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Column Research Council

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COLUMN RESEARCH COUNCIL

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by the Engineering Foundation

Proceedings 1973

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

My tenure of office as chairman of the Column Research Council will expire on October 1, 1974, and the next Foreword in these Proceedings will be written by my successor. It has been my pleasure, and a great honor, too, to serve the Council and through it the structural design professionals in these four years.

I want to take this opportunity to thank the Task Group Chairmen and the Task Group members for the great amount of volunteer work they have put into the third edition of the CRC Guide. Without your input the job would have been impossible. In addition, I want to express my great appreciation to Bruce Johnston, who as editor of the Guide is putting together a most comprehensive and valuable book for use by structural designers.

If I were to think of the most important CRC effort during the last four years, I would place the writing of the Guide in first place. This was, however, not the only important activity. There were many research projects under the auspices of the Column Research Council, and these Proceedings bear the record of the reports on these projects as presented in May 1973 at Los Angeles. The scope of the Task Groups, as well as the make-up of the attendees of the Annual Technical Sessions, indicates that the Council is broadening its scope. This is all to the good.

I want to thank my colleagues on the Executive Committee, Lynn Beedle, the CRC Director, and Barbara Freeman, the CRC Secretary, for their labor, their dedication, and, last but not least, for their patience with me. All our activities would not be possible without the financial support received from our friends in industry and government. The record of their contributions is given in this report, and I express the gratitude of the Column Research Council in this Foreword.

I would like to end this Foreword with a challenge to all of us. We know already a great deal about the instability of columns, beams, beam-columns, frames, plates, shells, etc. Let us set as our goal the unification of this knowledge so that from the stem of one theoretical base the behavior of all structural elements can be explained in a connected and coherent manner. In this way we can shorten the gap between research and practice, and lessen the confusion and compartmentalization which now still dominates our area of engineering.

T. V. Galambos
Chairman, Column Research Council
St. Louis, Missouri
November 1973
The CRC Executive Committee

MEMBERS OF THE EXECUTIVE COMMITTEE

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...in a meeting

Annual Technical Session

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

The 1973 Annual Technical Session was held on May 2 and 3 at the Los Angeles Hilton Hotel in Los Angeles, California. Sixty persons attended the Session and thirty-three papers were delivered.

A panel discussion on "Frame Stability Under Seismic Loading" was held in the evening of May 2, 1973. Panelists were E. J. Teal, M. H. Mark and D. R. Strand. The moderator was W. A. Milek.

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

Abstracts 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.
TASK GROUP REPORTS

TASK GROUP 1, CENTRALLY LOADED COLUMNS

Chairman, J. A. Gilligan, United States Steel Corporation (L. Tall, Lehigh University, presiding)

Considerations and Consensus of TG-1 With Respect to Multiple Column Curves

W. A. Milek, American Institute of Steel Construction

The report with the above named title was presented by Mr. Milek.

A Comparison of the American and European Multiple Column Curves

R. Bjorhovde, American Institute of Steel Construction, Inc.

The introduction of the concept of multiple column curves forms an important step in the direction of improving the method of column strength assessment. Simultaneous investigations in the United States and Europe over the past few years have dealt with the development of such sets of column curves. This paper presents a comparison of the methods of analysis, the data used in the generation of individual column strength curves, and finally the resulting sets of multiple column curves. It is shown that although the bases for the investigations were quite different, the American and the European multiple column curves both provide rational means of improving the method of column strength determination.

This report was in fact incorporated by Dr. Tall into his own presentation (see following).
Design Criteria for Steel H-Columns under Biaxial Loading

W. F. Chen, Lehigh University

Several design criteria for steel H-columns subjected to compression and combined with biaxial bending are discussed. The load carrying capacity of the columns is presented in terms of interaction surfaces. Three classes of problems are considered: short columns, long columns under symmetrical loading, and long columns under unsymmetrical loading conditions. The interaction surfaces are given in forms of tables suitable for analysis and design. Simple approximate formulas are also developed as an alternative method for the designer. The results are compared with the current Column Research Council design formula.

