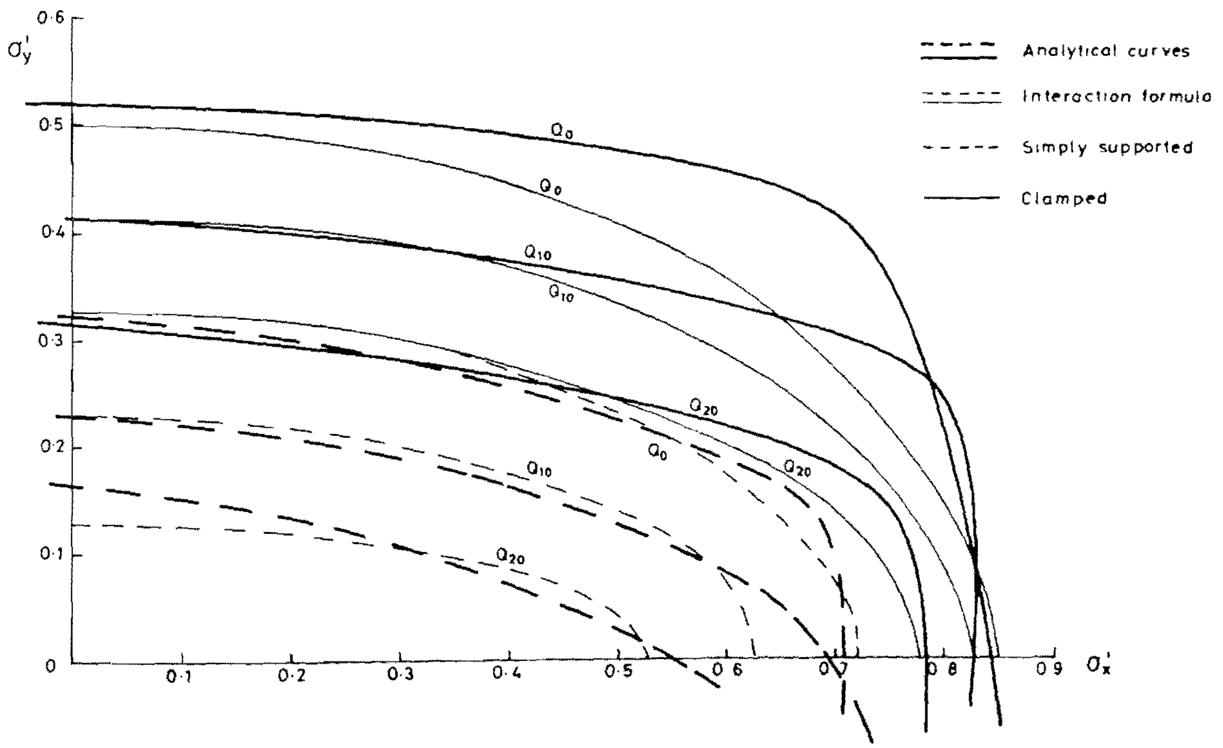

Fig 4 Comparison of analytical curves for plates under biaxial compression and lateral pressure with the biaxial interaction formula

Shear Strength of Longitudinally Stiffened Plate Girders

A. Ostapenko, Lehigh University

The principal difference between the analytical methods which have been proposed for determining the ultimate shear strength of longitudinally stiffened plate girders has been the manner of considering contributions of the individual web plate subpanels in the post-buckling range. In this study, the same basic shear strength formulation of a transversely stiffened plate girder panel was used in four different methods for longitudinally stiffened plate girders. The ultimate shears from these four analytical models were compared with the results of forty (40) tests on plate girder specimens.

The basic method used in this study assumes that the ultimate shear capacity of a plate girder panel consists of three contributions:

$$V_u = V_b + V_{tf} + V_f \ . \qquad (1)$$

In this equation,

V_u = ultimate shear strength

$$V_b = \tau A_w \qquad (2)$$

= buckling shear of web plate

where

$A_w = bt$ = web area

τ = buckling shear stress, computed using the buckling coefficient

$$k = 5(1 + 1/\alpha^2) \qquad (3)$$

with

$\alpha = a/b$ = aspect ratio

V_{tf} = tension-field strength, it is assumed to be according to the formula proposed by Wolchuk and Wang.

$$V_{tf} = \tau_t A_w \qquad (4)$$

where

$$\sigma_t = \frac{\sigma_y - \tau\sqrt{3}}{2\sqrt{1 + \alpha(0.5 + \alpha)}} \qquad (5)$$

σ_y = yield stress of the web plate

V_f = frame action shear needed to develop a panel plastic mechanism in a frame composed of the flanges (and longitudinal stiffness when they are present) and the transverse stiffeners.

$$V_f = \frac{2(\Sigma M_{pf} + \Sigma M_{ps})}{a} \qquad (6)$$

where

M_{pf} and M_{ps} are the plastic moments of the flanges and longitudinal stiffeners.

A segment of a typical longitudinally stiffened plate girder is shown in Fig. 1. Each web subpanel is characterized by its slenderness and aspect ratios, b_i/t and a/b_i, respectively.

The four models for longitudinally stiffened plate girders are shown in Fig. 2.

The frame action shear V_f is relatively small, and it was assumed to be given by Eq. 6 for all four models.

Model 1 assumes that the ultimate strength of each subpanel is developed independently as a function of the slenderness and aspect ratios $\beta_i = b_i/t$ and $\alpha_i = a/b_i$ of the subpanel according to Eqs. 2 and 4, the web area of each subpanel being $A_{wi} = b_i t$. Then,

$$V_u = \Sigma V_{bi} + \Sigma V_{tfi} + V_F \qquad (7)$$

Model 2 treats the segment as if it had no longitudinal stiffeners except that the buckling stress τ is taken to be the smallest of the buckling stresses of the individual subpanels, computed as for Model 1. The tension field contribution is computed by Eq. 4 using τ_{min} and $\alpha = a/b$ for σ_t in Eq. 5.

$$V_u = (\tau_{min} + \sigma_t)A_w + V_f \qquad (8)$$

Thus, the only benefit of the longitudinal stiffeners in this model is to increase the buckling stress.

Model 3 is the same as Model 1 except that the same $\alpha_{min} = a/(b_i)_{max}$ is used in Eq. 5 for computing σ_{ti} for the subpanels.

$$V_u = \Sigma V_{bi} + \Sigma V_{tfi}(\alpha_{min}) + V_f \qquad (9)$$

Model 4 is analogous to Model 3 except that the common aspect ratio in Eq. 5 is taken to be $\alpha = a/b$ of the whole segment.

$$V_u = \Sigma V_{bi} + \Sigma V_{tfi}(\alpha = a/b) + V_f \qquad (10)$$

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Ultimate shears according to these four models were compared with the results of forty (40) tests on longitudinally stiffened plate girders reported in literature (Cooper, Yen, Ostapenko, Rockey). The following three parameters were used in the comparison:

Comparison Ratio

$$R = V_u/V_{exp} \tag{11}$$

where

V_u is the ultimate shear from Eqs. 7 to 10.

V_{exp} is the experimental ultimate shear.

Note that V_f which is part of V_u is the same for all four models.

Standard Deviation

$$S = \sqrt{\frac{(R_i - R_{avg})^2}{n - 1}} \tag{12}$$

Standard Deviation with respect to 1.0

$$S_1 = \sqrt{\frac{(R_i - 1.0)^2}{n - 1}} \tag{13}$$

The resulting values for the 40 tests are shown in Table 1a. Each model is symbolically indicated at the top by the pattern of the tension-field bands characteristic for it. The averages (1st line) show Model 4 with its value of $R_{avg} = 1.01$ to be the most accurate. Next best is Model 2 ($R_{avg} = 0.92$) although with one tension-field band it is the simplest.

The scatter, indicated by S in the second row, is again the smallest for Model 4 ($S = 0.073$) with Model 2 being quite close ($S = 0.075$). The standard-deviation-with-respect-to-one S_1 (row 3) is indicative of the accuracy and of the scatter. It is the lowest for Model 4 ($S_1 = 0.073$) with Model 2 somewhat worse ($S_1 = 0.112$) and Models 1 and 3 being the poorest.

Since most of the test girders had only one longitudinal stiffener, a separate comparison was made for the five girders which had two or more. This comparison should be more indicative of the generality of a method. As shown in Table 1b, the average for Model 4 is best (0.95) with the average for Models 1, 2 and 3 being approximately the same (0.84 to 0.87), but noticeably lower than for Model 4. Models 1, 3 and 4 (separate tensio fields) give much better consistency than the single tension-field Model 2 as shown by the values of S. S_1 values show that Model 4 is significantl better than other models ($S_1 = 0.060$ vs. $S_1 = 0.161$ for Model 2).

Additional comparisons between the Models were made with respect to the relative size and the number of subpanels.

All these studies lead to the following conclusions:

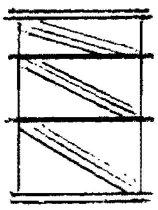
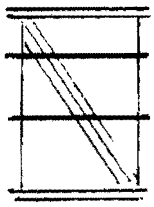
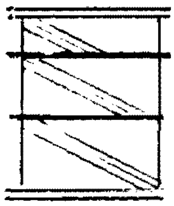
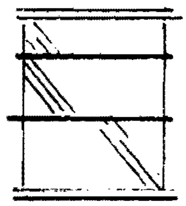
1. Model 4 has distinctly better validity for plate girders with unequally spaced multiple longitudinal stiffeners.

2. Model 4 is the most consistent (smallest S) and the most accurate (smallest S_1) for all types of girders.

3. Of the three models with separate tension fields (Models 1, 3 and 4), Model 1 (different inclination of the tension field bands -- Cooper, Ostapenko) is the least accurate.

4. Model 2 (single tension field) may be acceptable for girders with one longitudinal stiffener because of its simplicity and reasonable accuracy.

Table 1. Summary of Comparisons of Four Analytical Models with Test Results

MODEL	1	2	3	4
				
a) Comparison for 40 Tests				
R_{avg}	0.88	0.92	0.91	1.01
S	0.094	0.075	0.093	0.073
S_1	0.150	0.112	0.131	0.073
b) Comparison for 5 Tests (Two or More L.S.)				
R_{avg}	0.84	0.87	0.85	0.95
S	0.018	0.067	0.015	0.013
S_1	0.175	0.161	0.163	0.060

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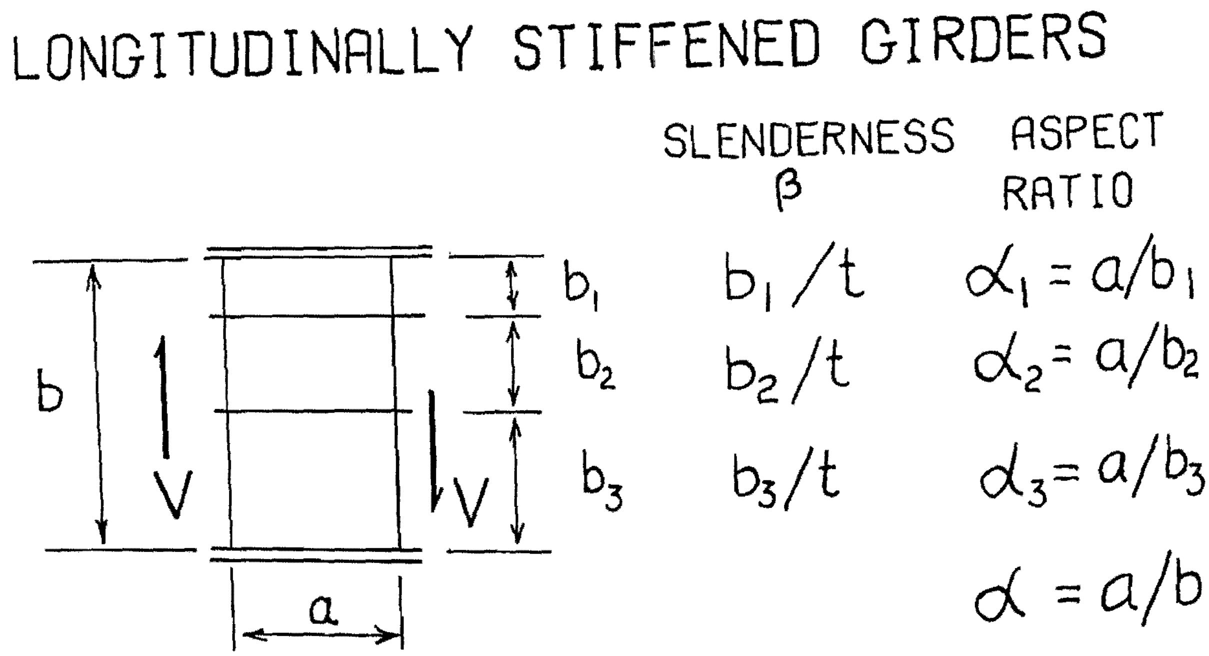


FIG. 1 PLATE GIRDER SEGMENT WITH LONGITUDINAL STIFFENERS

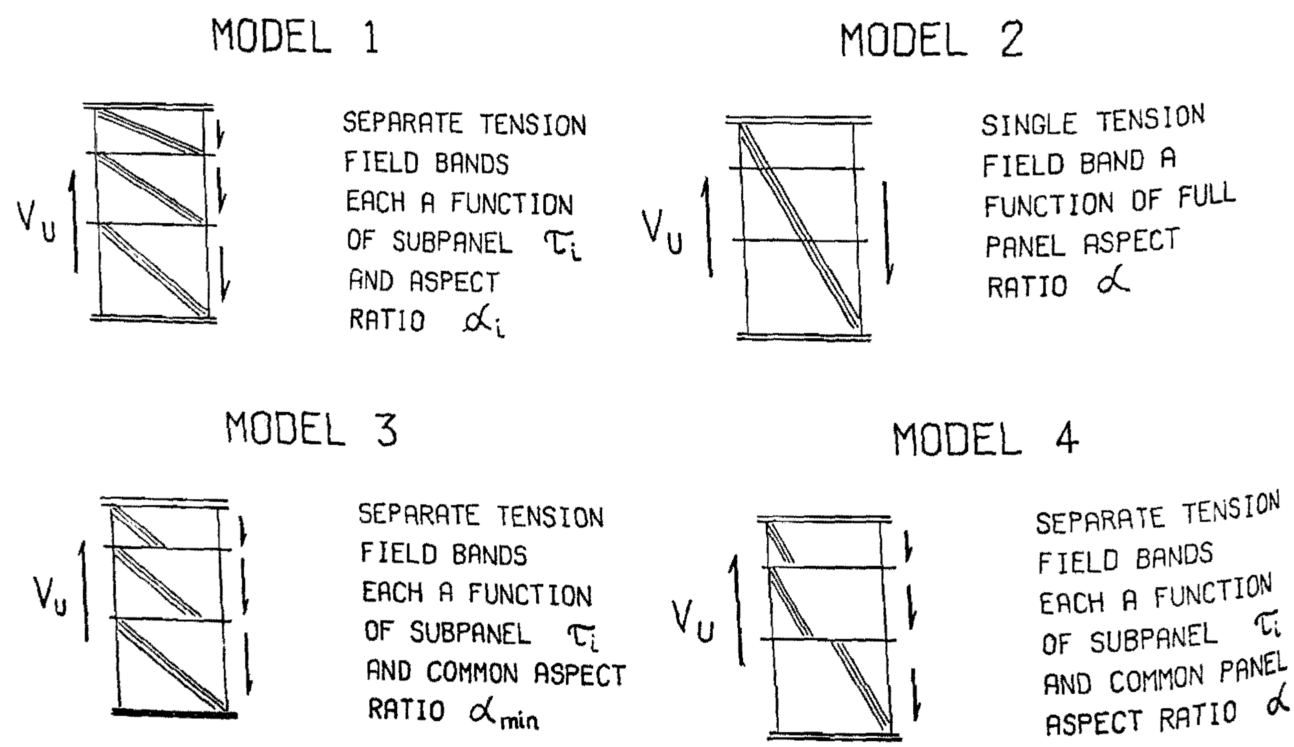


FIG. 2 ANALYTICAL MODELS COMPARED IN STUDY

Buckling Stresses of Web Panels in Girders

Y. S. Chen and B. T. Yen, Lehigh University

Buckling stresses of web panels in plate girders and box girders are computed for the following conditions. (1) Boundary restraint: fixed at top and bottom of web and simply-supported along the vertical edges. (2) Stress condition: linearly varying longitudinal normal stresses and uniform shearing stresses. The ratio of normal stresses at the top and bottom of the web is a primary parameter of study, and attention is focused on loading conditions where tensile normal stress in a cross section is higher than the compressive stress. Finite difference procedure is employed. The results are summarized in charts, and supplement the interaction formula developed by others.

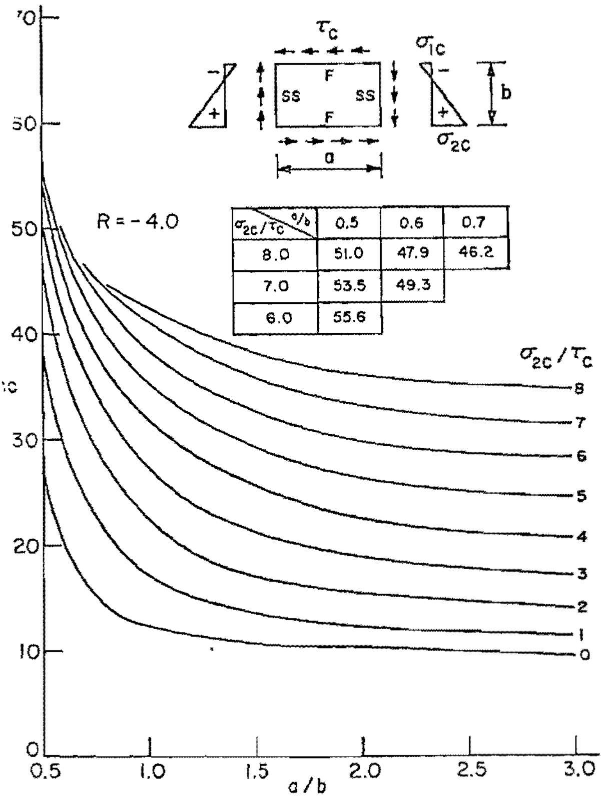


Fig. 1 Buckling Coefficient for $R = (\sigma_{2c}/\sigma_{1c}) = -4.0$

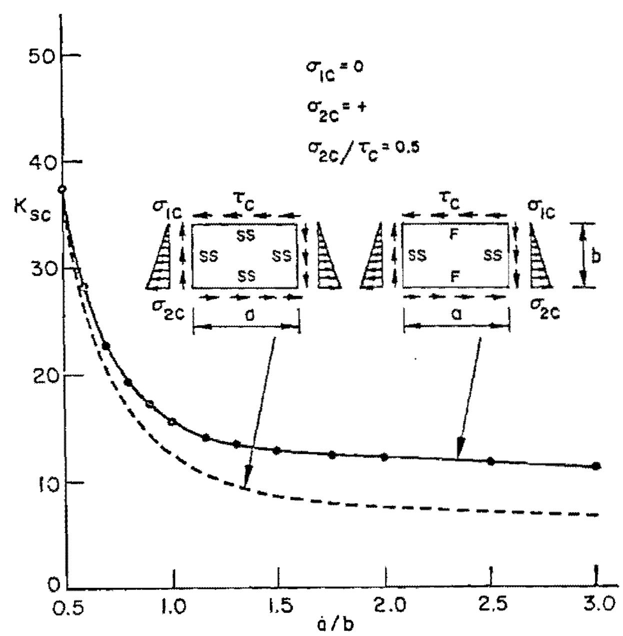


Fig. 2 Comparison of K_{sc} for Plate Subject to Shear and Triangular Tension

TG-16 cont'd

The Collapse of Stiffened Shear Webs

C. Marsh, Concordia University

By considering a shear panel as composed of diagonal strips in tension and compression, with the compression strips dictating the collapse load, a stress system is derived which satisfies all the boundary conditions. The manner in which the flange strength contributes to the collapse load is also explained. (Fig. 1)

The design treatment derived is a direct procedure, without the need for empirically established rules, which leads to a shear capacity for light flanges ($\mu < 0.005$) given by:

$$\frac{V}{bt\tau_y} = [2\ (\tau_c/\tau_y)^{1/2} - \tau_c/\tau_y] + (3\mu)^{1/2}$$

where τ_c = initial buckling stress

τ_y = shear yield stress

τ_u = shear ultimate stress

b,t = web depth and thickness

$\mu = M_o/b^2t\tau_y$

M_o = plastic moment for the flange plate.

For the general case the total shear force is given by:

$$\frac{V}{bt\tau_y} = \frac{\tau_e}{\tau_y} + \frac{\tau_e'}{\tau_y}$$

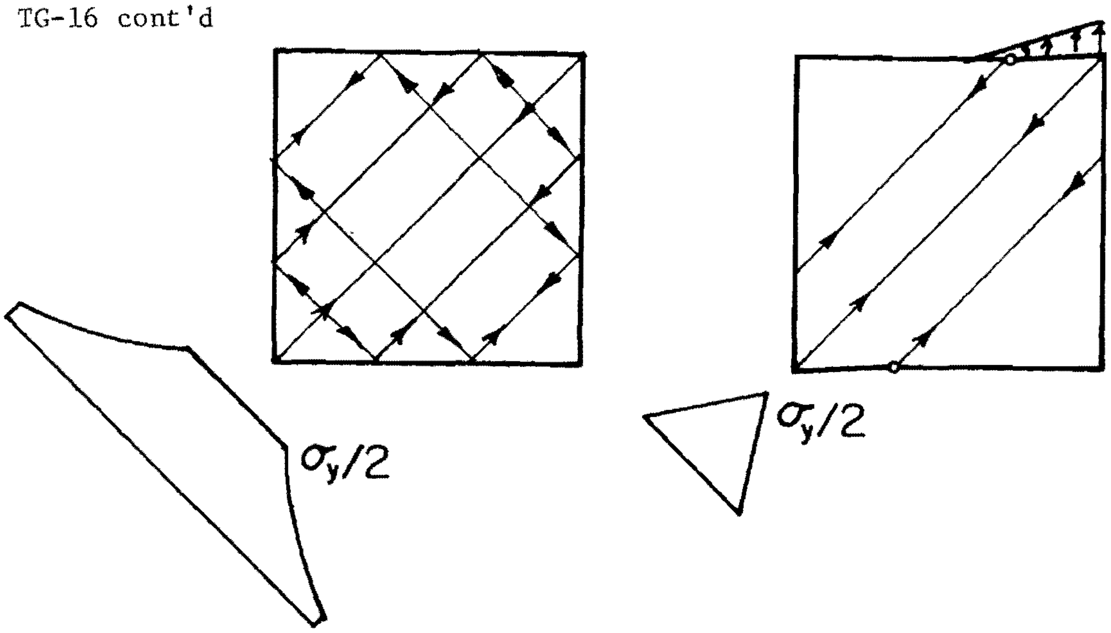
in which

τ_e/τ_y is contributed by the web

τ_e'/τ_y is contributed by the flange.

These values are function of μ , and are obtained from the Fig. 2.