Ultimate Strength of Columns Under Biaxially Eccentric Load

Edwin H. Gaylord, University of Illinois

An efficient procedure for determining the strength of eccentrically loaded columns was presented in this paper. For uniaxial bending the equation of equilibrium is

\[ y'' + f(P,Py) = 0 \]
Task Group Reports

The deflected shape for equal and eccentricities \( e \) (Fig. 1) is found by an initial-value solution of Eq. (1). With \( y(0) = y_0 \) and \( y'(0) = 0 \) (Fig. 1) the solution is obtained in the form

\[
y = y(P, y_0)
\]

(2)

It is shown that the condition for a maximum value of \( P \) is \( \partial e / \partial y_0 \). To satisfy this requirement Eq. (1) is differentiated with respect to \( y_0 \) to give

\[
\delta y'' + f'(P, y) \delta y = 0
\]

(3)

where \( \delta y = \partial y / \partial y_0 \) (Fig. 1). This equation must satisfy the initial values shown in the figure.

Equations (1) and (3) are integrated simultaneously proceeding along the \( z \) axis until \( \delta y = 0 \), which satisfies the condition for maximum \( P \) and determines the length of the column for the given values of \( P \) and \( y_0 \).

The procedure has been extended to the cases of uniaxial bending with unequal end eccentricities and the general case of unsymmetrical biaxial bending.

Hybrid Computer Solution of a Biaxially Loaded Beam-Column

T. Michael Basehart, James F. McDonough and Boyd C. Ringo, University of Cincinnati

Research is currently underway at the University of Cincinnati to develop a hybrid (analog/digital) computer model to describe the inelastic behavior of a biaxially loaded beam-column.
Since the behavior of biaxially loaded beam-columns can be described by three coupled differential equations, the problem is ideally suited for an analog computer. Basically, analog computers are designed to simulate systems represented by differential equations.

Previously published results of an analog solution used to determine the elastic response of a biaxially loaded beam-column did not compare favorably with established results, thereby leaving some doubt as to the reliability and accuracy of the analog computer. However, elastic response of a biaxially loaded beam-column using the University of Cincinnati hybrid facilities were in excellent agreement with established results.

As strains in the member exceed the yield strain, the stiffnesses of the cross-section at locations along the member change. Therefore, the stiffness coefficients in the three coupled differential equations are no longer constant, as in the elastic solution procedure. In order to constantly update the stiffness coefficients, the digital computer must be used in conjunction with the analog computer, hence, the meaning of the term hybrid solution procedure. These procedures are currently being verified by the use of numerical solutions.

As a final note, it is hoped that the current research project will illustrate that the hybrid computer is an effective tool that can be used by the structural researchers to solve problems which can be described by differential equations, but that have no adequate closed form solution.

Comparison of Interaction Curves for Long Columns
Task Group Reports

COLUMNS: Past, Present, and Future

L. Tall, Lehigh University

Columns have existed since the very beginning of time, from the logs of the stone-age man, to the huge jumbo shapes used in skyscrapers today. Perhaps the single most important factor influencing the strength of steel columns is the presence of residual stresses, the stresses formed due to plastic deformations during fabrication processes such as cooling after rolling or welding. The effect of residual stresses is always to lower the column strength from what it would be if no residual stresses were present. The effect can be negligible, or it can be of major importance, depending on the grade of steel, the fabrication method, and the type of structure of which the column is a part. Another important influence on column strength, and a more obvious one, is the influence of out-of-straightness of the column. Together, these two factors can decrease column strength as much as 40% from what could be attained in their absence.

Studies at Lehigh University over the past decade and a half, conducted under the general technical direction of the Column Research Council, have progressed from small rolled shapes, to welded shapes, and then to heavy shapes. The studies have been both theoretical and experimental. Residual stresses have been measured experimentally in representative samples of cross-sectional shapes and utilized in complex theoretical predictions of column strength taking all possible factors into account. Full-scale column tests were performed over the years on shape sizes up to and including the heaviest rolled jumbo shapes and their welded equivalents. Basically, the results of the studies showed a huge variation in strength among columns, and it indicated that the use of a single column strength curve should be reconsidered, since some columns obviously were stronger than others, whereas more strength was ascribed to some columns that didn't really have it.

This lead directly to the concept of multiple column curves -- and the decision in the research project was to divide the whole scatter of possible column curves into three basic curves, the middle one of which was the current CRC basic column curve. The column curves were based on the maximum strength rather than on the tangent modulus concept. The three curves represent distinctly different columns -- the top one, for instance, was meant for annealed columns, tubes, and columns made of high-strength steel, in all of which the influence of residual stress was at its minimum.

At this time, the CRC is considering these three basic column curves, and indeed, whether or not three curves should be recommended to be the basis for design. Much information is still to be obtained on the categories to which various column shapes and fabrication procedures belong. Finally, the thought should be remembered, that perhaps another approach to column strength studies is to find ways to improve the strength of the weaker columns so that all columns do, if fact, have the same strength allowing the use of only one design curve.
The ultimate capacity of the general case of beam-columns (which are capable of developing the full strength of the cross section without local buckling) is underestimated by the existing linear interaction equations. The comparisons shown in the figure illustrate this underestimate. This is true for both columns of zero length and of finite length.

It has been concluded that any improvement in equations sufficiently simple for design becomes shape dependent. (This conclusion has been reached in Britain also). The classification is suggested, based on the recommendations of the 3rd Edition of the CRC Guide.

1. Compact wide flange (H) sections.
2. Compact square & circular tubes.
3. All other shapes.

NOTE: The basic requirement for strength is the satisfaction of equilibrium at full plastification, for compact sections.

For the compact wide flange sections, the following interaction equation has been proposed by Chen:

\[
\frac{(M_x)}{(M_{ux})} + \frac{(My)^\alpha}{(M_{uy})} \leq 1
\]

This applies for strength and stability. The numerator terms are the equivalent single curvature moments; the denominator terms are the capacity of the column under axial load, with uniaxial moment about the respective axis only, determined readily from existing CRC equations. Expressions are given for the exponent \( \alpha \). Restrictions on the above interaction equation are suggested for sections prone to torsional buckling and situations in which repeated loading may cause progressive deformation.

For square tubular columns, the modifications to the interaction equations suggested by Pillai and Ellis are recommended. The principle here is one of using the resultant of the orthogonal eccentricities (or moments).

For all other columns, the present CRC interaction equations are recommended.
TASK GROUP 4, FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

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

Ultimate Strength of Frames Designed by the Allowable-Stress Method

O. S. Okten, S. Morino, J. H. Daniels and L. W. Lu, Lehigh University

A comprehensive study of the lateral load-carrying capacity of high-rise moment-resisting steel frames has been in progress at Lehigh University since 1970. The major objectives of this investigation are: (1) to determine the ultimate strength of unbraced frames designed by the allowable-stress method, (2) to examine the behavior of these frames at the ultimate load, and (3) to study the effect of frame instability on the strength and failure behavior. A total of seven frames, varying from 10-story, 3-bay to 40-story, 2-bay was chosen. They were then designed according to the allowable-stress provisions given in Part 1 of the current AISC Specification. Two types of structural analysis were then performed on these frames: the first-order, elastic-plastic analysis and the second-order, elastic-plastic analysis. The latter includes the effect of frame instability (or P-Δ moment).

Figure 1 shows the load vs. drift relationships of the 10-story, 3-bay frame for two loading conditions. One involves constant gravity load and gradually increasing lateral load. This is usually called non-proportional loading condition. The curves are shown as curves 1 and 2 for gravity load factors of 1.0 and 1.30, respectively. The load factor 1.30 is taken from Part 2 of the AISC Specification. The other loading condition assumes pro-
portionally increasing gravity and lateral loads and these results are shown as curve 3. With the gravity load maintained at the working value, the ultimate lateral load is 2.4 times the working lateral load assumed in the design. For the proportional loading condition, a load factor of 1.45 is obtained.