TG-16 cont'd



a) Shear in Web

b) Diagonal Tension Causing Flange Bending

FIG. 1 STRESS SYSTEMS IN THE WEB

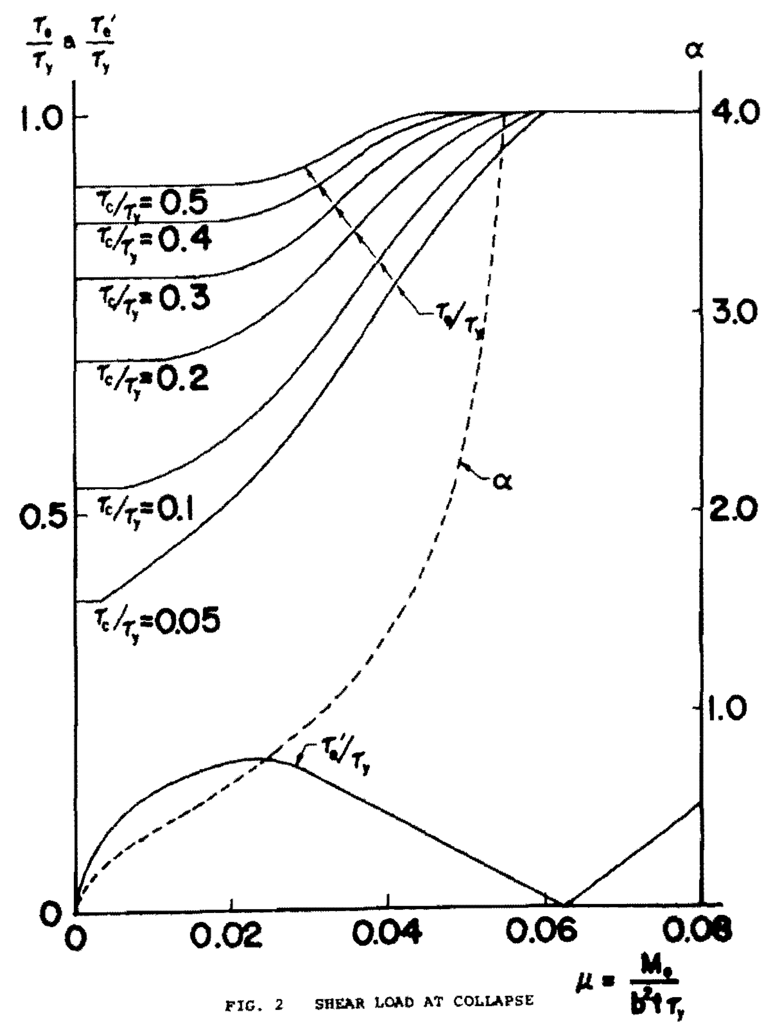


FIG. 2 SHEAR LOAD AT COLLAPSE

TASK GROUP 4 - FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

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

Nonprincipal Axis Bracing of Centrally Loaded Columns

M. Ojalvo, T. A. Bolte and V. Faridani, The Ohio State University

Although the occasion arises when a designer wishes to determine the capacity of or to design a column with an intermediate brace that is not parallel to either principal axis of the cross section, the problem seems not to have had a theoretical treatment until recently.[1]

In the present discussion a centrally loaded column of length L is braced at a distance βL from one end. The brace prevents translation normal to the column along a direction making the angle α with the minor principal axis (x). For a column with a coinciding shear center and centroid the equations applicable to the segments on either side of the brace are:

$$u^{iv} + k_y^2 u^{II} = 0 \qquad (1)$$

$$v^{iv} + k_x^2 v^{II} = 0 \qquad (2)$$

where $k_y^2 = \frac{P}{EI_y}$, $k_x^2 = \frac{P}{EI_x}$ and u and v are displacements of the buckled column in the directions of the principal axes. Flexural behavior in the zx and zy principal planes are coupled through two of the eight continuity conditions at the brace point.

The analysis indicates that a nonprincipal axis brace is always effective in raising the columns elastic buckling load over the unbraced condition. For a range of ratios of Iy to Ix and values of α the brace is as effective as if it were oriented in a direction parallel to the major principal axis.

An experimental program for verification of the theoretical results is planned. A variation of the problem in which shear centers and centroids do not coincide is also under study. For this variation the more elaborate system of three equations as proposed by Goodier and Vlasov and including torsional deformations may be needed. Noncoincident shear centers and centroids occur in angle struts. Angle compression members are more frequently braced by a single nonprincipal axis brace.

Reference

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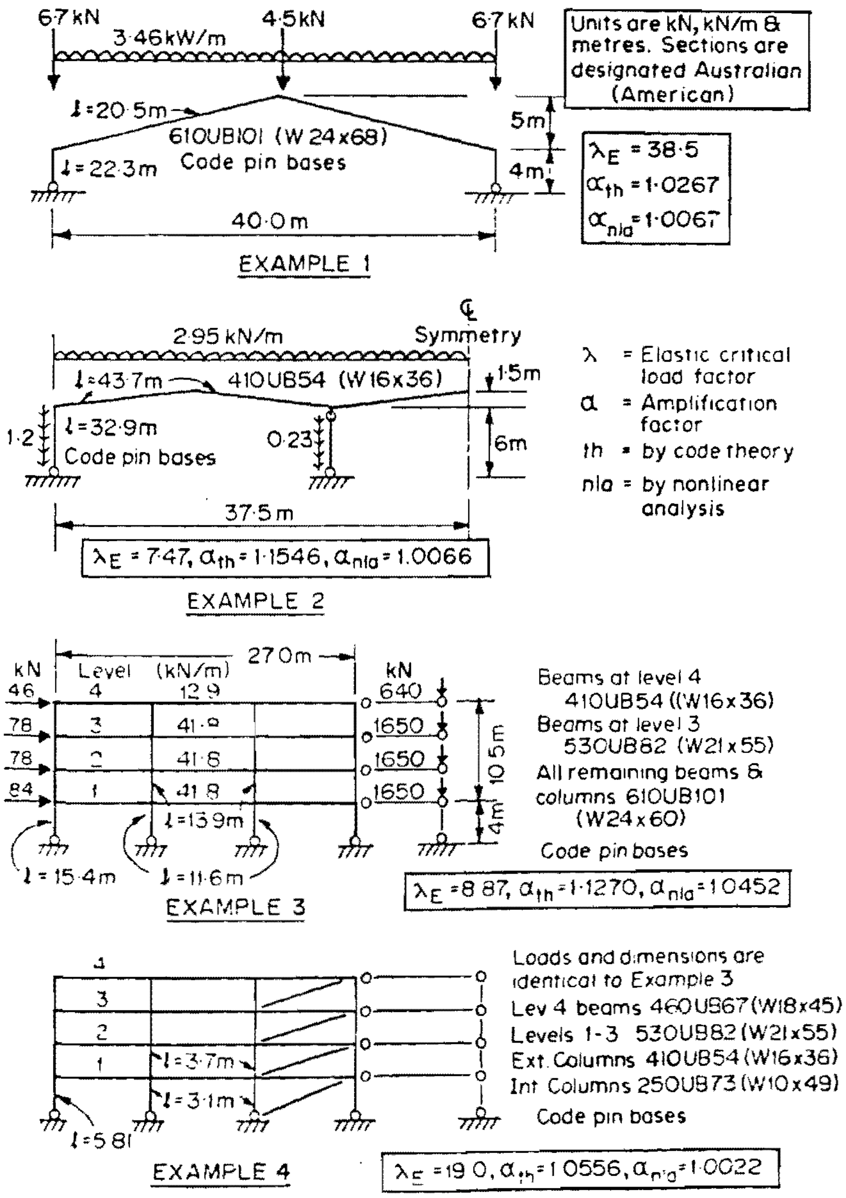
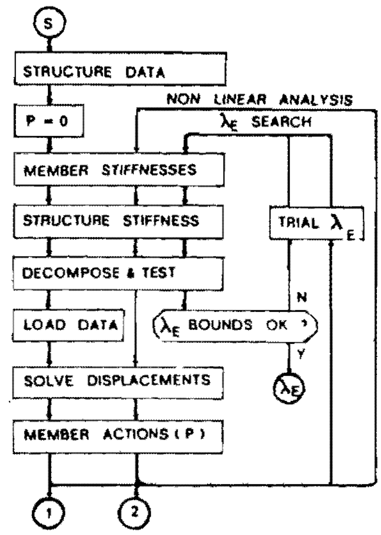


6·7 kN
4·5 kN
6·7 kN
3·46 kW/m
l = 20·5 m
610UB101 (W 24x68)
Code pin bases
l = 22·3 m
5m
4m
40·0 m
Units are kN, kN/m & metres. Sections are designated Australian (American)
λ_E = 38·5
α_{th} = 1·0267
α_{nla} = 1·0067
EXAMPLE 1
℄ Symmetry
2·95 kN/m
l = 43·7 m
410UB54 (W16x36)
1·5m
1·2
l = 32·9 m
Code pin bases
0·23
6m
37·5 m
λ_E = 7·47, α_{th} = 1·1546, α_{nla} = 1.0066
EXAMPLE 2
λ = Elastic critical load factor
α = Amplification factor
th = by code theory
nla = by nonlinear analysis
27·0 m
kN
Level
(kN/m)
46
4
12·9
78
3
78
2
41·8
84
1
41·8
kN
640
1650
1650
1650
10·5 m
4m
l = 13·9 m
l = 15·4 m
l = 11·6 m
Beams at level 4 410UB54 (W16x36)
Beams at level 3 530UB82 (W21x55)
All remaining beams & columns 610UB101 (W24x60)
Code pin bases
λ_E = 8·87, α_{th} = 1·1270, α_{nla} = 1·0452
EXAMPLE 3
4
3
2
1
l = 3·7 m
l = 3·1 m
l = 5·81
Loads and dimensions are identical to Example 3
Lev 4 beams 460UB67 (W18x45)
Levels 1-3 530UB82 (W21x55)
Ext. Columns 410UB54 (W16x36)
Int Columns 250UB73 (W10x49)
Code pin bases
λ_E = 19·0, α_{th} = 1·0556, α_{nla} = 1·0022
EXAMPLE 4



S
STRUCTURE DATA
NON LINEAR ANALYSIS
P = 0
λ_E SEARCH
MEMBER STIFFNESSES
STRUCTURE STIFFNESS
TRIAL λ_E
DECOMPOSE & TEST
N
LOAD DATA
λ_E BOUNDS OK ?
Y
λ_E
SOLVE DISPLACEMENTS
MEMBER ACTIONS (P)
1
2

TG-4 cont'd

Evaluation of Frame Systems Based on Optimality Criteria with Performance Constraints and P-Δ Effect

F. Y. Cheng, University of Missouri-Rolla

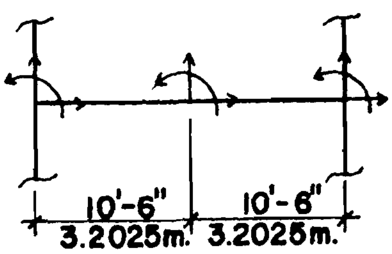
In the past decade, a considerable amount of literature has been published in the area of optimum structural design. The increasing number of publications correspond closely to the rapid demand for economical and reliable structural design mainly in aircraft engineering. The conventional design is based on the trial and error process and is recognized that this is an inadequate method, which yields solutions that cannot always satisfy safety and performance constraints and provide the lowest possible structural cost. The current computer applications in structural engineering is based on the conventional design and must have a preliminary assumption of member stiffnesses. If the initial stiffnesses are misjudged, repeated analysis, regardless of the program's sophistication, will not yield an improved design. The presentation shows a practical optimization technique and several versatile performance constraints. The constraints are the upper limits of stresses and displacements resulting from static loads and from the combined response of static loads and seismic excitations as well as the lower bound of vibrating frequencies. The computer program, ODSEWS (Optimum Design of Static, Earthquake, and Wind Structures), developed at UMR can be used to select the structural members for which the upper and lower bounds of the cross-sectional properties can be imposed. The main objective is to yield the lowest structural weight and to satisfy engineering performance requirements. The structural formulation is based on the consistent mass method with the P-Δ effect of the vertical gravity load and ground motions. In the presentation, four 15-story frameworks of unbraced, single-, double-, and K-braced systems shown in Figs. (A), (B), (C), and (D) are designed for coupling ground motions of the El Centro, 1940 earthquake with 5% damping.

The 15-story buildings have the same span length of 21 ft., the floor height, 12 ft., the dead load on each floor as nonstructural mass, w, 180 lbs/in., the modulus of elasticity, E, 29,000 ksi, and the mass density of the construction material $\rho = 0.283$ lbs/in^3. The allowable stress for bending combined with the axial force is assumed to be $\sigma \leq 29$ ksi, the allowable shear stress, σ_v, should be less than or equal to 0.65 σ, and the allowable deflection is based on the relative displacement between floors limited to 0.005 times the story height. The beams and columns are built-up wide flange sections and the bracings are bar elements. Several final design results are shown in Table 1 in which case (a),H, signifies the design resulting from the horizontal ground motion only, Case (b), H+P-Δ(DL), is due to the horizontal ground motion plus the P-Δ effect of static load of nonstructural mass acting on girders, Case (c) of H+V indicates the horizontal and vertical earthquake components, and Case (d) represented by H+V+P-Δ(DL+V) corresponds to the design obtained by considering horizontal and vertical earthquake components as well as the P-Δ effect of the vertical inertia forces associated structural and nonstructural masses. Included in the Table are the structural weight, natural periods, and the displacements at top floor from which one may observe that, for the same constraints, the K-braced system requires much less structural weight than other systems and the P-Δ effect demands heavier structural design for all the systems.

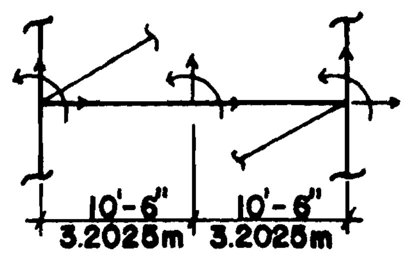
TG-4 cont'd

TABLE 1. FINAL WEIGHTS, NATURAL PERIODS, AND DISPLACEMENTS AT TOP FLOOR
(A=Unbraced, B=Single-Braced, C=Double-Braced, D=K-Braced)

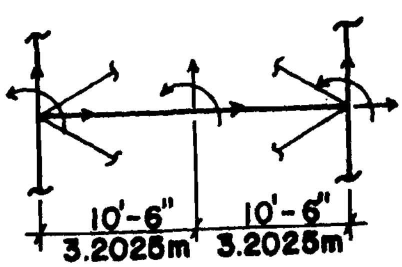
Group	Case	Final Weight (kips)	Natural Period (sec.) 1	2	3	4	5	Disp. at Top Floor (in.)
A	a	60.44	2.152	0.694	0.404	0.282	0.212	10.80
	b	62.21	2.179	0.671	0.386	0.268	0.202	10.80
	c	60.77	2.158	0.692	0.402	0.281	0.217	10.80
	d	62.21	2.152	0.677	0.395	0.276	0.212	10.80
B	a	48.32	1.879	0.490	0.256	0.247	0.187	9.00
	b	47.69	1.876	0.479	0.274	0.252	0.209	9.05
	c	48.76	1.874	0.488	0.291	0.255	0.224	9.02
	d	50.01	1.874	0.485	0.288	0.253	0.221	9.02
C	d	31.55	2.134	0.509	0.398	0.361	0.335	10.80
D	d	28.61	2.111	0.519	0.274	0.269	0.178	10.55



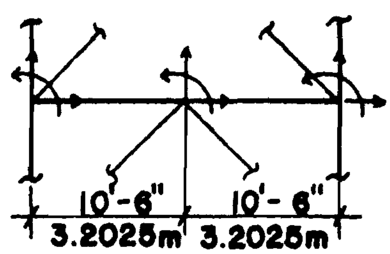
(A) Unbraced System



(B) Single-Braced System



(C) Double-Braced System



(D) K-Braced System

Fig. 1. Typical Degree-of-Freedom of Various Frame Systems

TASK GROUP 21 - BOX GIRDERS

Chairman, R. C. Young, Frank E. Basil, Inc.

Stability of Cable Stayed Box Girders

M. C. Tang , DRC Consultants

The box girder of a cable-stayed bridge is a highly stressed compression member with global and local bending. The global compression force in the girder increases the global bending moment, which will cause an amplified compression in the flanges of the box girder. This compressive force in turn will increase the local bending stresses due to dead load or wheel load. This problem is not mentioned in the existing design specifications, but requires special attention in the design.

TASK GROUP 7 - TAPERED MEMBERS

Chairman, A. Amirikian, Amirikian Engineering Co.

Design of Single-Story Rigid Frames Consisting of Tapered Members

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

This presentation describes the contents of a book to be published under the auspices of the Metal Building Manufacturers Association, authored by George C. Lee, Robert L. Ketter, and T-L. Hsu.

It is the primary purpose of this book to summarize all of the pertinent results of research and investigations achieved in recent years having to do with the design of steel, single story, rigid frames, and thereby facilitating their application by structural engineers.

The structures considered in this book have been widely used in industrial building construction. In the past, most designs presumed rolled shapes, and applications were limited by economic and other considerations to particular span lengths. In more recent years, however, with advancement in welding and cutting technology, in methods of structural analysis and design, and in the production of higher strength steels, single story rigid frames have been much more popular. At the present time this type of construction is used widely - for industrial plants, warehouses, laboratories, office buildings, schools, churches, shopping centers, and recreational facilities, to name but a few of the more obvious applications.

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

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

Load and Resistance Factor Design of Cold-Formed Steel Structural Members

W. W. Yu and B. Supornisilaphachai, University of Missouri-Rolla, and
T. V. Galambos, Washington University

The "Allowable Stress Design Method" has long been used for the design of cold-formed steel structural members in the United States, Canada and other countries. Recently, the "Load and Resistance Factor Design Method" for cold-formed steel has been developed from a joint research project conducted at Washington University and the University of Missouri-Rolla under the sponsorship of American Iron and Steel Institute.

This paper summarizes the research activities concerning the statistical analysis of mechanical properties and calibration of the AISI Specification for the Design of Cold-Formed Steel Structural Members. The development of new LRFD criteria is also discussed.

The Strength of Cold-Formed Steel Columns

D. T. Dat, Exxon Production Research, and T. Pekoz, Cornell University

The flexural buckling strength of cold-formed steel columns is investigated. The study covers press-braked and roll-formed stiffened channels of gage 13 and 14 steel, roll-formed hats of gage 7 and 11 and a thicker roll-formed hat.

Cold-forming effects are studied by means of tensile and compressive tests. The study confirms that cold-forming raises the yield strength and ultimate strength but decreases ductility. Cold-forming process also introduces significant residual stresses.

Longitudinal residual strains are released by sectioning and measured with electric resistance strain gages. The released strains are negative (contraction) on the convex face of the section, positive on the concave face, but the average is zero. Large residual strains are found in the flat portions of a section, but no significant or systematic difference between the residual strains of press-braked sections and those of roll-formed sections is observed.

Twenty stub column tests are performed. In order to remove residual stresses, some stub columns are annealed and exhibit higher strength than the non-annealed ones. Sixty pin-ended columns, whose slenderness ratios span the inelastic range, are tested under concentric load.

A mathematical model is developed to predict column strength. The lateral deflections, both initial and load-caused, are assumed sinusoidal. The model accounts for the non-uniformity of yield strength and the presence of residual stresses. Three distributions of residual stresses across the thickness is assumed: uniform, linear, and "rectangular," i.e., two rectangular blocks. Results are in fair agreement with experiments, except for the gage 14 channels, which exhibit lower strength than predicted.

Linear regressions using ordinary and generalized least squares are performed on the data. The adequacy of the "SSRC parabola" is discussed.

This research project was sponsored at Cornell University by the American Iron and Steel Institute.

TASK REPORTERS

TASK REPORTER 14 - FIRE EFFECTS ON STRUCTURAL STABILITY

K. Klippstein, U. S. Steel Corporation

A Report on the Effects of Fire on the Stability of Steel Columns

This report provides (1) a brief update on previous studies on the effects of fire on exterior structural steel in buildings, including solid and waterfilled columns, (2) a short overview of a recently completed study on the behavior of thin-walled cold-formed steel columns in wall assemblies exposed to the ASTM E119 fire test, and (3) some of the present and future effects of these studies on the ASTM E119 standard or on potential future design specifications.

In 1975, the Subcommittee on Fire Technology of the Committee on Construction Codes and Standards of the American Iron and Steel Institute (AISI) commissioned a study on the effects of fire on exterior structural steel in buildings. A two-volume report on this study was completed in 1977. In 1979, a condensed version of the reports was published as "Fire-Safe Structural Steel - A Design Guide." This GUIDE presents step-by-step discussions, analyses, and procedures for designing fire-safe exterior structural-steel building components, including solid steel columns. The procedures determine the maximum steel (column) temperature during a fire in the investigated structure. This temperature is compared to a "safe design temperature" of 1000° F. Similar procedures are available to

TR-14 - cont'd

determine whether the design of waterfilled exterior steel columns in a building is safe, in case a fire occurs. Basically, all of these methods are analytical because no "standard" fire tests are available for "exterior" structural members, but the developed engineering methods appear to be rational and growing widely in acceptance by building authorities.

For thin-walled cold-formed steel columns in exterior or interior load-bearing wall assemblies, an ASTM test standard (E119) does exist and has been utilized repeatedly for specific steel columns (studs) with limited variations of parameters provided by the other components in wall assemblies. To reduce testing of an almost infinite variety of wall assemblies with steel studs, the AISI Sheet Committees initiated a study in 1975 with the objective to obtain fire ratings for wall assemblies with generic steel studs. The fire ratings represent the minimum time during which a wall assembly exposed to the ASTM E119 fire test standard is capable of resisting a certain axial load and of meeting certain temperature-related requirements. A report of this study was published in 1979, and fire ratings for exterior and interior load-bearing wall assemblies with steel studs are to be published by Underwriters' Laboratories during 1980. The allowable loads of the generic steel studs in a variety of fire-rated wall assemblies are based on a crude approximation of the stud stability conditions; however, further refinements are expected as a result of additional future tests.

The studies described have already resulted in some changes of the ASTM E119 test standard, and more changes are expected. For instance, a previous requirement that the wall assembly (or the stud) should carry twice the design load after the fire test is completed, was stricken from the standard because the safety factor of any steel column exposed to room temperature conditions never exceeds 1.92 rather than the implied value of 2.0. Also, the data obtained from the wall-assembly fire tests indicate that the test conditions for a particular stud (temperature, load, and deflection) within a tested assembly are not repetitive. To assure repetitive conditions, the test standard is in need of more stringent definitions of such test parameters as the deviation of furnace temperatures, uniformity of stud loads, etc. Also, more extensive data acquisition (temperature distribution, deflection, and load for each stud during entire test) would be desirable.

It appears conceivable that the results of the research described in this report, and those of future research, would have a significant bearing on the development of design specifications that consider the effects of fire on the stability of columns in buildings.

TASK REPORTER 15 - CURVED COMPRESSION MEMBERS

W. J. Austin, Rice University

Inplane Stability of Parabolic Arches

H. B. Harrison, University of Sydney

Some numerical studies are reported of the response to uniformly distributed loading of parabolic arches of rise equal to a fourth of the span. Base conditions are both fixed and pinned. The distributed loading varies between covering less than half the span to covering the complete span. The numerical studies use a discrete element, large deformation method that enables the prediction of the complete load-deformation history of an elastic arch up to full inversion if necessary. Account is taken of axial as well as flexural strains and, under full span loading published data are confirmed for the load intensities at which equilibrium bifurcation occurs. The significant result, however, is that the intensity of load that can be supported by a fully elastic arch is significantly less than the bifurcation value when approximately three-fourths of the span is loaded. The reduction in load intensity is nearly 20 percent for the fixed ended arch.

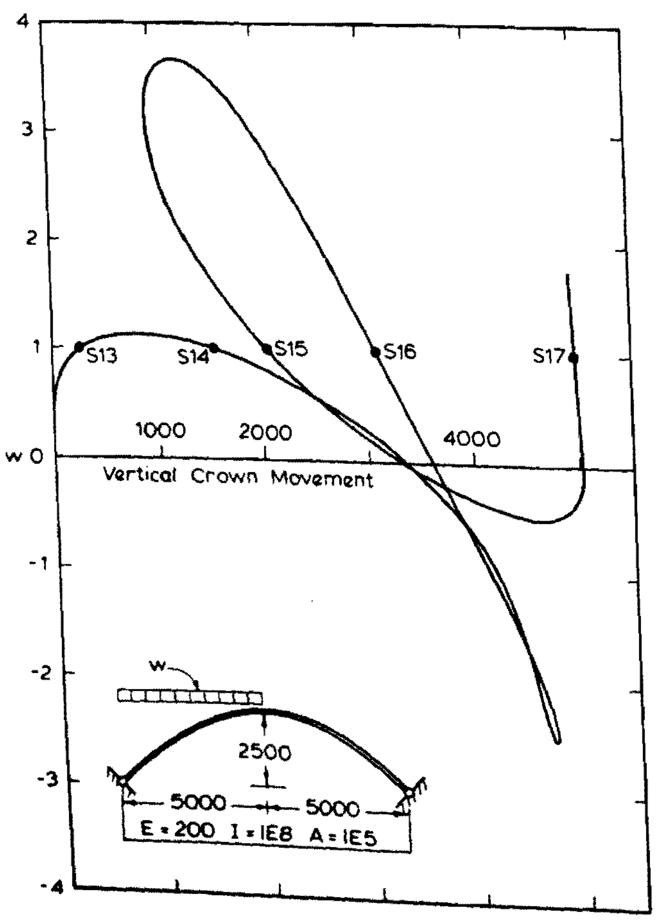


FIG 8 LOAD - CENTRAL DEFLECTION FOR HALF-LOADED PARABOLIC ARCH WITH PINNED BASES

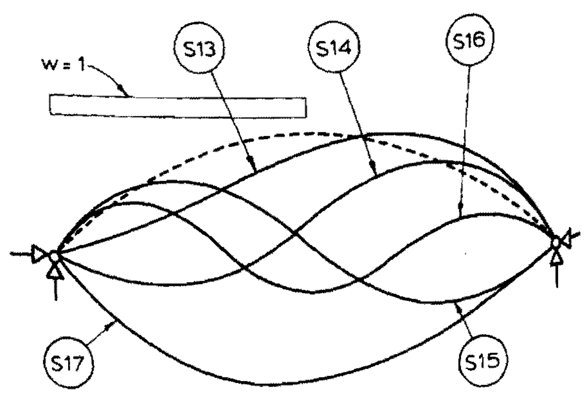


FIG. 9 DEFORMED SHAPES OF HALF LOADED PARABOLIC, PINNED ARCH (refer to Fig. 8)

An Ultimate Strength Formula for Braced Twin Arches

T. Sakimoto and S. Komatsu, The Ohio State University

Effective lengths for inelastic lateral buckling of through-type arches, composed of twin box-section ribs with a double Warren truss bracing system which are subjected to in-plane uniform loads, has been determined by utilizing an analogy between an arch and a column[1].

The non-dimensional slenderness parameter for twin arches is newly defined as follows:

$$\bar{\lambda}_y = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \quad \frac{K_e K_\beta K_l L}{r_y} \tag{1}$$

where K_e, K_β, K_l are effective length factors related respectively to end conditions associated with lateral bending of the arch, to the lateral constraint supplied by the bracing system and to the load direction during buckling, L is the bow-length of the arch and r_y is the radius of gyration of the cross section of an individual arch rib for lateral bending.

A designer can use the effective slenderness ratio to obtain an ultimate stress, σ_u, according to the column design formula of his choice. The inelastic lateral buckling strength of parabolic braced twin arches can be determined by the following equation:

$$P_u = \frac{2A\sigma_u}{l\sqrt{\frac{1}{16}\left(\frac{l}{f}\right)^2 + 1}} \tag{2}$$

where P_u is the ultimate uniform load distributed over the entire span, A is the cross-sectional area of an individual arch rib, l is the arch span and f is the rise of the arch.

The applicability of the proposed formula has been examined with a computer analysis for various arches of practical proportions with a computer program developed for large deflections of spatial inelastic frames by the writers[2].

It is shown that the stability criteria proposed may be sufficiently satisfactory for the preliminary stability analysis of through-type bridge arches. The equations may provide a basis for future studies of code requirements for inelastic lateral stability of steel arches.

References

1. Tatsuro Sakimoto, Elasto-Plastic Finite Displacement Analysis of Three Dimensional Structures and Its Application to Design of Steel Arch Bridges, Thesis presented to The Osaka University in partial fulfillment of the requirements for the degree of Doctor of Engineering, Feb., 1979.

2. Sadao Komatsu and Tatsuro Sakimoto, "Ultimate Load Carrying Capacity of Steel Arches," Journal of the Structural Division, Proc. of the ASCE, Vol. 103, No. ST12, pp. 2323-2336, Dec., 1977.

OTHER RESEARCH REPORTS

Buckling of Rail-Road Tracks

A. Chajes, University of Massachusetts

In recent years, due to increasing train speeds, railroad tracks with expansion joints are frequently being replaced by continuously welded tracks. As a consequence, when large increases in temperature occur, these new tracks are unable to expand freely, and buckling takes place. Experiments as well as actual track failures indicate that: (1) the tracks buckle laterally (2) the two rails and the ties buckle as a unit relative to the crushed stone ballast (3) the resistance of the ballast to lateral track movement is non-linear.

Although a rigorous analysis of track buckling requires the solution of non-linear differential equations, it is possible by means of the simple model, depicted in Fig. 1, to simulate the qualitative aspects of the phenomenon. The model consists of two rigid bars, hinged to immovable supports and to each other. The flexibility of the system is simulated by axial springs with stiffness K, a torsional spring with stiffness C and a non-linear restoring force Q that is due to the resistance of the ballast to lateral track motion. The axial force acting on the model is

$$P = K(\alpha TL - \tfrac{1}{2}\theta^2 L)$$

in which α = the coefficient of thermal expansion of the tracks and T is the rise in temperature.

Equilibrium of the model leads to the relation

$$T = \frac{2C}{K\alpha L^2} + \frac{Q_o(1-e^{-r\theta})}{2K\alpha L\theta} + \frac{\theta^2}{2\alpha}$$

A plot of this equation is shown by the solid line in Fig. 2. It is obvious from the curve that the system has an unstable post-buckling curve and that failure of actual imperfect tracts will consequently occur at temperature increases below the critical increase as indicated by the dashed curve.

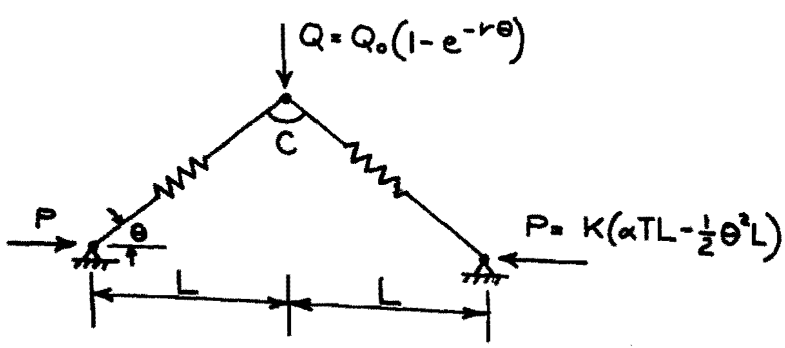


FIG. 1

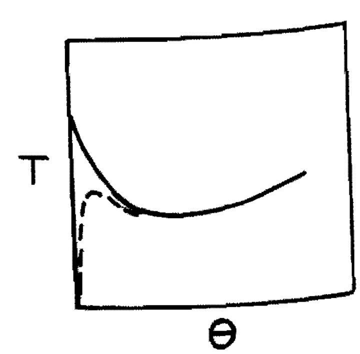


FIG. 2

Lateral-Torsional Buckling of T-Section Steel Beam-Columns

Shao-Fan Chen, Xian Institute of Metallurgy and Construction Engineering

When a T-section column is loaded eccentrically in its plan of symmetry, the load may be on the side of the flange (load with positive eccentricity) or on the side of the web tip (load with negative eccentricity). There is a vague view point that column under load with negative eccentricity is more liable to buckle lateral-torsionally because the shear center is on the flange plate. However, when we relate the average buckling stress σ_o (and there from the buckling coefficient $\phi_1 = \sigma_o/\sigma_y$) with eccentricity ratio ε = eccentricity/kern distance, we can see that, in the range of slenderness ratio commonly in use, the buckling coefficient ϕ, of T-section with negative eccentricity is greater than that with positive eccentricity. The main reason lies in that, except for slender members, lateral-torsional buckling takes place in elasto-plastic range. In case of negative eccentricity, when the compressive stress on the free edge of web plate exceeds the proportional limit σ_ρ, the lateral bending stiffness will not be harmed. In contrast, if the flange stress exceeds σ_ρ under load with positive eccentricity, this stiffness will be lowered.

In order to check the validity of our analytical work, an experiment program had been carried out at Xian Institute of Metallurgy and Construction Engineering. There were 24 test specimens altogether, 15 of them were welded T-section, and 9 were built-up with two angles.

The three double-angle specimens with positive eccentricity all failed by lateral-torsional buckling, whereas those with negative eccentricity failed by in-plane instability. This means that T-section columns with negative eccentricity have higher reserve in resisting lateral-torsional buckling. For welded T-sections, comparison of specimens with negative and positive eccentricity lead to similar conclusions.

Calculation of lateral-torsional buckling of T-section in inelastic range was investigated. For members with positive eccentricity, when the flange compressive stress exceeds σ_ρ, we can make use of the concept of tangent modulus by reducing the flange thickness from δ to $\delta E_t/E$, and then consider the column as an elastic member. For members with negative eccentricity, part of web may be yielded before lateral-torsional buckling load, taking into account the bilinear stress distribution. A more simplified approach to compute buckling load is to consider the T-section as wholly elastic. When yielding penetration is small, the results are quite satisfactory.

We have also worked on the calculation of lateral-torsional buckling of partially yielded beam-columns of I-section. Here we have to solve the problem according to the discontinuous process of slip plane formation. The lateral bending stiffness of the yielded part should not be taken as zero, but of some value E'. In order to take the effect of residual stress into account, we think feasible to adopt a pre-yield modulus of 0.3E instead of E, and a formula of determining E' is accordingly suggested. The modulus E' is then considered as a tangent modulus, and approximate formula for buckling load has been derived. Agreement between calculation and test conducted at 1972 is satisfactory.

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

BRIDGE STABILITY PROBLEMS

MODERATOR: Gerard F. Fox, Howard Needles Tammen & Bergendoff

PANELISTS: Bernard P. Wex, Freeman Fox & Partners
Heinz Nölke, Technische Universität Hannover
Herbert B. Rothman, Weidlinger Associates

BERNARD P. WEX - A British Viewpoint

In a short presentation to the Structural Stability Research Council on Bridge Stability Problems, coupled at the request of Gerry Fox with some comments on the new British Standard 5400 - for Steel, Concrete and Composite Bridges, I clearly cannot go deeply into theoretical matters. Indeed, I am neither an expert in stability theory nor even a member of any British Standard Committee responsible for drafting the new Bridge Rules. I have, of course, been able to refer to others who fulfil these roles.

However, throughout my professional life, I have been concerned with the design and construction of large structures, mainly in steel and, during the last ten years, with some very large steel bridges indeed. One of these, the Avonmouth Bridge, a major box girder structure, some 4,500 feet long, was opened to traffic in 1974. It was amended during construction, from its original BS 153 based design, to suit the Rules developed by the Merrison Committee, in which much of the new UK steel bridge code has its roots.

In this experience I have inevitably acquired an overview of the effects of rules and regulations upon design in the office, work in the fabricating shop, in the field and, in consequence, upon costs and completion schedules.

I have also learnt that those who draft Codes of Practice have an onerous task and a high responsibility. It is assuredly most difficult to digest a plethora of research material, structural theories, statistical data and the like, couple it with practical realism and regurgitate in a form readily usable by the practising bridge engineer.

These comments are particularly opposite to the question of steel box girder bridges and I will therefore direct the gist of this short address to that matter.

Steel Box Girder Bridge History

During World War II, stressed skin metal aircraft became highly developed. In Germany, after the war, where skilled and intellectual manpower was plentiful, while material was not, the box girder bridge formed of stiffened steel plates was a natural evolution from aircraft structural concepts. The box girder became popular with British designers also and a large number of these structures have been erected; to the best of my knowledge giving no real structural stability problems in service. However, four bridges failed during erection, one in Austria, one in UK, one in Australia, and one in Germany. The collapses resulted from instability problems, three arising in compression flanges and one in a pier diaphragm. These failures precipitated the vast amount of research of the last ten years into steel box girder problems. A great deal of that work was carried out in UK.

In common with other construction failures, a degree of vulnerability during erection was demonstrated by these accidents. Subsequent to the first three failures (the German failure occurred later), the British Government appointed a Committee, chaired by Dr. Merrison, a distinguished atomic physicist, with terms of reference reaching far beyond conditions and stresses relating to erection. The Committee, composed of very able people in their own specialties, contained no-one with first-hand practical experience of big bridge design or fabrication. All problems were considered ab initio. A most comprehensive, complicated and conservative set of interim rules emerged which underwent various modifications and corrigenda up to about 1972. Finally, by the end of 1973, these rules became "firm" as the Interim Design and Workmanship Rules (IDWR). These documents however were not really design specifications but were for appraisal, i.e., checking purposes in respect of both erection and "in-service" conditions.

All box girder bridges in UK had (and have) to comply, whether already existing, in course of fabrication (and/or erection), or about to be designed. The numerical task entailed for checkers was vast, and certain bridges are still being modified to accord with the rules. My firm recently won a design and build competition in which the design specification was the IDWR. We are currently working on the detail design of the bridge to these rules.

Another aspect of the recommendations of Merrison to Government was related to checking of steel box girder bridges. The "in-service" design had to conform with the rules and was subject to an independent check for compliance. The Contractor too had to ensure that stresses and conditions during erection conformed to the rules, and his work was similarly subjected to independent check. The complication alone of the rules merited these precautions if compliance was to be achieved.

Of course, the design of all structures should be checked, the more complicated the code, the stronger the case becomes. Now, in UK, all new bridge designs have to be checked whether in steel or concrete. In the case of major or complicated structures, these checks are carried out by firms independent of the original designers.

As can be imagined, the complexity of the Merrison Rules created no small disaffection among the bridge designers who had to operate them. However, several positive gains arose from the situation:-

PANEL DISCUSSION (Wex)

1. The Merrison Committee undoubtedly did a thorough job in identifying possible problem areas.

2. Through conferences, much debate arose.

3. A very wide programme of testing of elements and large scale models, coupled with theoretical studies, was undertaken in UK, largely at Government's expense. Many of the results supported designers' contentions that much of the rules was highly conservative and were instrumental in the partial modification of the rules.

4. The results of this work were to become available to the BS 5400 Committee for their task in writing Part 3 - Code of Practice for Design of Steel Bridges & Part 6 - Workmanship.

British Standard 5400

Current Position

The specification is in ten parts, eight of which were published:-

Part No.	Title	Date Published	No. of Pages
1	General Statement	1978	4
2	Specification for Loads	1978	43
4	Code of Practice for Design of Concrete Bridges	1978	48
5	Code of Practice for Design of Composite Bridges	1979	34
6	Specification for Materials and Workmanship - Steel	1980	15
7	Specification for Materials Workmanship Reinforcement and Prestressing Tendons	1978	11
8	Recommendations for Materials and Workmanship Concrete Reinforcement Prestressing Tendons	1978	19
10	Code of Practice of Fatigue	1980	53

Two sections are missing. Part 3 - Design of Steel Bridges was published as a draft for comment in August, 1979, and in that form contains about 200 pages which will doubtless condense considerably when finally published.

Part 9 - Code of Practice for Bearings, has yet to appear in draft. BS 5400 will indeed be quite a tome!

PANEL DISCUSSION (Wex)

A symposium to discuss Part 3 was held in London in January, 1980. A further conference on "The New Code for the Design of Steel Bridges" was held in Cardiff in March 1980. Unfortunately for me, owing to the pressures of real bridge building, I was unable to attend.

Concurrently, Government presumably mindful of the great difficulties of IDWR and the unfavorable reaction to complex codes by the profession and industry, has commissioned pilot studies of various parts of the draft code by several independent bodies. These studies are intended to reveal its useability, and the sort of results it gives compared with previous practice. This, in my view, will be a very valuable exercise if, as a result, the code drafters rationalize with lucidity and economy the complexities of the Part 3 draft.

As intimated at the London Symposium, some of these studies have indicated problem areas in the design of transverse stiffeners on longitudinally stiffened deep webs and in certain aspects of compression flange stiffener design.

I will attempt to comment only on a few detailed technical aspects relating to BS 5400. For some of my comments I am indebted to Dowling and Rockey for their permission to show slides made from their papers to the Cardiff Conference. Similarly, I am indebted to BSI for permission to show slides made from Part 6 - Workmanship.

Partial Safety Factor and Limit State Design

The new British code is based on limit state design using partial safety factors. The factors vary according to whether the limit state of serviceability or ultimate is being examined. They also are subject to variation depending upon the combinations of loading which are being considered, figs. 1 and 2. The basic loadings themselves are, in many cases, variable depending upon the loaded lengths being considered, e.g., HA loading and wind, and may vary between one loading combination and another, e.g., wind on the bridge alone or reduced wind on the bridge plus live load. Each loading condition has to be built up ab initio.

The factor for serviceability is generally unity, so that the resulting loads are "real" for the designer in the sense that they represent what could happen to the structure with combinations of loading envisaged, i.e., conditions much akin to the old allowable stress method of design. In general though, a serviceability check is not required for box girders designed in accordance with ultimate load requirements of Part 3.

The factored loads for ultimate conditions are "unreal" to the designer in the sense that they take account of unfavorable variations of design loadings which are already heavy and combine them in the manner indicated by figs. 1 and 2.

PANEL DISCUSSION (Wex)

A further complication arises in the material safety factor for ultimate conditions, which is generally stated in Part 3 to be 1.15 but in certain clauses has values ranging from 1.05 to 1.35. Some differ by only .05.

The basis of the philosophy lies in simplistic terms on the degree of overlap of the tails of histograms: What number of loads in excess of estimates will coincide with what number of structures having strengths below calculations? This is a very useful concept for estimating safety factors but it seems to me, in the present state of knowledge, not to be regarded as a refined technique. Subtle variations in safety factors, justified in abstract statistical theory, give the impression that bridge and indeed other forms of structural engineering are populated with experience of a high rate of failure due to under-estimation of loads coupled with over-estimation of strength. Such, however, is not the case. The vast majority of failures arise from errors or lack of appreciation of the manner in which the structure works. A large percentage of accidents, although relatively rare, occur during erection. Bridges are no exception.

The loading combinations in fig. 1 indicate at ultimate a factor of 1.15 x temporary erection loads in combinations 2 and 3. On Humber Bridge, a recent widely publicised accident occurred where 30 tons of temporary erection equipment fell about 250 feet onto the steel bridge deck. It was extremely fortunate that no one was killed and local damage only was done to the deck sections. Load factor or any other basis of design is irrelevant to events of this type. Yet I think every constructor will acknowledge that it is the sort of event which in erection constitutes a real hazard. Irradication is probably impossible - constant vigilance seems the only palliative. Study of the causes of error would seem to me more profitable than over-elaborate variations of loading combinations and safety factors.

Whilst I wholeheartedly support the philosophy of limit state and partial load factor design, there is no denying that its numerical complexity in BS 5400 is appreciable. When it is combined with all the new material within other parts of the steel bridge code, designers seem to face a formidable prospect. It is certainly to be hoped that in their final revision of the document the drafters can reduce the numerical work load and, in consequence, the room for error.

Analysis

In BS 5400 global analysis of bridge structures is based on elastic concepts, even for ultimate loading combinations. This means, in general terms, with redundant structures, a particular cross section may be designed to reach collapse under ultimate load: however, almost certainly there would be redistribution of load throughout the structure, thereby hindering total collapse. That fact is acknowledged in Part 3, but no numerical allowance is made for it because of the many uncertainties in so doing.

PANEL DISCUSSION (Wex)

By contrast, during erection, which frequently involves cantilevering, failure at one cross section only could mean collapse (two box girder accidents were of this type).

Only if heavy shear lag is present in flanges does Part 3 require a serviceability check. For the ultimate load condition which neglects shear lag because of plasticity, the effects of longitudinal stresses arising from restraint of torsional and distortional warping can be ignored. In other words, they disappear at ultimate. Shear stresses arising from torsion and transverse loading have to be considered.

Webs

In their paper to the Cardiff Symposium in March, 1980, entitled "Design of Stiffened Web Plates - State of Art Report", Rockey, Evans and Porter show a comparison obtained by an ECCS Committee of ultimate loads predicted by different theories, with actual test results from a number of independent authorities throughout the world. In the case of transverse stiffened webs, predictions from 44 tests by the 'Cardiff' method showed results on average 2% high, with a standard deviation of .06. For webs having transverse and longitudinal stiffeners, 66 tests achieved apparently a spot-on mean prediction with a standard deviation of .07. The mean predictions by other investigators were pretty accurate, but standard deviations were twice or more those associated with the Cardiff tests. Rockey quotes the following result for one of his box girder tests:

PREDICTED ULTIMATE LOADS			ACTUAL
IDWR	BS5400	CARDIFF	
160	128	167	182 tonnes

Concerning transverse web stiffeners, Rockey maintains that there is ample evidence to show at ultimate, full plasticity can be permitted in the transverse stiffener without impairing seriously its effectiveness. BS 5400 Part 3, on the other hand, requires the stiffener to remain elastic, thereby considerably increasing its required cross section.

Rockey's paper makes it clear that the problem of longitudinal stiffeners in webs suffering high shear and bending, is not solved with respect to the design of longitudinal stiffeners.

Compression Flanges

Dowling's paper to the Cardiff Symposium reviews Compression Flange Design Philosophy, as contained in the codes of USA, Switzerland, Germany, Czechoslovakia and UK. The Czechoslovakian and Swiss documents are established codes, while all the others are drafts for public comment.

PANEL DISCUSSION (Wex)

The British code analyses compression flanges as a series of isolated struts whose strength is assessed by inelastic beam column methods. Limitations are set on stiffener proportions to inhibit local buckling before the total strut strength is achieved. Residual compressive stresses arising from welding are assumed to be 10% of yield, which in Dowling's practical researches is demonstrated to be adequate for the tolerances on plate panel construction required under the Part 6 Workmanship Rules.

Dowling emphasizes the insignificance of shear lag effects demonstrated at last by practical tests and sophisticated theoretical analysis. The relative unimportance of shear lag upon flange ultimate capacity in box girders has long been maintained by designers in my firm, and it is gratifying to see that view so clearly corroborated.

An area of possible concern to which his paper draws attention might perhaps occur in bottom flanges adjacent to pier diaphragms in continuous bridges. Here, if diaphragms oversail the bearings, significantly, large compressive stresses mutually at right angles can develop. Studies of such stress systems have been carried out at Imperial College. BS 5400 Rules currently do not propose a check should be carried out in this area.

Perhaps the most interesting commentary in the paper comes from comparative designs of four box girders carried out in accordance with the five national codes, by designers each experienced in the use of the code. Fig. 3 illustrates the specimens examined which each contain the same amount of material, and the answers obtained.

It is interesting that the American Rules, based I believe largely on British research, give higher values than the draft British code for specimens 1 and 3 by an appreciable amount. Indeed, it is only in the case of Box specimen No. 2 that the American Rules do not predict higher than any other code. All the codes predict Box 2 to be capable of carrying the largest load.

Mises-Henckey Equivalent Stress Criterion

The Merrison Rules made much use of the Mises-Henckey equivalent stress criterion to assess the effects of direct membrane stresses coexisting with shear. The predicted results for flange ultimate loads were very low. One of my partners conceived a test type in which a box beam was, in effect, used as a balanced cantilever, so proportioned that the effects of high and low shear could be simultaneously examined with the same bending moment, until collapse. Figs. 4 and 5 show the sort of set-up. Figs 6 and 7 give the plots of behavior up to ultimate. Merrison serviceability limits are indicated, as are collapse predictions by BS 5400 Part 3. Out of three tests, two of the failures were on the low shear side of the test specimen. Much better predictions of collapse in each case were obtained by treating the flange as a series of tee-section struts formed by applying the BS 153 welded plate width criteria and the Perry Robertson strut formula, but totally ignoring shear in the flange plate.

PANEL DISCUSSION (Wex)

The results of all the computations plotted on figs. 6 and 7 employed the actual known yield strength of the plate and stiffeners.