![Wind Load vs. Drift Relationships of a Frame](image)

From the results of all the frames that have been studied so far, the following general observations appear to be valid:

1. The amount of gravity load acting on the girders affects significantly the lateral load-carrying capacity of the frames. Also there is an appreciable difference between the load vs. drift relationships for the proportional loading condition and those for the non-proportional loading condition.

2. The lateral load factor of moment-resisting frames designed by the allowable-stress method falls in the range of 1.30 to 1.65. Therefore, it is possible for these frames to resist a wind pressure which is 30 to 65% above the design wind pressure during a heavy windstorm.

3. Moment-resisting frames usually fail by frame instability caused by the P-Δ moment. The P-Δ moment, in fact, becomes a dominant factor in affecting the behavior of the frames immediately before and after the attainment of the maximum load.

4. The load vs. drift relationships obtained from the first-order analysis differs appreciably from those obtained from the second-order analysis.
Task Group Reports

Ultimate Strength of Plane Multistory Steel Rigid Frames

S. Liapunov, New York University

This presentation was concerned with the results of a study of the ultimate strength of plane multistory steel rigid frames, designed for realistic office building loads and in accordance with current allowable stress design practices. The ultimate strength of these frames was determined through the use of an incremental load technique with which the theoretical load-displacement relationship of a frame was established. The ultimate strength is defined as that loading at which the theoretical displacements of the frame approach infinity. In this research, the effects of joint displacements, the spreading of yielding zones and the presence of residual cooling stresses were taken into account. Stress reversal, strain hardening and shear displacements were neglected and lateral-torsional buckling was assumed to be prevented.

In all examples considered, the frames were found to be stable and elastic at the full design gravity and wind loading. As the loading was increased beyond the design condition, yielding was found to occur in the frame, eventually resulting in the formation of plastic hinges. The formation of plastic hinges was observed to commence in the lower story girders and progress to upper level girders with further increases in the applied load. Only occasional plastic hinges in columns were noted. In all examples, frame instability occurred before the formation of a mechanism.

Analysis of the obtained information has shown that the two principal causes of frame instability are the P-delta effect and the formation of plastic hinges. For the frames considered, the increasing of girder sizes, beyond those required by stress considerations, is the most effective way to reduce drift (and thus the P-delta effect) and also, to retard the process of plastic hinge formation. Therefore, for these frames, if a higher ultimate strength were required, girder sizes should be increased in preference to column sizes. As an example of the effect of an increase in column sizes consider the load-displacement relationships of two 32-story, 3-bay frames (30'-0 bays, 12'-6 floor heights) in which the webs of all columns are in the planes of the frames. The only differences in the two frames were due to different design assumptions regarding in-plane K values for columns. In one case, in-plane K values were obtained from the Julian-Lawrence Alignment Chart (with sidesway), in the other, in-plane K values were taken as 1.0. Out-of-plane K values of 1.0 were assumed for both frames. It is evident that, for these frames, the use of the Alignment Chart, resulting in somewhat larger column sizes (244 tons versus 224 tons of steel per frame) but same girder sizes, did not lead to any appreciable increase in ultimate strength.
Approximate Analysis of Tall Buildings by Non-Prismatic Thin-Walled Beam Theory

George C. Lee, State University of New York

Methods for combined bending and torsional analysis of thin-walled beams are well defined if the beam cross section is such that the shear center axis is a continuous line parallel to the longitudinal fibers of the beam. If a beam consists of segments of different types of cross sections (segments of thin-walled open and closed sections) or if the beam is a channel shape tapering in depth only, subjected to combined bending and torsion, no satisfactory analysis procedure is available.