My firm has had considerable reservations about the applicability of the Mises-Henckey criterion to this type of problem. Those doubts were confirmed by these tests. Indeed, the draft Part 3 requires only half the peak shear stress in flanges to be considered (provided it is not induced by torsion).

It would indeed be interesting to hear in the discussion if anyone has conducted tests to ultimate in which stiffened plate stability in compression has been examined with high coexistent shear.

Stiffener Shapes

A few words concerning stiffener shapes are perhaps not out of place in this gathering. Theoreticians and, let it be admitted, some practical men, have a preference list for stiffeners running something as follows: closed stiffeners, tee stiffeners, angle stiffeners, bulb flats and flats. In terms of inertia and outstand stability, this order is understandable. However, fabrication and site splicing considerations can make flats and bulb flats attractive, at any rate for longitudinal stiffeners. The trough when applied to orthotropic steel decks requires much more care in workmanship to avoid fatigue problems, and its splice is more difficult. However, for the orthotropic application, its use is invaluable, justifying its difficulties. Limitations on stiffener dimensions from theoretical stability aspects are currently under investigation for Part 3.

In relation to splices, BS 5400 says both surfaces of spliced parts should be provided with cover material. If not, the effect of eccentricity is to be considered when determining the strength of the cover material and the part. However, it does not specify how. Fig 8 illustrates a bulb flat stiffener splice during the times of "Merrison stiffening". The double-covered splice shown was adopted to avoid xx and yy eccentricities, thereby avoiding protracted arguments as to eccentricity. We much preferred a single covered splice and had two or three tests executed to see whether such a splice in a stiffened panel behaved any differently from a panel identical in all respects except the stiffeners were continuous. There was virtually no discrepancy of result between the two. Figs. 9 and 10 illustrate the test pieces and the results. BS 5400 and BS 153 predictions of ultimate load have been indicated, for interest, assuming a steel yield of 23 tons psi. It is interesting that in this case both methods predict virtually the same load.

As can be seen from the illustration, a double covered splice not only entailed twice the work of the single cover, it also resulted in a nasty unpaintable pocket between the two splice plates which had to be filled with sealer. This example provides a clear illustration of the disadvantages, both in first cost and maintenance terms, of an over-theoretical approach.

PANEL DISCUSSION (Wex)

Tees and angles are more difficult to splice. One has seen details where the stiffener has been neatly and symmetrically spliced but the flange plate has been left quite unsupported over a significant length. In such a situation, almost inevitably a full penetration transverse butt weld would exist, producing significant plate distortion; a dangerous condition might arise in heavy axial compression.

Conclusions

From the foregoing comments I hope the following points may be gleaned.

A great deal of practical and theoretical research has been done into box girder problems. In consequence, the behavior of this type of structure is, for the most part, well understood.

Theoretical predictions of ultimate load are best respected by practising engineers when the results consistently accord well with test data. One accepts that simplified procedures may give somewhat poorer agreement than more rigorous treatment. It is disconcerting if the reverse is found to occur.

Engineers have in the past sometimes compared one national code with another. I believe that in future such comparisons will be more systematic and far reaching in their effects. A national code of a major developed country carries the cachet of authenticity, especially to a developing nation, whether rich or poor. The national code which results in least cost, i.e., optimises material, workmanship and design costs, is going to be very important in the competition for world markets. The country practised in using that code in its own home markets is going to have a considerable advantage.

Against this background, code drafters cannot of course afford to be unsafe, but neither can they afford to be too conservative or too complicated, if they are best to serve the interests of their country.

Indeed, code drafters do have a most difficult and responsible task, to integrate successfully, knowledge and experience, from theory, testing, design, construction and long term behavior.

BS 5400 LOADING COMBINATIONS - ULTIMATE

PRIMARY EFFECTS

1. [1·05 D.L. (Steel) + 1·15 D.L. (Conc.) + 1·75 D.L. (Superimposed)] = Factored D.L's
+1·50 Earth Pressure +1·50 L.L. (HA) or 1·30 L.L. (HB)

2. Factored D.L's + [1·10 Wind (Erection) + 1·15 Temporary Erection Loads]
+1·40 Wind (D.L. Only or Members Primarily Resisting Wind.)

2. Factored D.L's + 1·10 Wind + 1·50 Earth Pressure + 1·25 L.L. (HA) or 1·10 L.L. (HA+HB)

3. Factored D.L's + 1·30 Temp. Restraints + 1·0 Temp. Differentials
+1·50 Earth Pressure + (1·15 Erection Loads) + 1·25 L.L. (HA) or + 1·10 L.L. (HA+HB)

NOTE:- For all cases factor for D.L. Steel D.L. Conc. & D.L. Superimposed to be 1·0 if this produces more severe effect.

SERVICEABILITY

Combinations As Above

Factors are generally 1·0 except 1·20 D.L. (Superimposed) & 0·80 Differential Temp.

Fig. 1

BS 5400 LOADING COMBINATIONS - ULTIMATE

SECONDARY EFFECTS

4. Factored D.L's + 1·50 Earth Pressure + 1·50 Centrifugal & Assoc. Primary
+1·25 Long'l & Assoc. Primary HA or 1·1 Long'l & Assoc. Primary HB
+1·25 Accidental Skidding & Primary L.L.

5. Factored D.L's + 1·30 Friction Restraint from Bearings + 1·50 Earth Pressure.

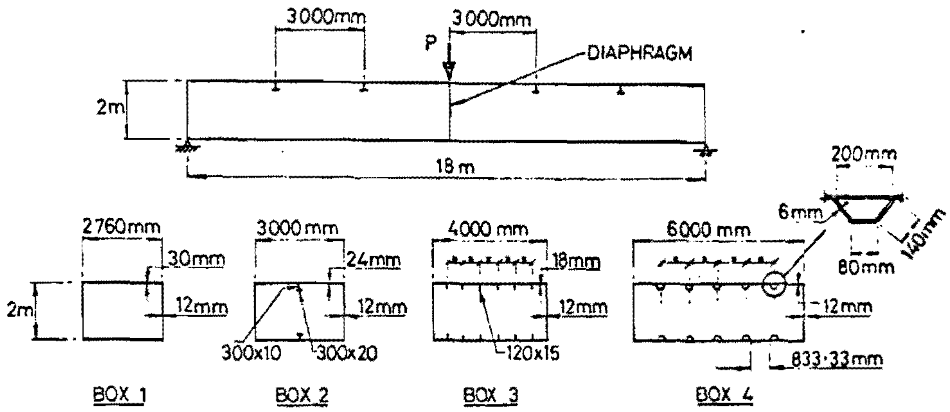
NOTE:- For all cases factor for D.L. Steel D.L. Conc. & D.L. Superimposed to be 1·0 if this produces more severe effect.

SERVICEABILITY

Combinations As Above

Factors are generally 1·0 except 1·20 D.L. (Superimposed) & 0·80 Differential Temp.

Fig. 2



Box	British Rules	Czech Rules	German Rules	Swiss Rules	U.S. Rules
1	3821	4151	3606	4755	5359
2	6921	6326	6730	7580	7266
3	4178	4242	*	*	5565
4	4962	4696	*	*	5025

* — VALUES NOT YET AVAILABLE

TABLE: TENTATIVE ULTIMATE LOAD VALUES OF P (kN) FOR BOXES

Fig. 3

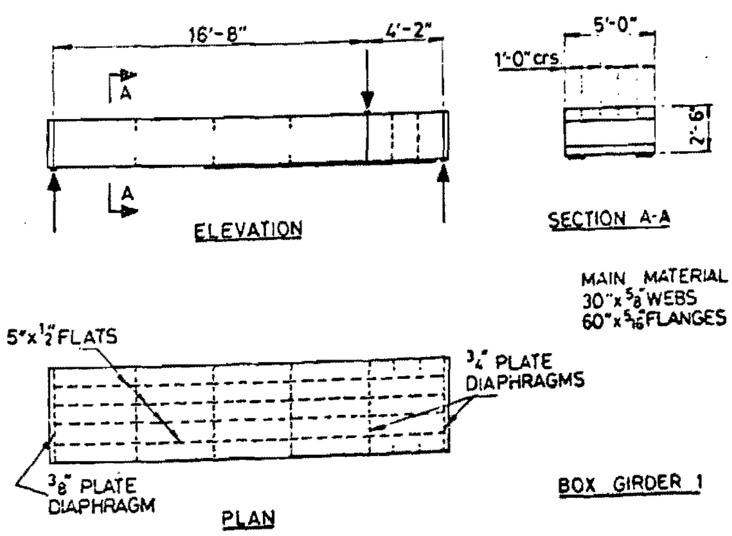


Fig. 4

PANEL DISCUSSION (Wex)

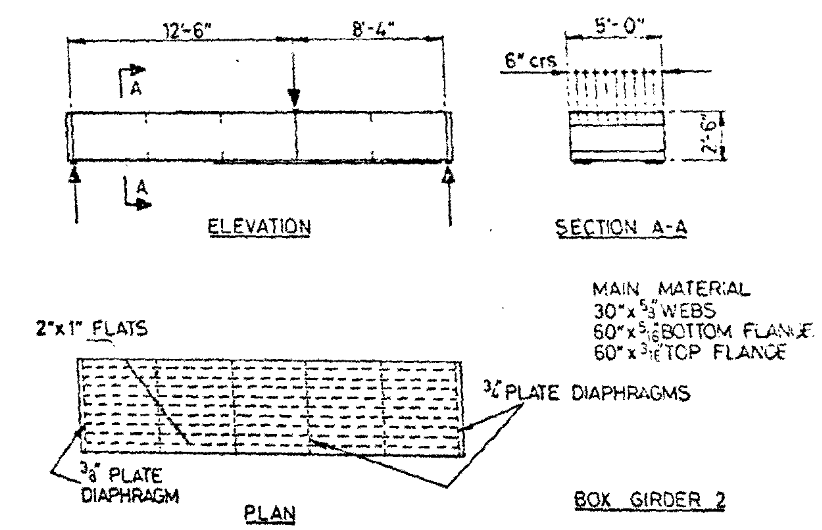


Fig. 5

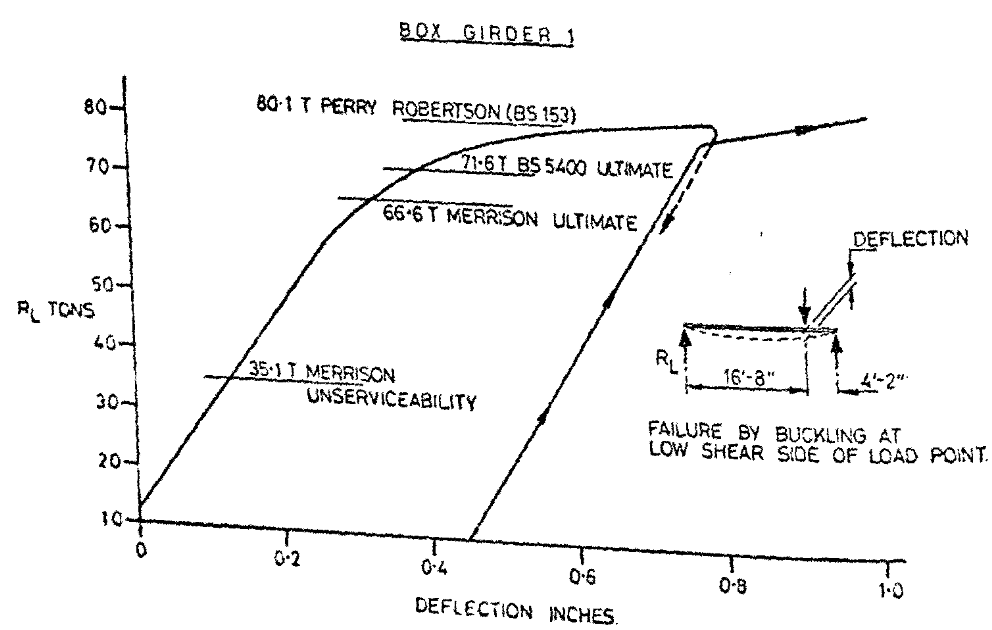
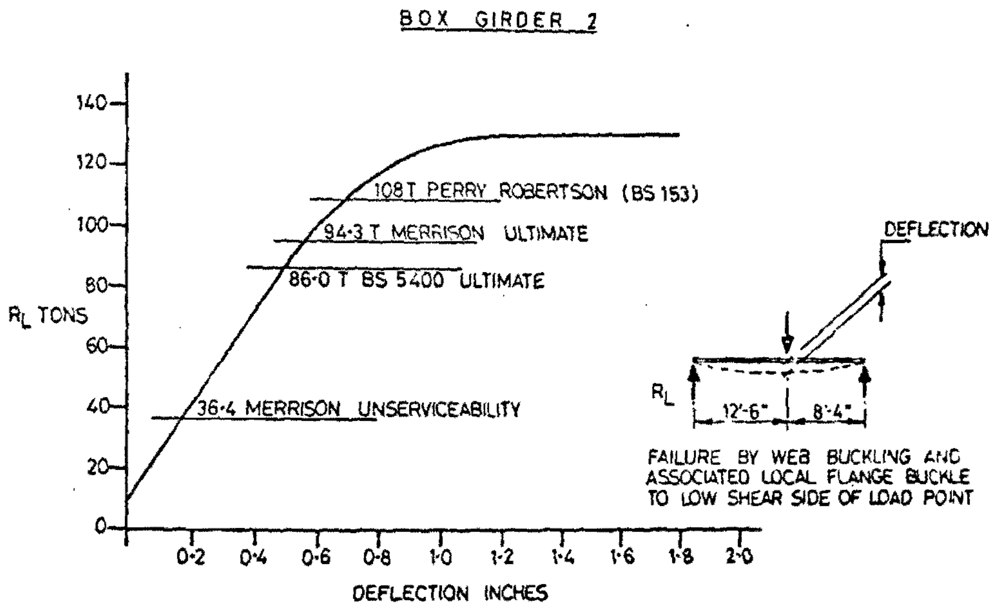
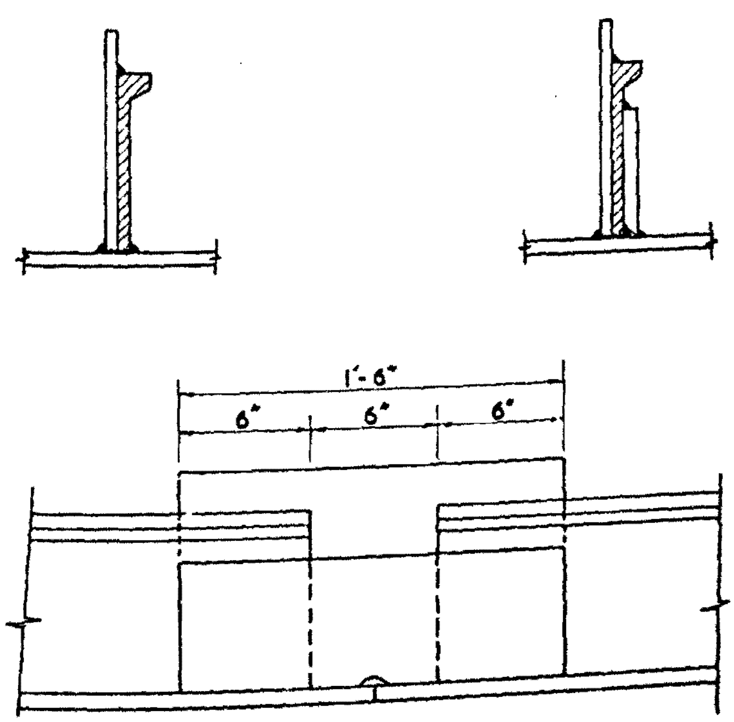


Fig. 6



BOX GIRDER 2
140
120
100
80
60
40
20
0
R_L TONS
108 T PERRY ROBERTSON (BS 153)
94·3 T MERRISON ULTIMATE
86·0 T BS 5400 ULTIMATE
36·4 MERRISON UNSERVICEABILITY
0·2
0·4
0·6
0·8
1·0
1·2
1·4
1·6
1·8
2·0
DEFLECTION INCHES
DEFLECTION
R_L
12'-6"
8'-4"
FAILURE BY WEB BUCKLING AND ASSOCIATED LOCAL FLANGE BUCKLE TO LOW SHEAR SIDE OF LOAD POINT

Fig. 7



1'-6"
6"
6"
6"

Fig. 8

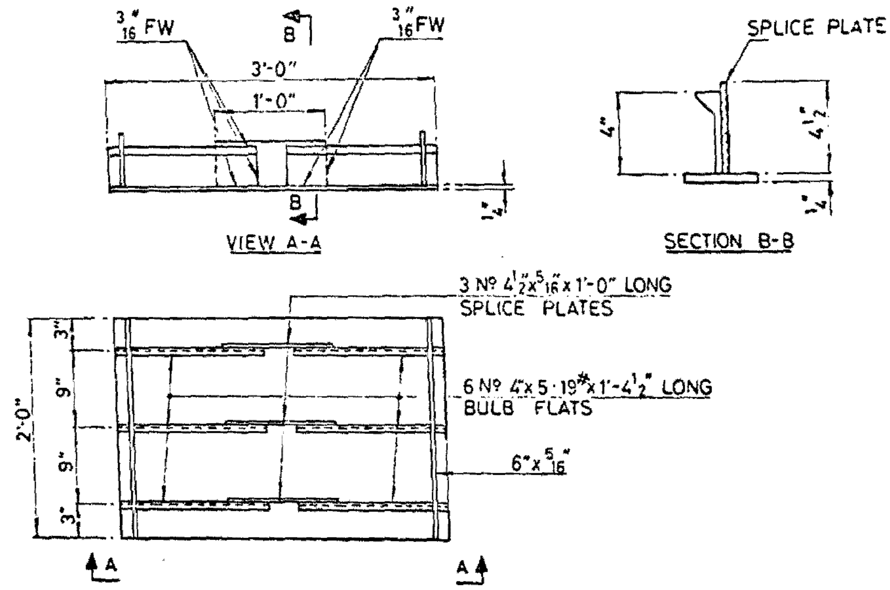

3/16" FW
3/16" FW
3'-0"
1'-0"
B
B
1/4"
VIEW A-A
SPLICE PLATE
4"
4 1/2"
1/4"
SECTION B-B
3 Nº 4 1/2"x 5/16" x 1'-0" LONG
SPLICE PLATES
6 Nº 4"x 5·19# x 1'-4 1/2" LONG
BULB FLATS
6" x 5/16"
2'-0"
3"
9"
9"
3"
A
A

Fig. 9

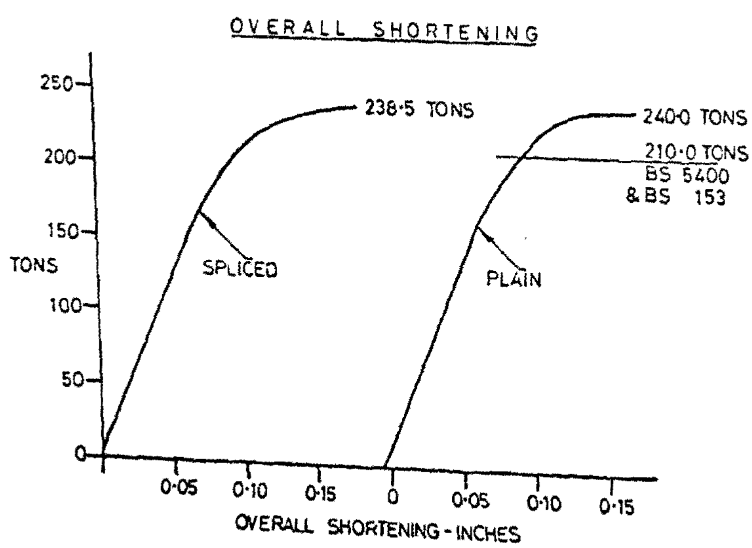

OVERALL SHORTENING
238·5 TONS
240·0 TONS
210·0 TONS
BS 5400
& BS 153
SPLICED
PLAIN
TONS
250
200
150
100
50
0
0·05
0·10
0·15
0
0·05
0·10
0·15
OVERALL SHORTENING - INCHES

Fig. 10

PANEL DISCUSSION

HEINZ NÖLKE

Certainly there is need to know about the ultimate load of steel bridges. In 1972 the German plate buckling committee began to revise the plate buckling design rules, and this committee was faced with the fact that many problems arising with the ultimate load design were unsolved.

This presentation outlines the course which has been adopted in the field of plate buckling problems. Some of the current tendencies in the Federal Republic of Germany and some of the most important approximations will be discussed.

Referring to stability problems the following notations are used (Fig. 1):

- The (traditional) slenderness ratio λ_c of a column depends on the Young modulus E and the critical stress $\sigma_{cr,col}$ of the column. It is well known that λ_c is equal KL/r, where K is the equivalent-length factor, L is the length, and r is the radius-of-gyration of the cross section.

- The reference slenderness ratio λ_Y depends on material properties only, these are the Young modulus E and the yield stress σ_Y.

- A normalized slenderness ratio $\bar{\lambda}_c$ is defined by the quotient λ_c/λ_Y.

Corresponding expressions for plates are defined analogously. We have only to replace the critical stress $\sigma_{cr,col}$ of the column by the critical stress $\sigma_{cr,pl}$ of the unstiffened or stiffened plate. In case of stress components acting simultaneously, $\sigma_{cr,pl}$ is the equivalent critical stress according to v. Mises criterion.

	Column	Unstiffened or Stiffened Plate
Slenderness Ratio	$\lambda_c = \pi\sqrt{\frac{E}{\sigma_{cr,col}}} = \frac{KL}{r}$	$\lambda_{pl} = \pi\sqrt{\frac{E}{\sigma_{cr,pl}}}$
Reference Slenderness Ratio	$\lambda_Y = \pi\sqrt{\frac{E}{\sigma_Y}}$	$\lambda_Y = \pi\sqrt{\frac{E}{\sigma_Y}}$
Normalized Slenderness Ratio	$\bar{\lambda}_c = \frac{\lambda_c}{\lambda_Y} = \sqrt{\frac{\sigma_Y}{\sigma_{cr,col}}}$	$\bar{\lambda}_{pl} = \frac{\lambda_{pl}}{\lambda_Y} = \sqrt{\frac{\sigma_Y}{\sigma_{cr,pl}}}$

Fig. 1

PANEL DISCUSSION (Nölke)

Fig. 2 intends to outline the scope of the proposed German plate buckling specifications. These will consist of three chapters. Chapter No. 1 was already published in 1978 (1) and also a commentary (2) which deals with the physical background, examples, and many references. These specifications are applicable to unstiffened, stiffened or multi-stiffened plates in all types of steel structures. We don't make any distinction between bridges, buildings, cranes, crane-runways, sluice gates, and others when the stability of steel plated elements is checked. The design method of this chapter is very simple. It consists in the main of two steps, the first of them leads to the normalized slenderness ratio λ_{pl} of the plate and the second one to the ultimate stress σ_u or to the normalized ultimate stress $\sigma_u = \sigma_u/\sigma_Y$.

The elaboration of Chapters No. 2 and 3 has not yet been finished. Chapter No. 2 will cover plate girders transversely stiffened only and will be probably and mainly based on the ideas of Rockey (3) referring to the diagonal tension field theory. Chapter No. 3 will consider slender plate girders without stiffeners between the supports. The behavior of this type of plate girder - dinominated in Sweden 'HSI'- girder - was investigated theoretically at first by Höglund (4).

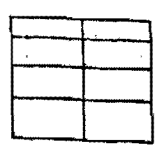
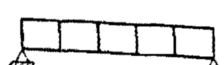
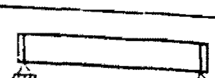
No.	Type of Structure	Basis of Calculation
1	 Unstiffened, Stiffened, Multistiffened Plates	1. Step: Normalized Slenderness Ratio $\bar{\lambda}_{pl}$ 2. Step: Stress at Failure or Limit of Serviceability $\bar{\sigma}_u$ (DASt 1978)
2	 Transversely Stiffened Plate Girders	Ultimate Load Design (Rockey)
3	 Slender Plate Girders without Stiffeners between Supports	Ultimate Load Design (Höglund)

Fig. 2

German Plate Buckling Design Rules

PANEL DISCUSSION (Nölke)

We are not going to adopt the diagonal tension field theory for box girders. One of the reasons is illustrated in Fig. 3, which shows the failure mode of a box girder loaded by a torsional moment. Collapse occurs when curved yield lines have developed and the longitudinal edges have failed.

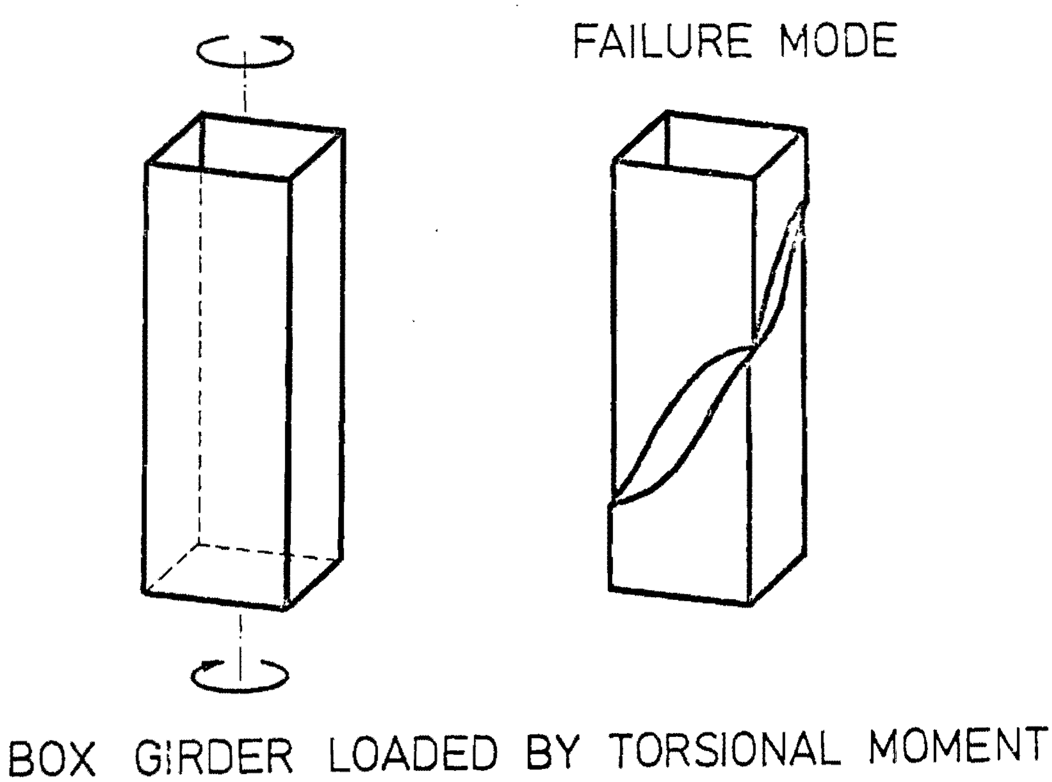


Fig. 3

Some years ago a lot of experimental tests with box girder models had been made (5) with the conclusion that the ultimate torsional moment and the accompanying failure mode cannot be predicted by the diagonal tension field theory.

While plate girders with transverse stiffeners only (Fig. 2, Chapter 2) and plate girders without stiffeners are characterized by only a few parameters, the general case of a multistiffened plate girder or box girder is much more complicated because the ultimate load depends on a lot of parameters. Let us return now to the latter case (Fig. 2, Chapter 1).

The task to predict precisely the ultimate load capacity of a multi-stiffened box girder seems to be extremely arduous.

PANEL DISCUSSION (Nölke)

The traditional and current engineering practice which has been adopted by the German specification committees, is to check all elements separately: the subpanels between stiffeners, the stiffened webs, the stiffened flanges, and so on.

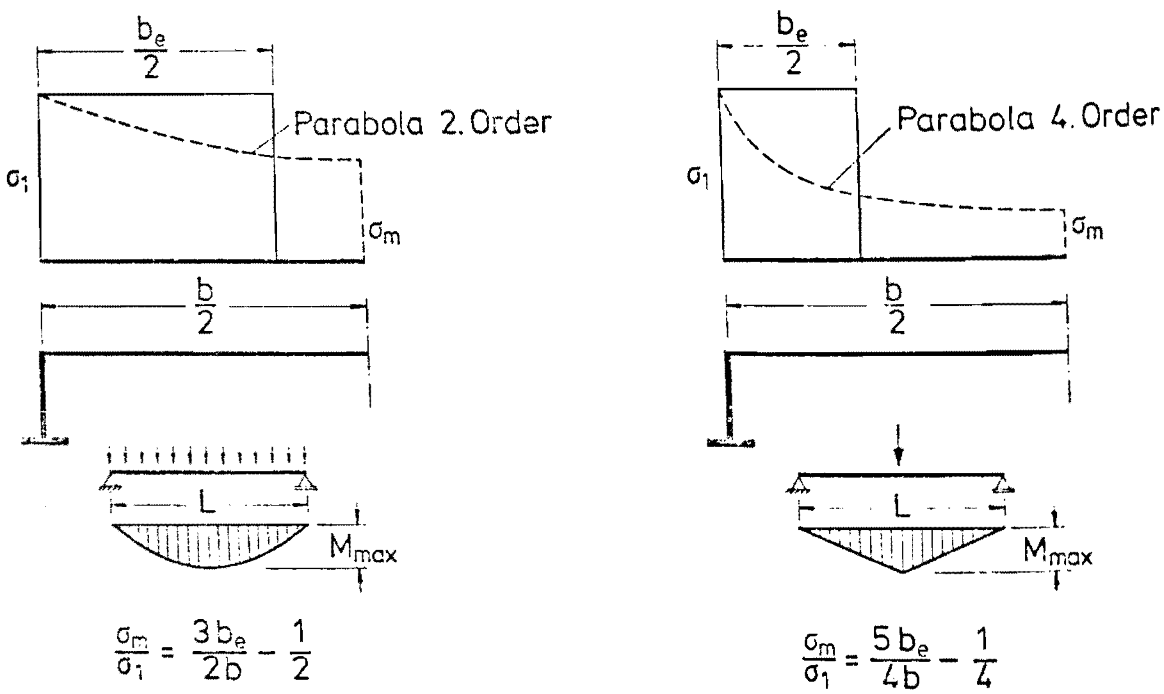


Fig. 4
Shear Lag Effects

The starting point of the calculations is an analysis of the inplane stress distribution. Fig. 4 shows the typical distributions of normal stresses in wide flanges due to shear lag effects. Parametric studies (6) based on the elastic plane theory have led to the following results: While in the case of uniformly distributed loads a parabola of order 2 is a good approximation of the stress distribution, in the case of a concentrated load it is a parabola of order 4. The effective widths b_e are given in charts (7). The normal stresses σ_m in the middle of the flanges are obtained from simple equations given in Fig. 4.

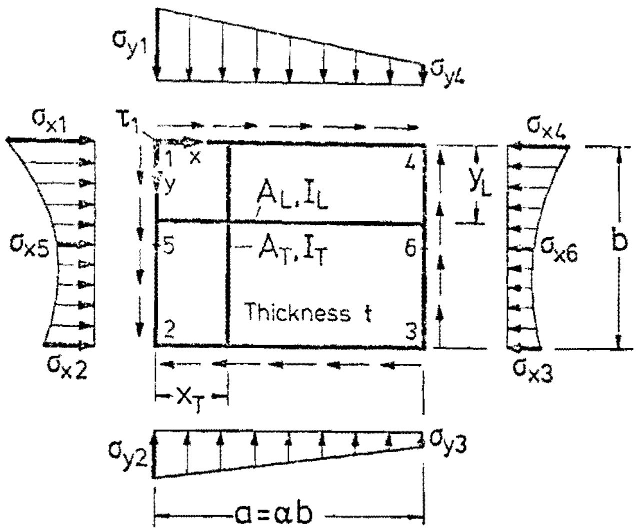


Fig. 5
Complex Edge Loading

Next after the inplane stress analysis the eigenvalue problem is to be solved. General formulae for the matrix elements of the eigenvalue problem for rectangular simply supported plates have been elaborated recently (8). They allow for rather arbitrary stress distribution, see Fig. 5, and an arbitrary number of longitudinal and transverse stiffeners (with cross sections areas A_L and A_T and moments of inertia of cross sections I_L and I_T). The solution of the eigenvalue problem leads to the buckling coefficient (new charts which allow for shear lag effects will be available very soon (6)), the critical stresses for stress components acting simultaneously, and the normalized slenderness ratio $\bar{\lambda}_{pl}$ of the plate.

1. Step: $\bar{\lambda}_{pl}$	2. Step: $\bar{\sigma}_u$
Geometry as Planned:	Yielding
Aspect Ratio	Imperfections
Properties of Stiffeners	Postbuckling Strength Reserve
Boundary Conditions of Plate	Interactive Buckling
Inplane Boundary Conditions:	
Distribution of Stress Components	
Stress Components Acting Simultaneously	

Fig. 6

PANEL DISCUSSION (Nölke)

The latter one is the most important result, because it is always possible to define a realistic plate buckling curve $\bar{\sigma}_u$ which is a function of $\bar{\lambda}_{pl}$. Fig. 6 outlines that on the basis of this simple approach all relevant parameters are taken into account; therefore a realistic result can be expected for all elements checked.

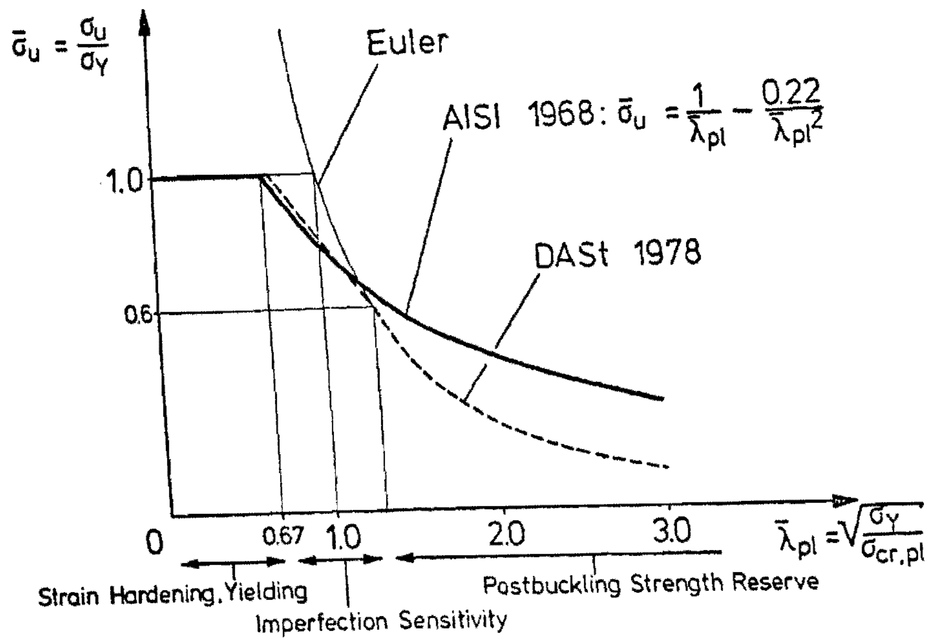


Fig. 7

Plate Buckling Curves

Fig. 7 shows a comparison between two plate buckling curves. The AISI-curve (9) p. 99, the formula of which is written here in terms of the normalized slenderness ratio $\bar{\lambda}_{pl}$, is plotted as a full line. The German plate buckling curve is plotted as a dotted line.

In the range of small values $\bar{\lambda}_{pl}$ the normalized ultimate stresses $\bar{\sigma}_u$ are nearly the same. This range is most important from a practical point of view. The German plate buckling curve follows at present still the Eul curve, if $\bar{\sigma}_u$ is smaller than 0.6. But I am sure the curve will be raised very soon, provided that the limits of serviceability will be defined. On Fig. 7 three ranges are distinguished in order to show the predominating effects; these are:

- strain hardening (always neglected) and yielding,
- imperfection sensitivity,
- postbuckling strength reserve.

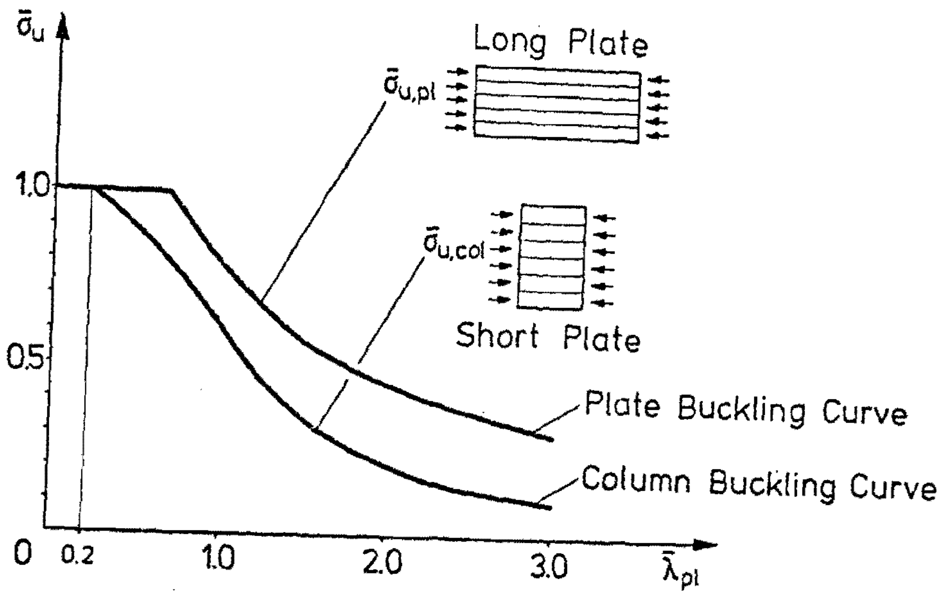


Fig. 8

Of course, the engineer must know the limits of validity of the methods he applies. One of the limits I would like to talk about to you is illustrated in Fig. 8. Let us consider a simply supported plate subjected to compression. For the normal case of a long plate, a plate buckling curve $\bar{\sigma}_{u,pl}$ may be specified. In the case of a short plate or a plate with strong longitudinal stiffeners, a nearly cylindrical buckling mode will occur. This means that then the behavior of the plate will approach to the behavior of a column. Therefore, the column buckling curve $\bar{\sigma}_{u,col}$ is a lower bound. This fact was already discussed in the SSRC Guide (9) p. 85. The question arises, what happens in the transition range? Obviously the ultimate stress cannot jump at once from the upper curve to the lower one.

Geometry	Buckling Coefficient Ultimate Strength
Long Plate	$k_{long} = k_{long}\ (B_x, B_y, H)$ Plate Buckling Curve $\bar{\sigma}_{u,pl}$ Valid
free "Short" Plate	$k_{short} = k_{short}\ (B_x)$ Column Buckling Curve $\bar{\sigma}_{u,col}$ Valid
Transition Range	Interpolation : $\bar{\sigma}_u = \bar{\sigma}_{u,pl} + (\bar{\sigma}_{u,col} - \bar{\sigma}_{u,pl})(2\,\frac{k_{short}}{k_{long}} - 1)$

Fig. 9

Columnlike Behaviour of Plates

Fig. 9 outlines the solution of the problem according to our specifications (1), (2). The buckling coefficient k_{long} of the long plate depends on the distributed bending stiffnesses B_x and B_y and the distributed torsional stiffness H. (The symbols are the same as in the Guide (9)). Instead of the so-called short plate, we can imagine a unit width of the original plate, the length of which being unchanged, the longitudinal edges being free, and the appertaining buckling coefficient k_{short} being a function of B_x only. If now the value k_{long} approaches to k_{short}, that means if the influence of B_y and H vanishes, the columnlike behavior of the plate will be more and more dominating. Therefore, there is no question that the transition formula must depend on the quotient k_{short}/k_{long}. We have chosen the linear interpolation given in Fig. 9; it holds for k_{short}/k_{long} greater 0.5.

Let us now turn to the problem of interactive buckling of a thinwalled column. It is well known that local plate buckling can reduce the overall buckling load of a column.

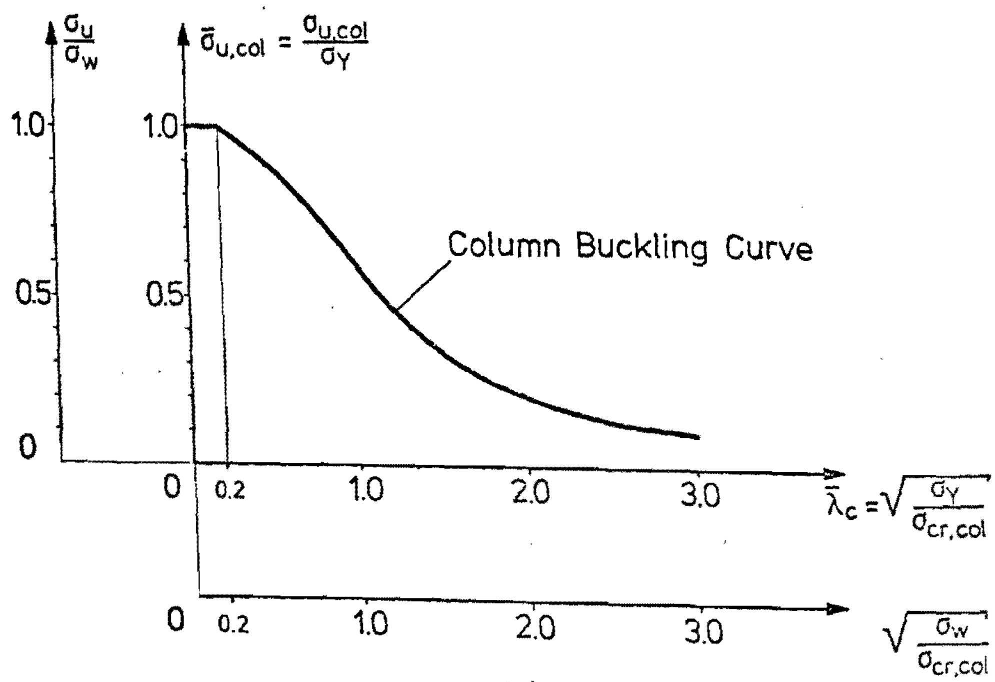


Fig. 10
Interactive Buckling (P.W. Marshall 1971)

A very simple and ingenious idea was published by P. W. Marshall in 1971 (10). He recommended to use the original column buckling curve for thinwalled columns with the modification that the yield stress σ_Y should be replaced by the local wrinkling stress σ_w in the appropriate column formulae. Fig. 10 shows the new notations, if this modification is applied to both axes. The local wrinkling stress σ_w can be determined experimentally with stub column tests or theoretically from the AISI formula, see Fig. 7.

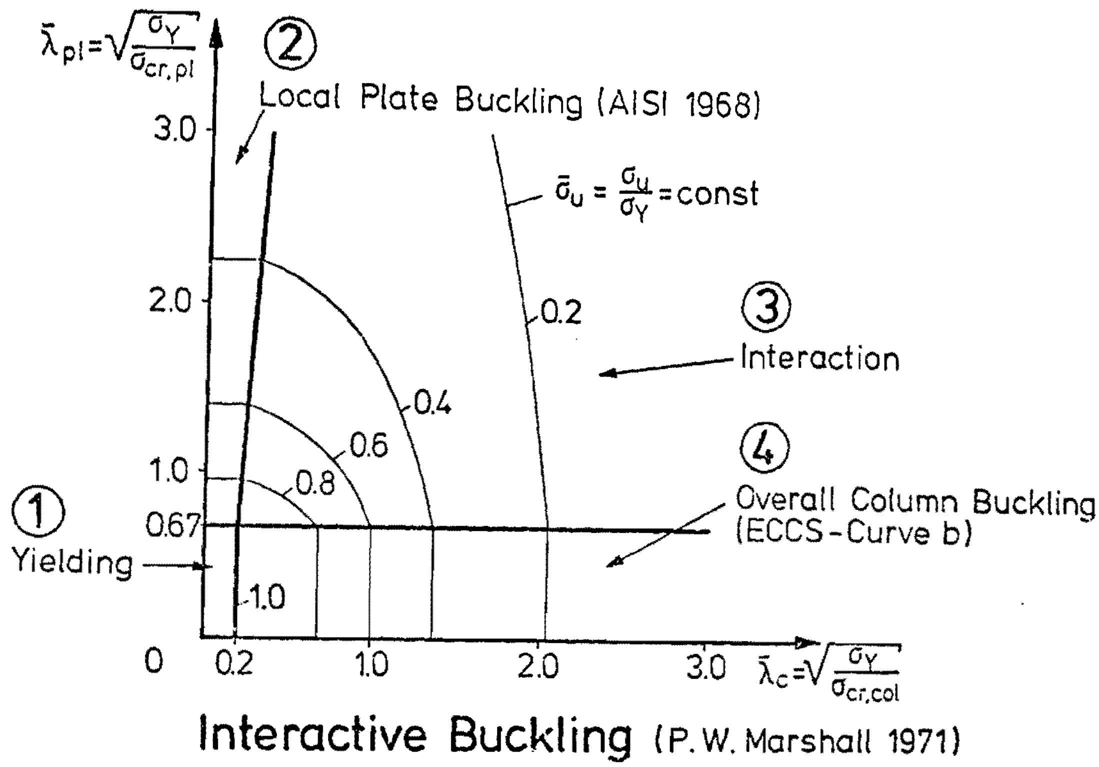


Fig. 11

For the purpose of simple practical applications I prefer diagrams from which the normalized ultimate stress σ_u can be read directly as a function of the two parameters, the normalized slenderness ratios $\bar{\lambda}_{pl}$ of the plate and $\bar{\lambda}_c$ of the column, see Fig. 11.

The curves of constant normalized ultimate stress $\bar{\sigma}_u$ = const are contour lines, and they are based here on the simple approach just being discussed. The diagram can be subdivided into four ranges; these are

- range 1: pure yielding,
- range 2: pure local plate buckling,
- range 3: interaction of local and overall buckling,
- range 4: pure overall column buckling.

If the assumptions for local plate buckling and overall column buckling are well-defined as being done here, the boundaries of these ranges and the curves will follow clearly.