A modification of the standard finite element procedure for thin-walled member analysis is introduced to accommodate the types of situations described above. This is done by first selecting the nodes such that each element can be idealized as a prismatic element, even the centroids and shear centers of two adjacent elements do not coincide at the nodes. Then, a transformation of local coordinates for each element is made to selected locations such that prismatic beam theory will apply when all the "transformed element stiffnesses" are further transformed to the global system.
Task Group Reports

This "double-transformation" finite-element procedure is now applied to determine the stresses in columns of tall, slender buildings due to combined gravity and horizontal loads. Then the plan of the building is not rectangular or symmetrical, torsional analysis of the building due to horizontal load can be conveniently done by the proposed procedure, which deals with the entire building as a three-dimensional structure and considers all floor levels as nodes. In this analysis, deformations of the floor systems are neglected, and the shear walls, if any, are converted to thin plates with the same material properties as those of the columns such that a combined open and closed, multi-cellular cross section for each element can be assumed. Numerical solutions of the column stresses in several illustrative structures are given.

This research has been financially supported, in part, by the American Institute of Steel Construction, the Metal Building Manufacturers Association, and the Navy Facilities Engineering Command, to the State University of New York at Buffalo.

TASK GROUP 8, DYNAMIC INSTABILITY

Chairman, D. A. DaDeppo, University of Arizona

Transient Response of a Column Subjected to a Periodic Axial Force

Dusan Krajcinovic, Argonne National Laboratory

It appears that the mechanism of the stability loss known as the parametric resonance is not always well understood. We can justly ask ourselves what is the agency that suddenly generates transverse vibrations in a column experiencing purely longitudinal vibrations up to that point.

For a better insight into the phenomenon we examine the transient response of a column subjected to a periodic axial force. The solution of the governing Mathieu differential equation is given in terms of Mathieu (elliptic cylinder) functions that are both tabulated and easy to compute using rapidly converging series.

It is demonstrated that the final relation governing the transverse motion depends on the initial transverse displacement and velocity. It can be concluded that:

a) the transverse motion cannot be initiated if both initial crookedness and velocity in transverse direction are absent,

b) the transverse motion for an arbitrarily small initial imperfection of the column axis becomes unstable if the forcing frequency or intensity belongs to the unstable region on the Mathieu diagram.
Hence, since a real column is never perfectly straight the transverse motion associated with an axial periodic force always exists, but only for certain combinations of the force intensity and frequency the amplitude of the motion grows beyond acceptable limits.

Fig. 1. Resultant transverse motion of midspan cross section of parametrically excited column; stable case. \( f_0 \) - initial imperfection at midspan, \( r \) - mode number

Fig. 2. Resultant transverse motion of midspan cross section of parametrically excited column; unstable case

Dynamic Stability of Yielding Structures Subjected to Earthquakes

S. C. Goel and W. Y. L. Wang, University of Michigan

It is known that typical building structures when subjected to a strong earthquake are called upon to dissipate large amounts of energy through inelastic action of the structural members. Repeated oscillations of the yielding structure may also produce large deformations and drift from the original equilibrium position. The interaction of large lateral deflections and the gravity loads may sometimes cause collapse of the structure due to instability. It is important to know the ductilities and energy dissipation requirements of the structural members and connections caused in a structure by a severe ground motion. It is also desirable to know whether the structure would or would not encounter instability during the expected large amplitude oscillations.
An analytical study of the dynamic stability of single degree-of-freedom systems subjected to strong earthquakes is in progress. Elasto-plastic hysteresis behavior of conventional and slip types is assumed which are representative of moment-resistant and braced frames of steel, respectively. The objective is to determine the structural and ground motion characteristics which lead to structural instability.

Two accelerograms, the N-S component of the El Centro 1940 earthquake and a simplified accelerogram based on equivalence of amplitude and duration of ground impulses in the real accelerogram are used. The simplified accelerogram facilitates investigation in a closed form of the interaction between ground motion and structural response in the vicinity of collapse. It also gives a criteria to predict instability. The results obtained by using this criteria and the simplified accelerogram are in good agreement with the numerically computed response due to the real accelerogram (Figure 1). The results also indicate that some energy dissipation early in the response may have significant influence on the stability of a structure (Figure 2).
Nonplanar, Nonlinear Oscillations of a Beam

C. H. Ho, R. A. Scott and J. G. Eisley, University of Michigan

The work is concerned with possible out-of-plane motions of simply supported columns whose cross section has two perpendicular axes of symmetry (the x,y-axes in the sequel). The columns are excited by a sinusoidal external force which is confined to a plane (xz-plane). They are taken to be compact, that is, \( \frac{I_{xx}}{I_{yy}} \), the ratio of the moments of inertia about the symmetry axes is taken to be close to unity. With this restriction, it is assumed that torsional effects can be neglected in the equations of motion. Shear deformations, and rotatory and longitudinal inertia are also neglected. The equations do take into account however, the possibility of a uniform tensile or compressive preload (below the Euler buckling level).