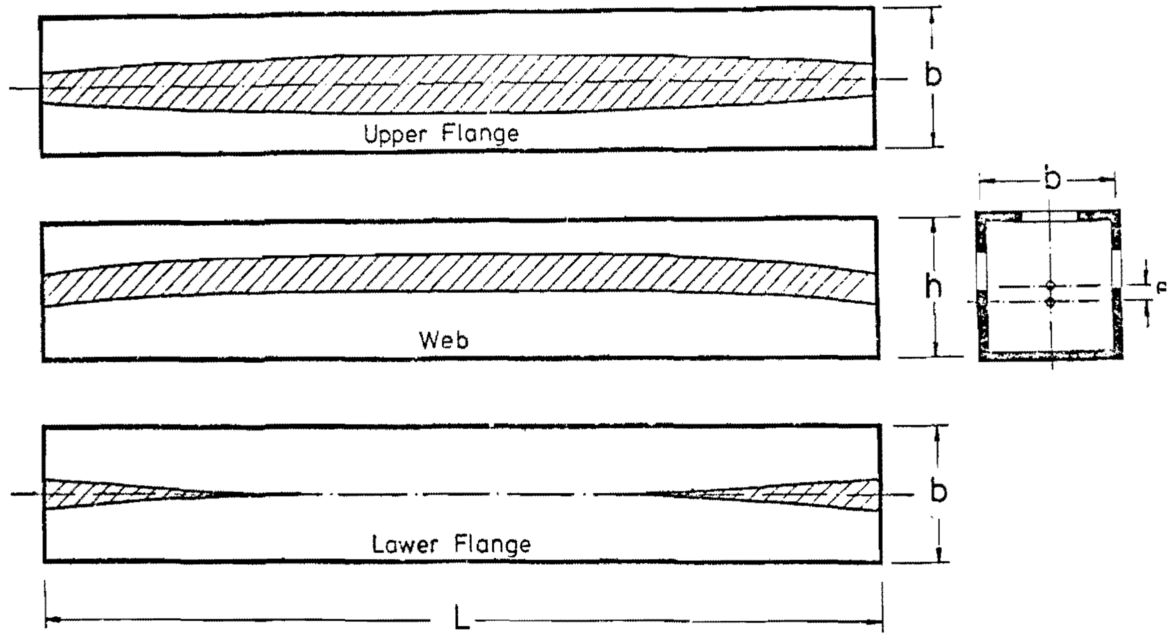
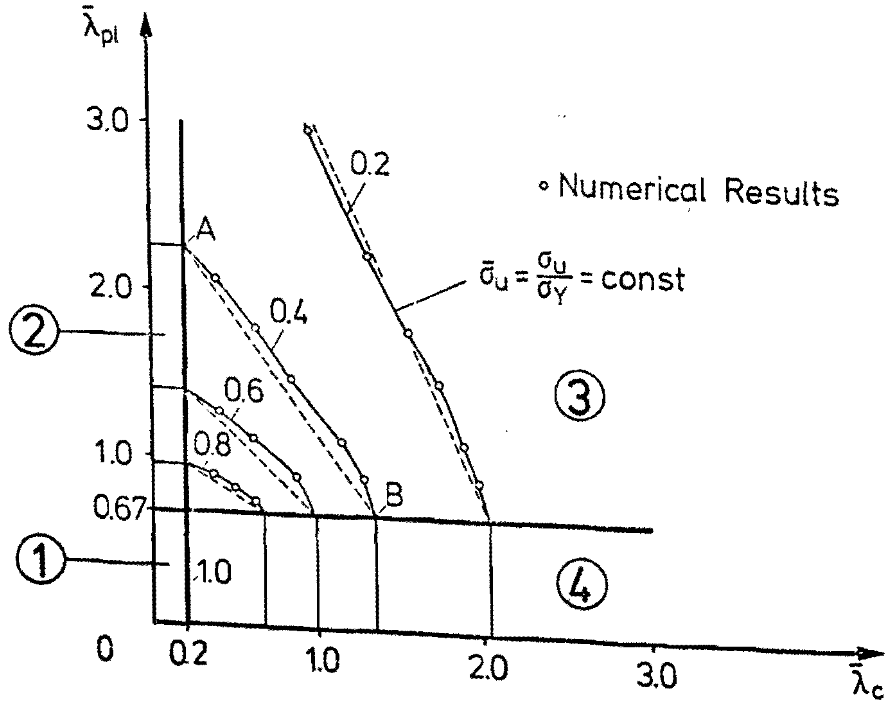


Fig. 12
Effective Widths

The German design rules (1), (2) referring to interactive buckling are based on the effective widths concept combined with the second order theory. Numerous parametric studies have been carried out taking into account that the effective widths vary along the length of the column. As an example, Fig. 12 illustrates the shaded non-effective areas of the flanges and the webs of a simply supported centrally loaded geometrically imperfect column. Similar results have been published by Skaloud and Naprstek (11).



Interactive Buckling (H.Nölke 1976,1980)

Fig. 13

Some of the numerical results are concentrated in Fig. 13. Again the normalized slenderness ratio $\overline{\lambda}_{pl}$ of the plate is plotted versus the normalized slenderness ratio $\overline{\lambda}_c$ of the column. The main results are:

- Again 4 ranges must be distinguished similar to those mentioned before.
- The simple approach mentioned before (see Fig. 11) is partly on the unsafe side, although both methods have been combined here with the same local buckling and overall buckling curves.
- In the interaction range No. 3 the numerical results should be approximated by straight lines between the points A and B (dotted lines in Fig. 13).

On the basis of these findings an interaction diagram with straight lines has been adopted in the German specifications. It holds for columns with thinwalled unstiffened plates or with stiffened plates.

The last figures of this presentation are devoted to problems which seem to be rather unsolved. Patch load acting on one of the longitudinal edges of a plate girder occurs for instance, when the girder is erected by launching. In spite of progress in estimating the ultimate patch load when it acts separately (12), (13), the solution for the interaction problem for patch loads between the transverse stiffeners and overall bending and overall shear forces is still missing.

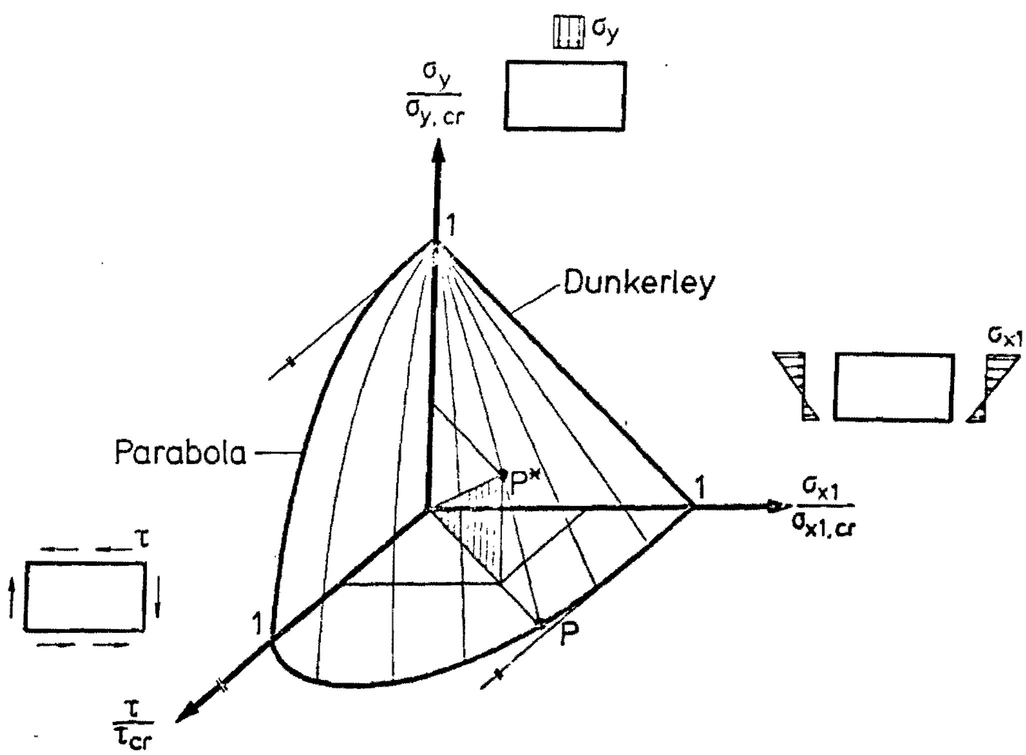
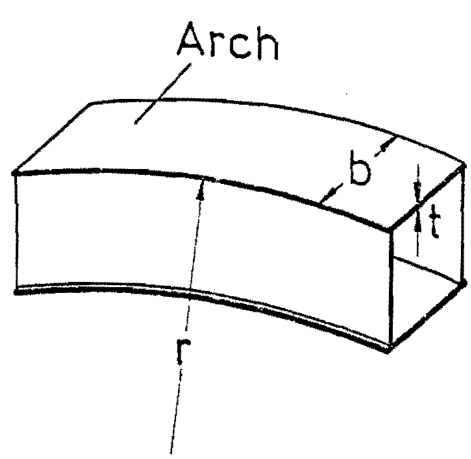


Fig. 14
Patch Load Interaction

Therefore, an interaction surface (Fig. 14) based on the solution of the eigenvalue problem has been included in our specifications. The three axes cover: normal stress due to bending moments and axial forces, shear stresses, and patch load stresses. The eigenvalue problem for patch loads acting separately has been solved by Protte (14).

Let us now switch to the local stability of curved plates. The flanges of an arch are continuously curved plates as planned. The curvature is not affinitive to the local buckling made. Such a type of planned curvature should be distinguished from the curvature due to geometrical imperfections.

The question arises: is the planned curvature always negligible when the local buckling of the flanges is checked? The limit of this simplification according to our specifications is mentioned in Fig. 15. It has been derived with the bending theory of shells and leads to approximately uniform distributed normal stresses. Further knowledge regarding this problem seems to be useful.



Curvature negligible if

$$r \geqq \frac{b^2}{t}$$

Fig. 15

Local Buckling of Curved Plate

In spite of all difficulties and problems we are grappling with, one should emphasize and recognize that steel bridge design operates on a high level of technology and is crowned with remarkable success.

PANEL DISCUSSION (Nölke)

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PANEL DISCUSSION (Rothman)

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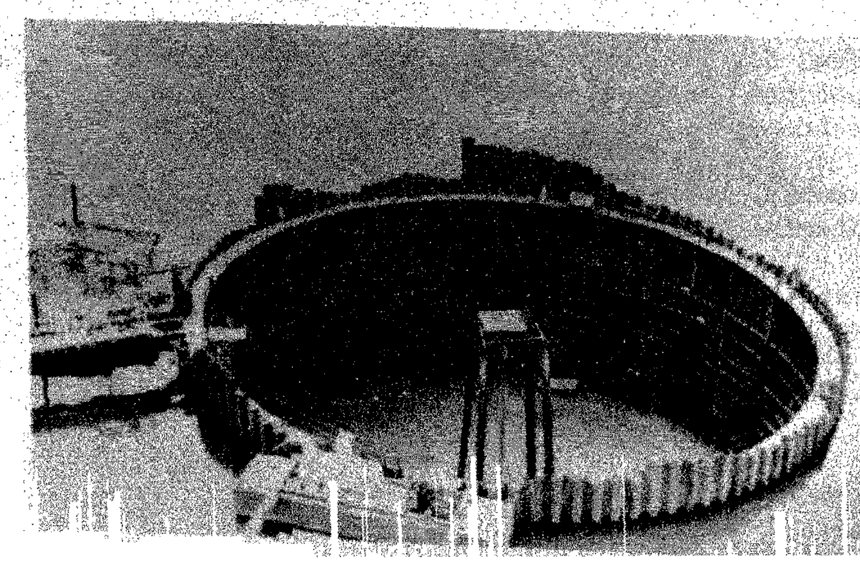
HERBERT B. ROTHMAN - Lake Keowee Cofferdam

Cofferdams for bridge piers have generally been rectangular braced single wall structures inside of which the permanent pier is built. Circular cofferdams have been used infrequently, because their advantages have only recently been recognized. Those that are built in great depths of water with little embedment in soil present major stability considerations. The cofferdam I will discuss tonight is, in fact, a reconstruction of one that had failed during pumpdown due to buckling.

A general view of the reconstructed Cofferdam is shown in Figure 1. It is a steel wall 120' in diameter sealed to the stream bed 50' below the water surface. It is designed so that the space inside can be pumped out and construction carried on in the dry. We believe it to be the largest of its kind ever built. At first glance it seems wasteful to use a circular cofferdam to construct a rectangular pier. However, the work space is unimpeded by internal bracing, a major advantage, and estimates show its cost to be less than for a rectangular, tight fitting cofferdam.



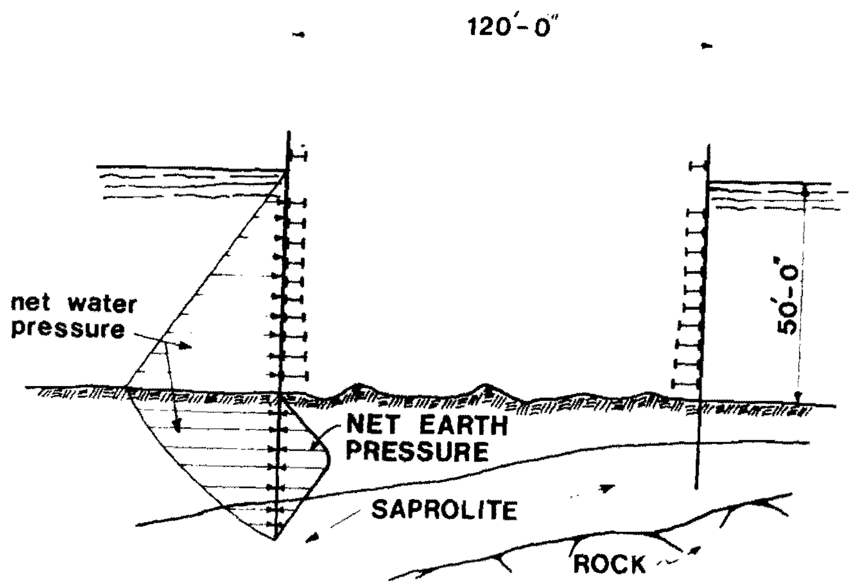


Fig. 2

Figure 2 is a schematic section of the new cofferdam. Except for the size and spacing of the ring wales, the first one was similar. While the pressures against the upper portion, due to standing water, is accurately known, loads on the lower portion are relatively uncertain, since they depend upon the water pressure distribution in soil and rock. These pressures in turn depend upon the variation of permeability of the residual soils (which had been disturbed by the collapse of the first cofferdam) and upon rock stratification. Lateral soil pressures against both sides of the embedded sheeting are still less certain since in addition to the variation of in place properties, their stress state depends upon the rate of water flow through the soil. Hydraulic and soils testing and analyses for the second cofferdam were done by Casagrande Consultants of Cambridge, MA. This work is not covered in this discussion.

However, regardless of load uncertainty, the failure of the first cofferdam was not due to an incorrect loading assumption, nor was it due to piping. The cofferdam simply buckled. It was not a progressive collapse, triggered by failure of a single member; the entire structure buckled at once. The weakest wales transferred their excess load to adjoining members through the sheeting, and when during pumpdown, aggregate load exceeded aggregate capacity, failure occurred.

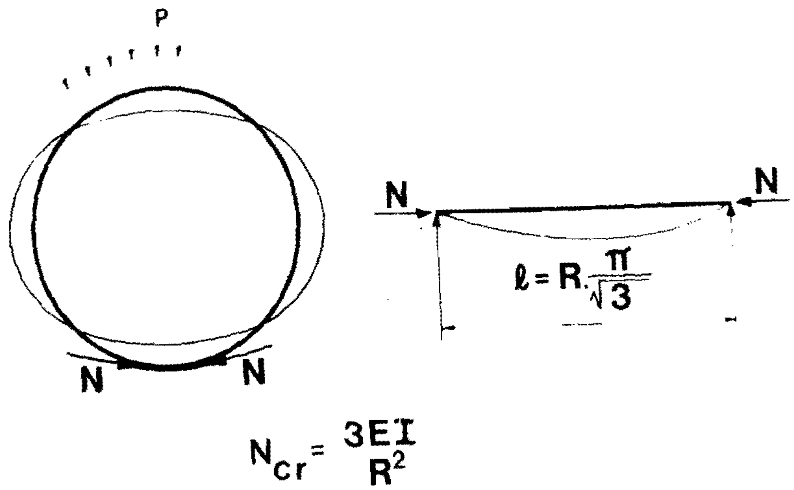


Fig. 3

Figure 3 shows that the lowest buckling mode for a ring is an oval shape, the critical external pressure q is $3EI/R^3$ and the critical ring compression is $3EI/R^2$. The failed wales were loaded to well above these values. Since these formulae assume elastic behavior, there is little doubt that the wales had insufficient buckling resistance under full load. It should be emphasized that the buckling failure occurred because the top 50 feet of the cofferdam was in water only. Had the lake bottom been higher, its load would have been greater but so would its buckling resistance.

The discussion that follows, covering the redesign of the cofferdam, does not directly apply to one in saturated earth. Design loads, stability, details and construction methods would all be different. Although excavation would increase, it is probable that its economy would still be favorably greater relative to rectangular structures.

Because the sheet piling size is determinated by the lateral water and soil loading on the embedded portion, it is quite stiff relative to the wales. As a result, its influence on the distribution of soil and water loads to the wales must be considered. Design loads were therefore determined by two dimensional elastic analysis of a slice of the radially symmetric structure using a range of lateral loads and soil stiffnesses. (Asymmetry of the foundation was considered separately.) Individual wales were then designed for this loading. There is a slight conservatism in this procedure, since the sheeting prevents buckling of individual wales, by permitting the weaker wales to work above their buckling loads, provided their neighbors have some reserve.

As shown in Figure 3, the length of a straight column that has the same cross section and the same buckling strength in the linear elastic range as a ring is $\pi R/\sqrt{3}$; about 90% of the diameter. Since the column simulates the ring when the radius is very large (elastic buckling) and also when the radius is very small (no buckling), it is intuitively appealing to take the straight column as a measure of ring behavior for the entire inelastic range. In this way all development in straight column design, including the vital empirical part, is directly transferred to the ring design. Design codes that vary the safety factor to account for initial crookedness clearly demonstrate that there is still a great measure of empiricism in column design. It is for such reasons that reliability based design requires calibration of design codes.

For the straight column to be a direct measure of ring capacity:

- inelastic behavior of the two must be similar;
- initial crookednesses must be the same;
- residual stresses should be the same.

Reasonable similarity of inelastic behavior for realized stress strain diagrams has been demonstrated and need not be pursued in detail for this discussion. The possibility of low post buckling strength as in shells has been raised, however. Such low strength is associated with some form of snap through which can only occur if there are restraints which permit very high axial compressions. Such restraints include the three dimensional behavior of a barrel (buckling in a checkerboard pattern) or the horizontal reaction components of a two-hinged arch. (No such non-moving nodes exist in a ring.) Because it lacks these restraints, the buckling load in a ring is not high enough to have a counterpart large deflection configuration to which it can jump. This was not demonstrated analytically in this design since even the existence of low post buckling capacity would not have changed the design.

Initial crookedness presents no major analytic problem since methods for designing for imperfections greater than those assumed (but not defined) by modern column formula are available. The effect of geometric imperfections was more significant, in fact, because of its impact on splice design. Butt splices are shown in Figure 4. They are economical for compresssion rings, provided they are not designed for the full member bending capacity. They were instead designed for actual thrust plus eccentricity about both axes, amplified by the compression.

Fig. 4

PANEL DISCUSSION (Rothman)

Because initial crookedness assumptions were a matter of economic importance, the design values used were the result of detail discussions with the fabricators and erectors. Wale shape was monitored in the field, even to the extent of making underwater surveys using divers and closed circuit television. The eccentricities in the plane of the ring were minimized by comparing each ring to a best fit circle. If a theoretical center and radius are used, unrealistically large eccentricities result. A similar expedient was available but unnecessary for out of plane deviation. These measurements of the assembled wales showed their accuracy to be of the same order as for a straight column, when the deviation is considered as a percent of the diameter. However, the measurements were made after the wales were designed and erected; design deviations were much greater. In general, combinations of weak and strong axis eccentricity ranging up to 3" were assumed. It is probably economical to continue to use larger values since the construction and surveillance methods needed to limit them can become excessive.

As Figure 5 shows, the cold bent rings (all but the bottom two which are welded) are actually superior to a straight column from the point of view of residual stresses. When the section is in the bending rolls, it is nearly completed at the yield stress level and all previous residual stress history is washed out. The residual stresses at this stage are therefore known. The relief of stress after rolling to be superimposed on the cold rolling stress is the elastic response to the full plastic moment. This too is accurately known. The resulting residual stress, the sum of the two, is thus quite reliably known. For the wales in this cofferdam, cold rolling leaves the flanges with a residual stress of about 10% of the yield point compared with 50% for straight columns. The only high residual stresses are in the web near the neutral axis where the effect is negligible.

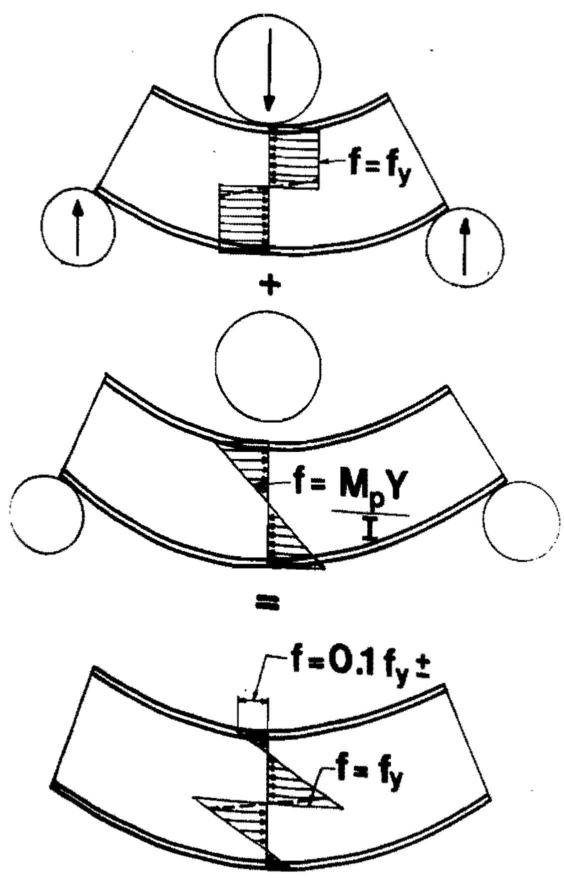


Fig. 5

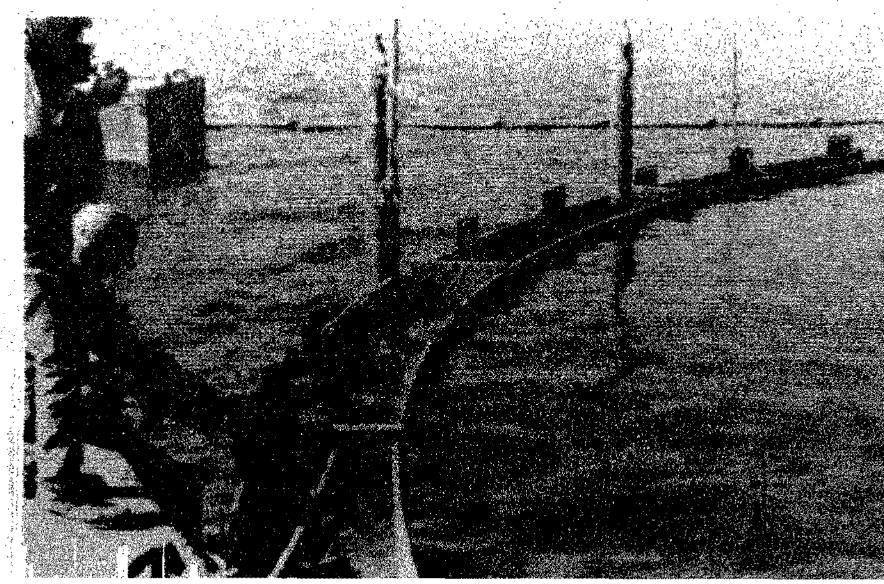
DISCUSSION (Rothman)

This favorable condition probably deteriorates at the segment ends ere it is further exacerbated by the welds to the butt plates. Further, : isn't possible to take full advantage of such low residual stresses since the material proportional limit, normally ignored, becomes a new lower limit which is less than 90% of the yield strength. For conservatism, therefore, the normal column formula was used. Another, perhaps over-riding need for conservatism lies in the fact that there is a small risk that a wale can be damaged by a concrete bucket, pile lead or some as-yet-to-be-invented construction accident.

We therefore concluded that for the strong axis behavior, the wales could be designed as straight columns having a length 10% less than the diameter of the caisson.

There is a similar albeit more complex linear elastic formula for the lateral buckling capacity of the wale about its weak axis. However, it showed that with closely spaced vertical supports, the wales could be taken as straight columns, and it also showed that the supports had better be closely spaced for stability. Unfortunately, these were the only things that were clear, and weak axis strength was the subject of more discussions than any other aspect of this project. This is because the capacity of the vertical supports was not certain, nor was the loading that would be imposed on them.

The loads imposed on the supports is a function of member waviness and, more importantly, of the out-of-plane tolerance of the supports in the cofferdam. Figure 6 illustrates the reason for the uncertainty of their out-of-plane accuracy. Final assembly including installation of the supports was by divers rather than by iron-workers. Because of difficult working conditions under water, what accuracy can be expected?



Fig

PANEL DISCUSSION (Rothman)

Figure 7 illustrates the reason for concern about the support strength. The flattened device on a pipe pile is a friction clamp which carries the wale. High tensile bolts squeeze the two halves of the clamp against the pile, providing the friction which enables it to support vertical loads. The connection must carry the wale weight plus the vertical component of the wale load caused by geometric inaccuracy. Can a friction clamp depending upon bolts tightened under water be trusted?

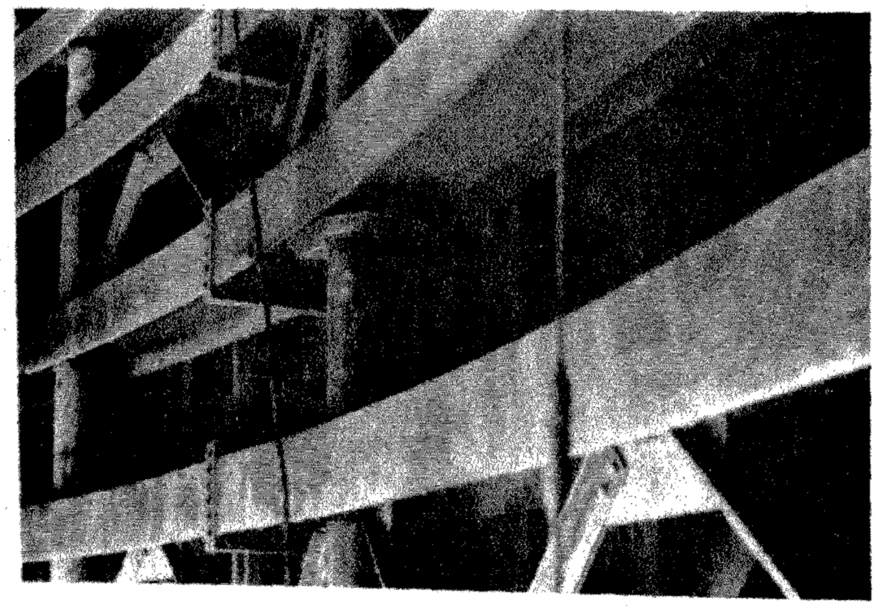


Fig.

These are not trivial questions; the most conservative answers, which were adopted because of the previous history of the cofferdam, added very greatly to its cost. The results of the conservative answers are the diagonal bracing shown in the figure, intended to increase lateral bending stiffness of the wales.

PANEL DISCUSSION (Rothman)

The relationship of the beam flange and the waterline in Figure 8 dramatically answers one of the questions: the vertical out-of-plane accuracy is exceptionally good and proved to be so throughout. The other questions were resolved by establishing a rigorous inspection procedure. The bolts were installed with load indicating washers, each of which were checked by inspectors. The bolts were then additionally tested under water before pumpdown with torque wrenches. With minor exception, the second test showed that the bolts had been tightened sufficiently.

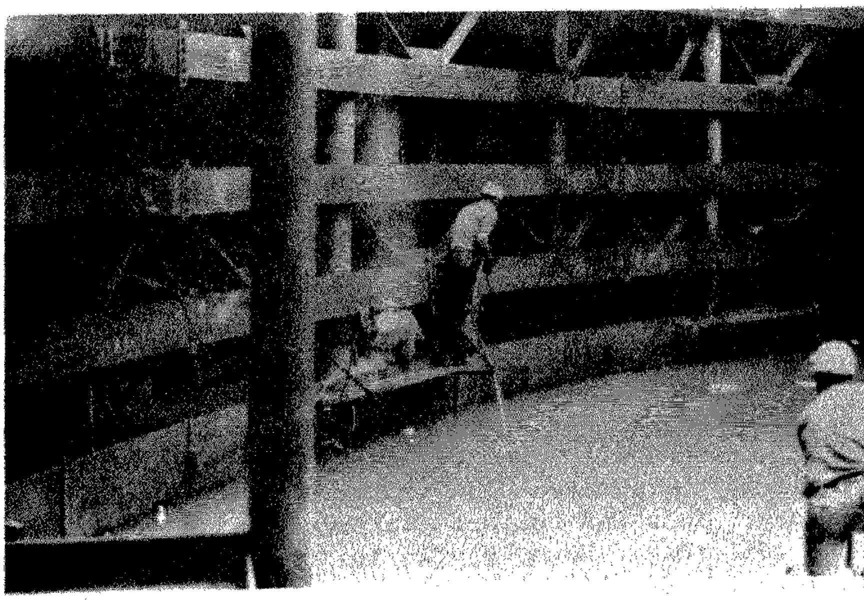


Fig. 8

Figure 9 is an overall view of the completed structure. I believed in view of the field experience that most of the bracing is unnecessary.

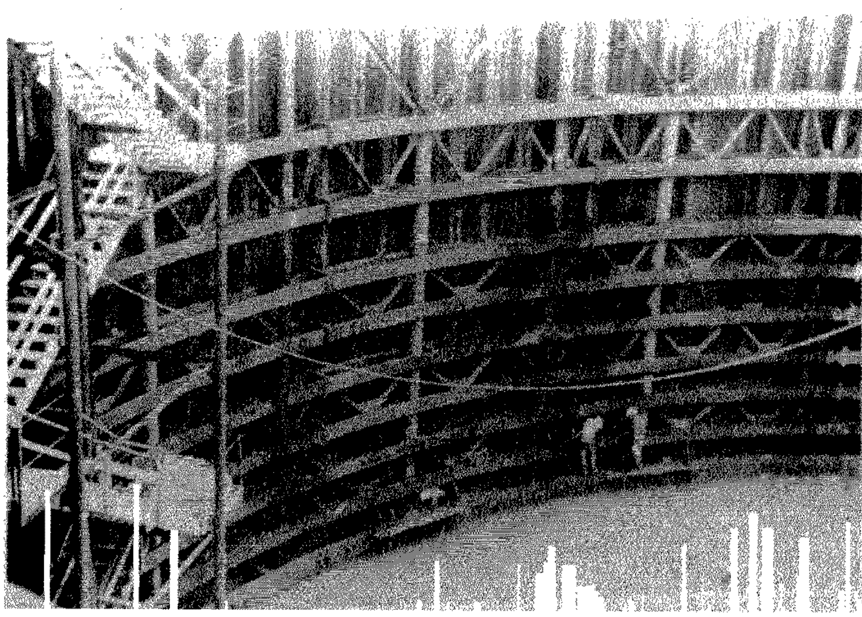


Fig. 9

PANEL DISCUSSION (Rothman)

The cofferdam behavior was checked with 26 strain gages and 24 pizzometers. Because we felt that buckling stability was no longer in question, the bulk of the gages was placed on the bottom wale where the effects of lateral water and earth pressure were least known. All gages were placed on the center line of the webs so that they measured thrust but not moment.

The gages generally confirmed the design load redistribution based on the relative stiffness of rings and sheet piling.

They also showed transverse stresses in the webs which were greater than what could be expected from the radial pressure. We believe they were due to non-parallelism of sheet piling and flanges. This caused the intervening blocking to bear against the edge of the flange, bending it and the web. These stresses act the same as residual stresses and lower the non-linear portion of the column stress-strain curve. This contingency is covered by the design procedure which did not take advantage of the expected low residual stress pattern.

The gages also showed a significant redistribution of loads between the bottom two wales from what was expected. The bottom was unloaded and the second loaded more heavily. The change was undoubtedly due to a substantial change in the assumed flow regime because of low permeability pockets in the rock. This was further confirmed by the existence of minor boils within the cofferdam. However, the load redistribution remained within the envelope of variable pressure and soil modulus used in the design.

The innovative solution using divers to assemble steelwork in the water was in retrospect efficient and economical. It would not have been feasible had it not been for the cooperation between many disciplines, including: Southern Construction & Engineering of Birmingham, AL, the knowledgeable contractors, who conceived of the method and then adopted their procedures to the technical requirements of the structure; Casagrande Engineers, who developed means for holding water pressures to manageable levels and for predicting the range of soil properties to be assumed; Weidlinger Associates, the structural engineers; and Professor Bernard Budiansky of Harvard, who reviewed the structural designs.

PANEL DISCUSSION

Questions and Answers

1\. Q. C. O'Connor
In connection with the slide showing the web of a large plate girder with longitudinal stiffeners, I was rather surprised by the dis-continuity of the spacing.

A. H. Nölke
The member shown was calculated before the specification was written. The first stiffener on the compression side must be continuous.

2\. Q. D. Hall
How does the specification apply to fatigue problems in stiffeners?

A. H. Nölke
The spec does not cover fatigue analysis for highway bridges - only for railroad bridges.

3\. Q. P. Dowling
The approaches for both the German and British Specifications are different. You (H. Nölke) stated that the German code takes into account imperfections. Do you allow for redistribution of stresses due to shear lag?

A. H. Nölke
Redistribution of stresses is not taken into account.

Q. If they are not, it would seem that the solutions would be uneconomical.

A. All elements are checked separately, and when all results are safe, the construction will be safe.

Q. Isn't this a very conservative approach?

A. Test results indicate that the approach is realistic.

Q. Do the tests with shear lag and without shear lag give the same results?

A. No answer.

4\. Q. G. Fox
Does the Engineer have to do anything if it is not necessary to calculate fatigue?

A. H. Nölke
The Engineer is not obliged to make calculations, but details should be designed to eliminate stress concentration.

PANEL DISCUSSION

5. Q. R. Graham
How was cofferdam started?

A. H. Rothman
The first wale was placed on the lake bottom, guide piles were driven, and the other wales were slid down along the piles.

Q. What was the degree of accuracy?

A. It originally looked bad when one viewed the out-of-plumb piles. However, the variations turned out to be from 1/2" to 3/4", which was really excellent. The units were assembled on land and placed into the water, and, as a result, the usual problems associated with true alignment that result from transportation, shocks, etc., were precluded.

6. Q. C. Miller
Why not use the ring formula for design rather than analyze it as a straight column?

A. H. Rothman
Actually a modified ring formula was used. But in direct answer to your question, we used the equivalent straight column to take advantage of all the technology on straight columns and also to take into account initial crookedness.

Q. There is also a lot of technology on the ring formula.

A. The results are the same - using 90% of the diameter.

7. Q. L. Beedle
You (H. Rothman) noted that the bottom wales were all welded. What were the assumed residual stresses for top and bottom?

A. H. Rothman
50% yield for all wales - top and bottom.

Q. J. Durkee
The Merrison rules never had an official status. What is the status now? Is the new code to supersede the rules? Will they be phased out?

A. B. Wex
The Ministry rules were established by an Advisory Committee to the Government. They made recommendations and the Government instituted them. The Advisory Committee has disbanded. For box girders, an interim code is used in the U.K. The new BS 5400 code will supersede the interim work rules.

For concrete bridges, the Ministry rules will continue to be used.

PANEL DISCUSSION

Q. Do all competitive designs - Design/Build Competitions - conform to one rule?

A. This is very confusing. If one knows all the rules intimately, it is O.K. - otherwise it is a big problem.

9. Q. H. Globig
You (B. Wex) noted that the BS 5400 is not applicable for concrete bridges.

A. B. Wex
Ministry rules are presently in use.

Q. We cannot get copies of the Ministry rules anywhere - in the U.K. or the U.S.

A. Yes. They are rare. I'm not defending the situation.

10. Q. C. H. Yoo
How do you (H. Rothman) select the size of bracing members?

A. H. Rothman
We started out with minimal based on 2% of compressive forces and added stiffeners.

11. Q. P. Grundy
The analogy between a strut (column) and a ring can go only so far. Shells have poor characteristics in buckling behavior.

A. H. Rothman
It is not a shell. It has interlocks with friction forces.

Q. Even a ring has post buckling characteristics that are different than a column.

A. Did not take into account post buckling.

12. Q. J. Melcher
You (H. Nölke) do not take into account tension. What about inter-actional stress components?

A. H. Nölke
Problem can be solved with all stress components acting simultaneously. The plate buckling curve is valid for stress components acting simultaneously.

Q. How do you take into account variable cross sections of box beams?

A. Lots of tests were made with different lengths of boxes and

PANEL DISCUSSION

different thicknesses of plates. All the results were the same. The buckling mode is the same, i.e., there were not diagonal tension fields in the web.

13\. Q. R. Wolchuk
I'm gratified to hear your (B. Wex) complimentary remarks regarding our Spec. (U.S.). Much of it was based on the results of British research.

I would like to see the U.S. Spec. used as the international standard.

A. B. Wex
Yes! It's understandable. However, I feel that the British Standards should be used! (Laughter)

Q. The new U. S. Code is actually not a Spec. It is published by FHWA and is presently available. It has already been used for the Seattle Bridge and others.

On one of the slides you presented, there seems to be a big difference in values. How was it analyzed? What was the number of stiffeners alone used for the analysis? In an analysis using the U.S. Spec., the number of stiffeners plus the entire flange area are taken into account.

A. The comparison was: British 4,000 Tons; U.S. 5,500 Tons for the multi-stiffened box. The U.S. practice uses more flange participation.

Q. Dr. Little did not use the Perry Robinson formulas. I feel that U.S. Spec. is less conservative than the British code but that it still is too conservative in the assumptions.

A. No answer.

13a.Q. P. Dowling
A more effective plate width results in larger load capabilities. The top curve from the scatter band of parametric studies should be used.

A. R. Wolchuk
Actually, we used the areas closer to the bottom curves. We could have developed all sorts of curves for different shapes, but we didn't do it. We just tried to select something less conservative than the British.

B. Wex - With good results.

PANEL DISCUSSION

G. Fox - The U.S. Spec. is still not adopted but rather proposed. It will undergo a lot of revision and conservatism is bound to creep in as reviews continue.
No doubt BS 5400 will also undergo this process.

14. Q. D. Hall
What were the ranges of "d" over "t" for the boxes shown?

A. H. Nölke
Between 82 and 400.

15. Q. B. Wex
Were the thicknesses of the wall the same?

A. H. Nölke
Yes.

Q. In cases of high shear, high loading and/or high torsion, wouldn't tests indicate tension fields developing in the webs? I think we could forget shear in flanges.

A. Tests were rather academic. A diagonal tension field cannot be applied to box girders.

Q. It depends on whether the flanges remain stable and where the diagonal tension field occurs.

A. Flanges must be very stiff. The distance between the diaphragms must be small.

16. Q. B. Johnston
You must have transverse stiffeners to develop a diagonal tension field. Boxes shown on slides did not have these stiffeners.

A. H. Nölke
Yes!

17. Q. C. Marsh
Please clarify the distinction between shear lag and post tension buckling behavior with relation to the BS 5400 Code. You (B. Wex) spoke of effective area and then effective area due to shear lag. Were they the same - due to shear lag - due to buckling?

A. B. Wex
A two step calculation is required. A plate with shear lag effects has a greater effective width.

1980 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 1980 Annual Business Meeting was held on April 30, 1980, in conjunction with the Annual Technical Session at the New York Sheraton Hotel, New York City.

The minutes of the 1980 Annual Business Meeting follow:

CALL TO ORDER

The meeting was called to order at 11:45 a.m. by the Chairman, Jerome S. B. Iffland. Approximately 50 persons were present.

The Chairman introduced the Vice Chairman, Jackson L. Durkee, the Director, Lynn S. Beedle, the Technical Secretary, M. Nuray Aydinoglu, and the Administrative Secretary, Lesleigh G. Federinic.

The Chairman thanked the Federal Highway Administration and Urban Mass Transit Administration for supporting the conference and Lehigh Structural Steel Co. for cosponsoring the social hour.

ELECTION OF EXECUTIVE COMMITTEE MEMBERS

The Nominating Committee, chaired by W. W. Yu, renominated incumbants S. J. Errera, R. M. Meith and J. Springfield for three year terms on the Executive Committee.

Voting for all nominees was conducted by letter ballot to the membership. All three nominees were elected effective immediately.

MEMBERS AT LARGE

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

J. R. Maison, Southwest Research Institute
R. Zandonini, Politecnico di Milano

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

LIFE MEMBERSHIP

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

John W. Clark

The motion that Clark become a Life Member was carried unanimously.

BYLAWS

Voting on proposed Bylaw changes was conducted by letter ballot to the membership. The proposed changes were approved. All suggestions will be reviewed by the Committee and a recommendation made to the Executive Committee. No suggestion will be ignored.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director including the proposed budget for fiscal year 1980-81.

Budget 1980-81:

Expected balance, 1 Oct 1980	$19,700.00
Income	35,000.00
Expenditures	43,800.00
Expected balance, 30 Sep 1981	$10,900.00

The budget was approved.

DIRECTOR'S REPORT

The Director summarized the Executive Committee activies, task group highlights and announced that Bruce G. Johnston, a founding member of the Council, would undertake the task of writing the SSRC history.

NEXT ANNUAL TECHNICAL SESSION AND MEETING

The Chairman announced that the next Annual Technical Session and Meeting will be held at the Conrad Hilton Hotel in Chicago. The dates will be 5-8 April 1981. The title of the Panel Discussion will be "Stability of Tall Buildings".

ADJOURNMENT

The meeting was adjourned at 12:05 p.m.

1980 ANNUAL TECHNICAL SESSION & MEETING ATTENDANCE

Participant	Affiliation
Abrahams, M. J.	Parsons, Brinckerhoff, Quade, & Douglas, New York
Austin, W. J.	Rice University, Houston, Texas
Aydinoglu, M. N.	Lehigh University, Bethlehem, Pennsylvania
Beedle, L. S.	Lehigh University, Bethlehem, Pennsylvania
Bernstein, M. D.	Foster Wheeler Energy Corp., Livingston, New Jersey
Birkemoe, P. C.	University of Toronto, Ontario, Canada
Birnstiel, C.	Consulting Engineer, New York
Biswas, M.	Texas A & M University, College Station, Texas
Bjorhovde, R.	University of Alberta, Edmonton, Alberta, Canada
Capanoglu, C.	Earl & Wright Consulting Engineers, San Francisco, CA
Cary, R. L.	Armco, Inc., Middletown, Ohio
Chajes, A.	University of Massachusetts, Amherst, Massachusetts
Chapuis, J.	Washington University (Student), St. Louis, Missouri
Chen, A. C. T.	Exxon Production Research Company, Houston, Texas
Chen, J. H.	Ministry of Metallurgical Industry, Peking, China
Chen, S. F.	Xian Institute of Metallurgy and Construction Engineering, Xian, Shaanxi, China
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
Cluley, N.	ITT Grinnell Corp., Providence, Rhode Island
Cooper, P. B.	Kansas State University, Manhattan, Kansas
Dat, D. T.	Exxon Production Research Co., Houston, Texas
Downs, T. J.	Bakke, Kopp, Ballou, & McFarlin, St. Louis, Missouri
Durkee, J. L.	Consulting Structural Engineer, Bethlehem, Pennsylvania
Edwards, W. E.	Bethlehem, Steel Corp., Bethlehem, Pennsylvania
Elnaschie, M. S.	University of Ridyadh, Saudi Arabia
Errera, S. J.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Fallon, B. W.	Ohio State University, Worthington, Ohio
Federinic, L. G.	Lehigh University, Bethlehem, Pennsylvania
Finzi, L.	Politechnico of Milan, Milan, Italy
Fleischer, W. H.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Fox, G. F.	Howard, Needles, Tammen & Bergendoff, New York
Galambos, T. V.	Washington University, St. Louis, Missouri
Gaus, M. P.	National Science Foundation, Washington, D. C.
Graham, R. R.	U. S. Steel Corp., Pittsburgh, Pennsylvania
Grubb, M. A.	U. S. Steel Research Laboratory, Monroeville, Pennsylvania
Grundy, P.	Monash University, Clayton, Victoria, Australia
Gunzelman, S. X.	Brown & Root, Houston, Texas

Haaijer, G.	U. S. Steel Corp. Pittsburgh, Pennsylvania
Hall, D. H.	Bethlehem Steel Corp., Bethlehem, Pennsylvania
Harrison, H. B.	University of Sydney, NSW, Australia
He, B.	Cornell University, Ithaca, New York
Higgins, T. R.	Consultant, Epsom, New Hampshire
Hoizer, S. M.	Virginia Polytechnic Institute and State Univ., Blacksburg, VA
Iffland, J. S. B.	Iffland Kavanagh Waterbury, New York
Jayachandran, P.	Worcester Polytechnic Institute, Worcester, Massachusetts
Johnston, B. G.	Consultant, Tuscon, Arizona
Ketter, R. L.	State Univ. of New York at Buffalo, Buffalo, NY
Kinra, R. K.	Shell Oil Company, Houston, Texas
Klippstein, K. H.	U. S. Steel Research, Monroeville, Pennsylvania
Krajcinovic, D.	Univ. of Illinois at Chicago, Chicago, Illinois
Lee, G. C.	State Univ. of New York at Buffalo, Buffalo, NY
Loomis, R. S.	Loomis and Loomis, Inc. Windsor, CT
Loomis, R. W.	Loomis and Loomis, Inc. Windsor, CT
Lu, L. W.	Lehigh University, Bethlehem, Pennsylvania
Malhotra, S. K.	Nova Scotia Technical College, Halifax, N. S.
Mangelsdorf, C.	Concordia University, Montreal, Canada
Masoumy, G.	Washington University (Student), St. Louis, Missouri
McDermott, R. J.	Lehigh University (Student), Bethlehem, Pennsylvania
Melcher, J. J.	Exchange research visitor, Lehigh U., Bethlehem, PA; Technical U. of Brno, Brno, Czechoslovakia
Miller, C. D.	Chicago Bridge and Iron Co., Plainfield, Illinois
Morgenstern, B.	Buckland & Taylor Ltd., North Vancouver, B. C.,Canada
Noelke, H.	Hannover University, Hannover, Federal Republic of Germany
O'Connor, C.	University of Queensland, Brisbane, Australia
Ojalvo, M.	Ohio State University, Columbus, Ohio
Ostapenko, A.	Lehigh University, Bethlehem,Pennsylvania
Palmer, F. J.	Copperweld Tubing Group, Pittsburgh, Pennsylvania
Paulis, G.	Dominion Bridge Co., Ottawa, Ontario, Canada
Pekoz, T.	Cornell University, Ithaca, New York
Pillai, S. U.	Royal Military College of Canada, Kingston, Ontario, Canada
Puglia, L.	Iffland Kavanagh Waterbury, New York
Razzaq, Z.	University of Notre Dame, Notre Dame, Indiana
Ross, D. A.	University of Akron, Akron, Ohio
Rothman, H. B.	Weidlinger Associates, New York
Sakimoto, T.	Ohio State University, Columbus, Ohio
Sherman, D. R.	University of Wisconsin, Milwaukee, Wisconsin
Silva, M. A.	University of Illinois at Chicago Circle, Chicago, Illinois
Spencer, H. H.	Rutgers University, Piscataway, New Jersey
Springfield, J.	Carruthers & Wallace, Rexdale, Ontario, Canada
Stea, W. J.	Ebasco Services Corporation, New York, New York

Temple, M. C.	University of Windsor, Ontario, Canada
Varney, R. F.	Federal Highway Administration, Washington, D. C.
Waldner, H. G.	Modjeski & Masters, Harrisburg, Pennsylvania
Wang, C. K.	University of Wisconsin, Madison, Wisconsin
Wex, B. P.	Freeman Fox & Partners, London, England
Winter, G.	Cornell University, Ithaca, New York
Wolchuk, R.	Wolchuk, Mayrbaurl, and Lally, New York
Yen, B. T.	Lehigh University, Bethlehem, Pennsylvania
Yoo, C. H.	Marquette University, Milwaukee, Wisconsin
Yu, W. W.	University of Missouri-Rolla, Rolla, Missouri
Zandonini, R.	Politecnico of Milan, Milan, Italy
Zellin, M. A.	Sverdrup & Parcel & Associates, St. Louis, Missouri

SSRC Chronology

18-19 Oct 80	- Executive Committee Meeting, Lehigh University, Bethlehem, Pa.
1 Jan 80	- Dr. M. Nuray Aydinoglu assumed duties of SSRC Technical Secretary
11 Jan 80	- Chairman's Meeting, Lehigh University, Bethlehem, Pa.
13 Mar 80	- Finance Committee Meeting, New York City
27-30 Apr 80	- Annual Technical Session & Meeting; Executive Committee Meetings; Task Group Meetings; New York City
30 Apr 80	- New Task Reporter established. TR-18 - Application of Finite Element Methods to Stability Problems (R. H. Gallagher)
23 Jun 80	- SSRC Guide Committee Meeting, Pittsburgh, Pa.