A modular approach to the mathematical problem is adopted, that is, solutions are sought in the form of products of linear mode shapes (or their sums) with functions of time (to be subsequently determined). For nonlinear systems such as the one at hand, this is not an exact procedure, but gives accurate results when used with the approximate method of Calerkin, which is the scheme used here. In using the method, it is assumed that the external load is such that at most one mode in each plane is excited.

Under the above conditions, planar and nonplanar sinusoidal responses of the column have been obtained. For certain frequency-amplitude ranges small disturbances of such responses grow and the original steady state motion is radically altered. These frequency-amplitude ranges (instability zones) have been found for the modes examined.
Some typical results are presented in Figs. 1 and 2. Shown are dimensionless, first-mode, planar displacement amplitudes as a function of frequency ω of the external load, for several values of the dimensionless generalized force F₁, which arises from Galerkin's method and is defined by

\[ F₁ = \frac{2L}{AE} \int F_x(s) \chi_1(s) ds \]

The notation is as follows: L and A are the length and cross section of the column, respectively, E is the modulus of elasticity, s = z/L, z being the coordinate along the neutral axis of the column, Fₓ is the magnitude per unit length of the external force, and \( \chi_1 \) stands for the dimensionless first mode shape (or an approximation to it, in the sense of Galerkin's method). Omitting the time-dependence, the amplitude \( A₁ \) is defined by

\[ \frac{u}{L} = A₁ \chi_1(s) \]

where \( u \) is the displacement in the x-direction. Dimensionless frequencies are used in the plots, being made so by dividing by \( \omega₀ \), the fundamental bending frequency in the xz-plane. For both plots the preload is taken to be zero, and \( l_{xx}/l_{yy} \) has the value 0.5. The value of damping, which in this work is taken to be viscous, is different for the two figures. In Fig. 1 it is taken to zero, whereas in Fig. 2 its value is specified by

\[ k = \frac{cL}{\sqrt{\pi E \rho A}} = 0.02 \]

where \( \rho \) stands for density and \( c \) is a viscous damping coefficient.

Each response curve has two branches. For zero damping, the left and right branches are in phase, and \( \pi \) degrees out of phase, with the external load, respectively. The boundaries of the instability zones are given by the curves designated by letters. The region AEC is one in which perturbations in the plane of forcing grow. It illustrates the well known planar jump phenomena. A new and important feature to emerge from the work is the zone A'B'C'. In this region, perturbations, corresponding to first mode motions, perpendicular to the plane of forcing grow. Thus a two-mode, out-of-plane, steady-state motion is predicted. A two-mode stability analysis shows that this motion is stable, so that whirling does indeed occur. Fig. 1 shows that in the absence of damping, it occurs for all force levels but only for the in-phase response.

Other instability zones, corresponding to higher spatial modes and responses with frequencies other than that of the driver, exist. Some of these are shown in Fig. 2, but they will not be discussed in any detail here, since it is felt that they are relatively unimportant. Fig.
Task Group Reports

2 also shows that the main effect of damping is to raise the tips of the zones off the frequency axis. It is seen that for force level above a small threshold value, whirling motions exist even in the presence of damping.

The effects of preload were also investigated. The above general conclusions apply as before. The main effect found was that as the preload increases, the instability zones move to lower frequencies.

Fig. 1. Response and stability curves for planar beam motions for the damping parameter $k=0$.