List of Publications

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

Birnstiel C. and Iffland, J. S. B.
FACTORS INFLUENCING FRAME STABILITY, Journal of the Structural Division, ASCE, Vol. 106, No. ST-2, Proc. Paper 15196, February, 1980

Chen, S. F.
LATERAL-TORSIONAL BUCKLING OF T-SECTION STEEL BEAM-COLUMNS, SSRC Annual Technical Session, New York, 1980

Chen, S. F.
LATERAL-TORSIONAL BUCKLING OF PARTIALLY YIELDED BEAM-COLUMNS, 1980

Committee 16, Tall Building Council
STABILITY, CHAPTER SB-4, STRUCTURAL DESIGN OF TALL STEEL BUILDINGS, Monograph on the Design and Planning of Tall Buildings, 1980

Dux, P. F. and Kitipornchai, S.
BUCKLING APPROXIMATIONS FOR LATERALLY CONTINUOUS ELASTIC I-BEAMS, University of Queensland, Dept. of Civil Engineering Research Report Series, Research Report No. CE 11, April, 1980

Haaijer, G., Carskaddan, P. S., and Grubb, M. A.
PLASTIC DESIGN WITH NONCOMPACT SECTIONS INCLUDING COMPOSITE BRIDGE MEMBERS, SSRC Annual Technical Session, New York, 1980

Hall, D. H.
PROPOSED STEEL COLUMN STRENGTH CRITERION, 1980

Hoglund, T.
DESIGN OF TRAPEZOIDAL SHEETING PROVIDED WITH STIFFNESS IN THE FLANGES AND WEBS, Swedish Council for Building Research, Stockholm, Sweden, D28:1980

Holzer, S. M. and Plaut, R. H.
DYNAMIC STABILITY OF SHALLOW ELASTIC ARCHES AND SHELLS, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, April, 1980

Jones, S. W., Kirby, P. A., and Nethercot, D. A
THE ANALYSIS OF FRAMES WITH SEMI-RIGID CONNECTIONS - A-STATE-OF-THE-ART REPORT, University of Sheffield (Dept. of Civil and Structural Engineering), January, 1979

Loomis, R. S., Loomis, R. H., Loomis. R. W., and Loomis, R. W.
TORSIONAL BUCKLING STUDY OF HARTFORD COLISEUM, Journal of the Structural Division, ASCE, Vol. 106, No. ST-1. Proc. Paper 15124, January, 1980

Maquoi, R. and Rondal, J.
COMPARISON OF RECENT PROPOSALS FOR THE ANALYTICAL REPRESENTATION OF STEEL BUCKLING CURVES, Dept. of Applied Sciences, University of Liege, Report No. 94, December, 1979

Melcher, J. J.
CLASSIFICATION OF STRUCTURAL STEEL MEMBERS INITIAL IMPERFECTIONS, SSRC Annual Technical Session, New York, April, 1980

Pillai, S. U.
COMPARISON OF TEST RESULTS WITH DESIGN EQUATIONS FOR BIAXIALLY LOADED STEEL BEAM-COLUMNS, Civil Engineering Research Report No. 80-2, Royal Military College of Canada, Kingston, Ontario, August, 1980

Structural Stability Research Council, Task Group 20
A SPECIFICATION FOR THE DESIGN OF STEEL-CONCRETE COMPOSITE COLUMNS, Engineering Journal, AISC, Vol. 16, No. 4, Fourth Quarter, 1979

Wang, C. K. and Talaboc, C. P.
A RESEARCH REPORT ON SECOND-ORDER ELASTIC-PLASTIC ANALYSIS OF STEEL FRAMES, SSRC Annual Technical Session, April, 1980

Yamamoto, K., Okumura, T., and Akiyama, N.
STUDY ON LOAD CARRYING-MECHANISM OF GUSSETED JOINTS, 1980

Finance

	Fiscal Year 10/79-9/80 Budget (approved 4/25/79)	Fiscal Year 10/79-9/80 Cash Statement 10/1/79-9/30/80	Fiscal Year 10/80-9/81 Budget (approved 4/30
BALANCE at Beginning of Period	$18,130.00	$27,547.32 (a)	$19,700.00
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
MBMA	---	1,000.00	1,000.00
FHWA	---	--- (b)	---
UMTA	---	6,000.00 (b)	---
NSF	10,000.00 (b)	---	14,000.00 (c)
Participating Organizations	1,500.00	1,900.00 (d)	2,000.00
Participating Firms	1,800.00	3,100.00 (e)	2,500.00
Total Contributions	$23,300.00	$22,000.00	$29,500.00
Registration Fees	2,000.00	4,316.00 (f)	3,000.00
MAL Subscription Fees	50.00	25.50	1,800.00
Guide Royalties	800.00	1,128.14	500.00
Sale of Publications	---	63.30	---
Interest	200.00	255.46	200.00
TOTAL INCOME	$26,350.00	$27,788.40	$35,000.00
EXPENDITURES			
Technical Services (Headquarters)			
Staff Salaries	15,000.00	18,191.81 (g)	18,000.00
Supply, phone, mailing	1,400.00	1,377.86	1,600.00
Travel	500.00	352.65	700.00
Total Technical Services	$16,900.00	$19,922.32	$20,300.00
Research Support	5,000.00	5,000.00 (h)	5,000.00
Annual Meeting & Proceedings			
Annual Proceedings	1,200.00	2,109.00	2,200.00
Expenses & Services	5,000.00	5,850.36	7,000.00
Travel	3,500.00	4,342.56	5,000.00
Total Annual Mtg & Proceedings	$ 9,700.00	$12,301.92	$14,200.00
SSRC Guide (4th Edition)			
Expenses & Services	---	9.39	{ 3,000.00
Travel	---	736.23	
Total SSRC Guide	---	$ 745.62	$ 3,000.00
United Engineering Trustees	100.00	100.00	100.00
Travel	1,000.00	1,156.98 (i)	1,000.00
Contingencies	200.00	---	200.00
TOTAL EXPENDITURES	$32,900.00	$39,226.84	$43,800.00
BALANCE at End of Period	$11,580.00	$16,108.88 (j)	$10,900.00

EXPLANATORY NOTES

Depositories (as of 10/1/79)	
General Account (UET)	$16,430.13
Technical Services (Lehigh University)	713.79
NSF Grant (Pittsburgh ATS&M)	4,403.40
FHWA Grant (New York City ATS&M)	6,000.00
	$27,547.32

1980 Annual Technical Session & Meeting support originally budgeted for NSF. FHWA and UMTA supported the conference. FHWA money received during FY 1978-79.

A proposal in the amount of $14,500 has been submitted to NSF in support of the 1981 Annual Technical Session & Meeting.

Aluminum Association ($500); American Petroleum Institute ($100); American Society of Mechanical Engineers ($100); U.S. Army Corps of Engineers ($100); European Convention for Constructional Steelwork ($100); Federal Highway Administration ($100); General Services Administration ($200); Institution of Engineers, Australia ($100); Langley Research Center, NASA ($100); Naval Ship Research & Development Center, U. S. Navy ($100); Steel Joist Institute ($200); Structural Engineers Association of California ($100); Canadian Society for Civil Engineering ($100)

$100 each - Amirikian Engineering Company; Ammann & Whitney; Beiswenger, Hoch & Associates; Blauvelt Engineering Co.; Brown & Root; Carruthers & Wallace Limited; Chevron U.S.A., Inc.; Earl and Wright; Feld, Kaminetzky & Cohen,P.C.; Gannett Fleming Corddry and Carpenter, Inc.; Green International, Inc.; Hardesty & Hanover; Hazelet & Erdal; Howard Needles Tammen & Bergendoff; Iffland Kavanagh Waterbury, P.C.; LeMessurier Associates/SCI; A. G. Lichtenstein & Associates, Inc.; Loomis and Loomis, Inc.; Mobil Research & Development Corp.; Modjeski and Masters; Parsons, Brinckerhoff, Quade and Douglas, Inc.; Rummel, Klepper & Kahl; Sargent & Lundy; Seely, Stevenson, Value & Knecht, Inc.; Skilling, Helle, Christiansen, Robertson, P.C.; Steinman, Boynton, Gronquist & Berdsall; Sverdrup & Parcel and Associates, Inc.; Tippetts-Abbett-McCarthy-Stratton; URS/Madigan-Praeger, Inc.; Vollmer Associates, Inc.; Weiskopf & Pickworth; Wiss, Janney, Elstner and Associates, Inc.; Richardson, Gordon and Associates.

Includes 57 paid luncheons and social hour donation of $200 from Lehigh Structural Steel Co.

		ANNUAL MEETING GRANTS		
Technical Services (Hqtrs)	SSRC FUNDS	NSF PITT	FHWA NYC	UMTA NYC
Director	$ 1,446.10			
Technical Secretary	4,490.85			
Administrative Secretary	1,728.40	$1,606.79	$2,250.00	$2,143.00
Secretarial/Clerical	2,412.19	871.60	685.88	557.00
(Includes employee benefits)	$10,077.54	$2,478.39	$2,935.88	$2,700.00

$1600 grant to University of Wisconsin-Milwaukee (Vinnakota)
$1500 grant to University of Notre Dame (Razzaq)
$1900 grant to University of Illinois at Chicago Circle (Krajcinovic)

EXPLANATORY NOTES - cont'd

(i) Executive Committee Meeting, Bethlehem, Pa., Oct. 1979; SSRC History, Bethlehem, Pa., May 1980; ASTM Meeting, Chicago, IL, Jun 1980

(j) Depositories (as of 9/30/80)

General Account (UET)	$12,196,26
Technical Services (Lehigh University)	961.97
4th Edition Guide Account (k)	2,934.08
NSF Grant (Pittsburgh ATS&M)	-0-
FHWA Grant (New York City ATS&M)	16.57
UMTA Grant (New York City ATS&M)	-0-
	$16,108.88

(k) New account (continuing) as required by NSF.

Register

Chairman: J. B. S. Iffland
Vice Chairman: J. L. Durkee

EXECUTIVE COMMITTEE

J. S. B. Iffland (81)
J. L. Durkee (81)
L. S. Beedle (Director)
W. J. Austin (82)
S. J. Errera (83)
G. F. Fox (81)
T. V. Galambos (82)
R. R. Graham (81)
T. R. Higgins (Technical Consultant)
B. G. Johnston (81)
R. M. Meith (83)
W. A. Milek, Jr. (82)
J. Springfield (83)
G. Winter*

* Past Chairman

STANDING & AD HOC COMMITTEES

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
R. E. Beil

C. Ad Hoc Committee on Research Priorities

J. S. B. Iffland, Chairman
R. Bjorhovde, Editor
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: S. Kitipornchai, Technical
M. N. Aydinoglu, Technical
L. G. Federinic, Administrative

TASK GROUPS

Task Group 1 - Centrally Loaded Columns

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

Scope: To define the strength of centrally loaded columns, taking due account of the influence of the column geometry, the column cross-sectional geometric properties, the mechanical properties of the column material, and the variables associated with manufacture of column components and with column fabrication.

Task Group 3 - Columns With Biaxial Bending

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

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

Task Group 4 - Frame Stability and Columns as Frame Members

J. S. B. Iffland, Chairman*
P. F. Adams
C. Birnstiel
M. Biswas
F. Y. Cheng
H. de Clercq
J. H. Daniels
P. Grundy
T. R. Higgins
I. M. Hooper
T. Kanchanalai
L. W. Lu
W. A. Milek
Z. Razzaq
C. K. Wang
J. A. Yura
M. A. Zellin

Scope: To develop procedures for investigating the stability of structural frameworks and the stability of columns as frame members.

Task Group 6 - Test Methods for Compression Members

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

* Executive Committee Contact Member

Task Group 13 - Thin-Walled Metal Construction

W. W. Yu. Chairman
S. J. Errera
A. L. Johnson
C. Marsh
T. M. Murray
A. Ostapenko
T. Pekoz
W. P. Vann
S. T. Wang
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

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

Scope: To investigate the behavior of horizontally curved girders, taking due account of the effects of rolling and fabrication practices; and to develop criteria for adequate bracing for such girders.

Task Group 15 - Laterally Unsupported Beams

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

Scope: To study the behavior of and develop stability criteria for laterally unsupported beams, including those in framed structures; and to determine bracing requirements for such beams.

Task Group 16 - Plate Girders

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

Scope: To develop practical procedures for determining the ultimate strength of stiffened plate girders, and to extend these procedures to include plate girders with multiple longitudinal stiffeners.

* Executive Committee Contact Member

Task Group 7 - Tapered Members

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

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

Task Group 11 - International Cooperation on Stability Studies

D. Sfintesco, Chairman
W. A. Milek, V. Chairman*
G. A. Alpsten
L. S. Beedle
A. Carpena
J. H. Chen
M. Crainicescu
T. V. Galambos
M. P. Gaus
O. Halasz
J. S. B. Iffland
B. Kato
P. Marek
J. J. Melcher
G. W. Schulz
J. Strating
L. Tall
R. Zandonini

Scope: To provide liaison between national and regional research groups and to organize international colloquia in the field of stability problems. In particular, to provide liaison between SSRC task groups, the Japanese Column Research Committee, Committee 8 of the European Convention for Constructional Steelwork, and similar groups in other countries. To suggest joint research projects.

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

R. B. Testa, Chairman
G. A. Alpsten
G. F. Fox*
A. Gjelsbik
A. L. Johnson
B. G. Johnston
L. W. Lu
E. P. Popov
F. D. Sears

Scope: To obtain and interpret data on the mechanical properties in steel in the inelastic range that are of particular importance to stability problems, including the determination of the average value and variation of the following: yield stress level, yield strength, tangent modulus, secant modulus, strain-hardening modulus, and magnitude of strain at incipient strain hardening.

* Executive Committee Contact Member

ask Group 17 - Stability of Shell-Like Structures

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

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

ask Group 18 - Unstiffened Tubular Members

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

Scope: To develop stability criteria for manufactured and fabricated unstiffened cylindrical tubular members, and to study the behavior of unstiffened non-cylindrical tubular members.

Task Group 20 - Composite Members

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

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

ask Group 21 - Box Girders

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

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

* Executive Committee Contact Member

Task Group 22 - Stiffened Tubular Members

C. D. Miller, Chairman	R. C. DeHart	G. F. Lang, Jr.
C. Babcock	N. W. Edwards	R. M. Meith*
M. D. Bernstein	S. X. Gungelman	K. Minhaus
C. Capanoglu	E. H. Killam	R. Regl
A. C. T. Chen	R. K. Kinra	G. J. Simitses

Scope:, To investigate the stability of circular cylindrical and conical shells with longitudinal or circumferential stiffening alone or in combination. Stability criteria will be developed for local buckling and general instability type failures of cylinders and cones under axial load, external or internal pressure, beam type bending and torsion. Available test data will be compared with suggested stability criteria. Recommendations will be made for research where insufficient data is available

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

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

Scope: To study the effect of end restraint on individual, initially crooked columns for which residual stress patterns are generally known.

TASK REPORTERS

Task Reporter 11 - Stability of Aluminum Structural Members

M. L. Sharp, Aluminum Company of America

Task Reporter 13 - Local Inelastic Buckling

L. W. Lu, Lehigh University

Task Reporter 14 - Fire Effects on Structural Stability

K. H. Klippstein, U. S. Steel Corporation

Task Reporter 15 - Curved Compression Members

W. J. Austin, Rice University

Task Reporter 16 - Stiffened Plate Structures

A. Monsour, Monsour Engineering

Task Reporter 17 - Laterally Unsupported Restrained Beam-Columns

L. W. Lu, Lehigh University

Task Reporter 18 - Application of Finite Element Methods to Stability Problems

R. H. Gallagher, University of Arizona

SPONSORING ORGANIZATIONS

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A. Amirikian	1978
W. J. Austin	1976
L. S. Beedle	1976
J. W. Clark	1980
E. L. Erickson	1976
E. H. Gaylord	1976
J. A. Gilligan	1976
J. E. Goldberg	1976
T. R. Higgins	1976
N. J. Hoff	1976
S. C. Hollister	1976
M. Holt	1979
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*

PURPOSES OF THE COUNCIL

The general purposes of the Structural Stability Research Council shall be:

1. To maintain a forum where structural stability aspects of the behavior of frames, columns and other compression-type elements in metal and composite structures can be presented for evaluation, and pertinent structural research problems proposed for investigation.

2. To review the world's literature on structural stability of metal and composite structures and study the properties of metals available for their construction, and to make the results widely available to the engineering profession.

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

4. To promote publication and dissemination of research information in the field of structural stability.

5. To study the application of the results of research to stability design of metal and composite structures, and to develop comprehensive and consistent strength and performance criteria and encourage consideration thereof by specification-writing bodies.

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*Revised: August 21, 1947; October 1, 1948; November 1, 1949; August 15, 1951; May 20, 1955; October 1, 1960; May 7, 1962; May 21, 1965; May 31, 1968; March 27, 1974; May 7, 1975; November 15, 1976: and April 30, 1980.

3. A Director may be engaged by the Executive Committee, subject to the approval of the Council, to serve as the chief paid executive officer of the Council. Additional paid officers may be engaged by the Executive Committee as necessary, subject to the approval of the Council. The Director may appoint one or more Secretaries, paid or otherwise, who need not be members of the Council. The salaries of all such officers and employees shall be determined by the Executive Committee. The Director shall be ex-officio a member of the Council and of the Executive Committee.

4. Working in concert with the Chairman, the Director shall conduct the regular business and official correspondence of the Council. He shall handle the financial affairs of the Council in accordance with an approved budget and shall keep complete records thereof. He shall scrutinize all Council expenditures and certify to the accuracy of all bills or vouchers on which money is to be paid, exerting every effort to secure economy in the business administration of the Council. He shall engage such Council employees as may be authorized and shall be responsible for their work. He shall prepare and execute all contracts authorized by the Executive Committee. He shall attend all meetings of the Council and the Executive Committee and, to the extent practicable, of the task groups and committees.

COUNCIL EXECUTIVE COMMITTEE

1. The Executive Committee shall consist of the Chairman of the Council, the Vice-Chairman, the Director, the most recent Past Chairman and Past Vice-Chairman, and nine additional members elected by the Council from its membership. For the nine elected members the term of membership shall be three years, with three members elected each year. Members whose terms are expiring shall be eligible for immediate re-election. Members shall take office immediately upon their election.

2. An unanticipated vacancy shall be filled by appointment by the Chairman from the membership of the Council, and the appointee shall serve for the remainder of the term.

3. The Executive Committee shall determine and implement policies and programs to support and advance the general purposes of the Council and shall exercise general direction and supervision over the technical and business affairs of the Council. The specific responsibilities and duties of the Executive Committee shall include the following:

(a) Review and approve proposed research projects and contracts.

(b) Correlate and give general supervision to research projects and contracts.

(c) Set up task groups and committees and appoint chairmen thereof, and approve nominees for membership therein.

(d) Appoint a Committee on Finance, a Committee on "Guide to Stability Design Criteria for Metal Structures" and such other committees as may be deemed necessary from time to time.

(e) Review, approve and disseminate reports and manuscripts.

(f) Sponsor and implement the preparation of successive editions of "Guide to Stability Design Criteria for Metal Structures" and appoint the Editor thereof.

(g) Respond appropriately to inquiries relating to stability design criteria. Such inquiries may be referred to the appropriate task groups for evaluation and response.

(h) Prepare the program for the Annual Technical Session and Meeting of the Council.

(i) Direct the financial and business management of the Council including preparation of the annual budget.

4. From time to time the Executive Committee may ask consultants particularly interested in specific projects to serve in an advisory capacity with respect thereto.

5. Meetings of the Executive Committee shall be held in the spring and in the fall. Additional meetings may be held at the call of the Chairman, or at the written request of two members of the Executive Committee or ten members of the Council. An Executive Committee quorum shall consist of seven members.

6. The minutes of the Executive Committee shall be transmitted promptly to all task group chairmen, and furnished on request to any member of the Council. If no objection is made by any member within a reasonable period after the minutes have been issued, it shall be considered that the Council has no objection to the recorded actions of the Executive Committee. However, if objection to any Executive Committee action is entered by three or more Council members, then the action in question shall be submitted to the Council for vote, either at a meeting called for that purpose or by letter ballot.

E L E C T I O N S

1. Each year at the fall meeting the Executive Committee shall appoint three members of the Council to serve as the Nominating Committee, with one of the three named as Chairman. Members of the Executive Committee or of the previous year's Nominating Committee shall not be eligible to serve.

2. The Nominating Committee shall prepare a slate of candidates for Chairman and Vice Chairman of the Council and for members of the Executive Committee to fill the anticipated vacancies, and shall transmit this slate to the Chairman of the Council by January 15.

3. The election of the Chairman and Vice Chairman of the Council and of members of the Executive Committee shall be by letter ballot. The results of the balloting shall be reported at the regular Annual Meeting of the Council. To be elected Chairman or Vice Chairman a candidate must receive a majority of the votes cast. In the event no candidate for Chairman or Vice-Chairman receives such a majority, a run-off election between the two candidates receiving the largest number of votes shall be conducted.

STANDING AND SPECIAL COMMITTEES

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

2. The Committee on Finance shall solicit financial support for the work of the Council. The Chairman and the Vice Chairman of this committee shall be selected from the membership of the Executive Committee.

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

4. Chairmen and members of standing and special committees shall be appointed by and responsible to the Executive Committee, shall serve for three years, and shall be eligible for immediate reappointment.

TASK GROUPS

1. The Executive Committee may establish task groups, each for the study of a specific subject. Each task group shall consist of as small a number of members as feasible for the work at hand. Task-group members need not be members of the Council.

2. Task-group chairmen shall be appointed by and responsible to the Executive Committee, shall serve for three years and shall be eligible for immediate reappointment.

3. Prior to the Annual Meeting each task-group chairman for the ensuing year shall review the members of his task group with the objective of providing the most effective organization, and submit membership recommendations to the Executive Committee for approval.

4. The duties of a task group with respect to its designated area of responsibility shall include the following:

(a) Make recommendations as to needed research.

(b) Review proposed research projects and render opinions as to their feasibility and suitability as Council projects.

(c) Furnish advice and guidance in connection with research projects, and suggest improvements in details of research programs within budgetary limitations.

(d) Make recommendations as to termination of projects.

(e) Prepare summary reports covering results of ongoing research projects.

(f) Prepare state-of-the-art reports summarizing existing knowledge, procedures and practices.

(g) Prepare material for the "Guide," as requested by the "Guide" Committee or the "Guide" Editor.

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

6. Task groups shall be responsible to the Executive Committee for organizing and carrying out their projects, which shall be approved by the Executive Committee.

7. Each task group shall meet at least once in each fiscal year to review progress and plan activities for the ensuing year.

8. The chairman of each task group shall submit an annual report to the Executive Committee prior to the Annual Meeting, and he shall report to the Executive Committee at such other times as requested or as he deems necessary.

CONTRACTS AND AGREEMENTS

The Executive Committee may, within its budget, enter into contracts and agreements to implement the work of the Council. Contracts for research projects preferably should be for a fiscal-year period. At the end of such a period a contract may be renewed or extended by the Council for an additional period preferably not exceeding the next fiscal year. Employment agreements with the Director or other paid employees of the Council may be for extended periods.

FISCAL YEAR

The fiscal year shall begin on October 1.

REVISION OF BY-LAWS

These By-Laws may be revised by a majority vote of the entire membership of the Council conducted by letter ballot.

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. <u>Reporting of Task Group Activities</u>

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.