Fig. 2. Response and stability curves for planar beam motions for the damping parameter $k=0.02$. 
Task Group Reports

TASK GROUP 9, CURVED COMPRESSION MEMBERS

Chairman, W. J. Austin, Rice University

Buckling of Shallow Arches

W. J. Austin, Rice University

The bending and buckling behavior of shallow arches was reviewed. The presentation followed closely the draft submitted by Task Group 9 to Dr. Bruce Johnston for incorporation into Chapter 16 of the forthcoming CRC Guide. After describing the significant aspects of the behavior of shallow arches, useful design concepts were pointed out. Approximate upper and lower limits of the rise were suggested to define the proportions of arches which must be considered as "shallow". Finally, some typical theoretical buckling formulas were presented and the reliability of the theoretical buckling loads, as evidenced by experiments, was discussed.

TASK GROUP 11, EUROPEAN COLUMN STUDIES

Chairman, D. Sfintesco, C. T. I. C. M.
Vice-Chairman, W. A. Milek, American Institute of Steel Construction

Summary of Colloquium on Column Strength, Paris, November 1972

W. A. Milek

The report with the above named title was presented by Mr. Milek.
Chairman, G. F. Fox, Howard, Needles, Tammen & Bergendoff  
(A. L. Johnson, American Iron and Steel Institute, presiding)

Status of Quest for Steel Properties in Inelastic Range

A. L. Johnson, AISI

CRC Task Group 12, under the chairmanship of G. F. Fox, met in Pittsburgh November 2, 1972 to discuss ways to meet its charge by the CRC Executive Committee. After reviewing the objectives and discussing the nature of the Task Group mission with potential sponsors of research, it was determined that better definition of the problem is needed.

It was agreed that more information on our current state of knowledge is needed. To define the scope and the potential of development of information, it has been suggested that the assistance of those that have developed data be solicited to keep the Task Group work load within reasonable bounds. In addition to asking research organizations and individuals for information, assistance will be sought from other groups, such as the Metal Properties Council, and the U.S. Department of Defense. It is proposed that a letter be sent to all members of CRC, its Task Groups, directors of structural and material research facilities, principle researchers and others known as likely to give informative responses. The questionnaire will include solicitation for a variety of information such as yield strength, proportional limit, stress strain curves, initial, tangent, and secant moduli, strain at yield and at strain hardening.

Responses to the letter will be analyzed and used to further define and isolate areas where information is needed, and to provide a basis for information either through a specific research project or through extensions of other projects.
Task Group Reports

TASK GROUP 13, THIN WALLED METAL CONSTRUCTION

Chairman, S. J. Errera, Bethlehem Steel Corporation

Design Criteria for Interaction of Local and Overall Buckling

J. DeWolf, T. Pekoz and G. Winter, Cornell University

A design approach is presented which accounts for the combined effects of local buckling, column buckling and non-uniform material properties in compression members. It is based on the tangent modulus concept and utilizes the effective width expression developed by Winter.

Results agree satisfactorily with tests on two types of cold-formed columns, one of them a box shape where local buckling occurs in elements supported along both longitudinal edges, and one an I-shape where local buckling occurs in the outstanding flanges. Analytical results also compare favorably with tests by other investigators.

The method requires iteration but is otherwise fairly straightforward. This is so because the tangent modulus concept has been used as the theoretical basis. If an attempt had been made to include initial crookedness explicitly, in addition to the three named interacting factors, the problem would have become all but untractable from a design viewpoint.

It is believed that this investigation illustrates forcefully the desirability of retaining the tangent modulus approach at least for the more complex situations in column design, rather than attempting to include the effect of sweep explicitly. In this case, possible sweep effects, as heretofore, would be approximately reflected in the safety or response factor.

Buckling Behavior of Perforated Unstiffened Compression Elements and Beam Webs

W. W. Yu, University of Missouri-Rolla and C. S. Davis, Ford Motor Company

In cold-formed steel design, it is well known that the presence of holes in webs and/or flanges of beams and columns may result in a reduction in strength of individual component elements and/or the overall strength of the member.
Task Group Reports

The local buckling and post-buckling strength of perforated stiffened compression elements have been studied and reported by the authors at the 1971 annual meeting of the Column Research Council. This paper will deal with the findings obtained from the analytical and experimental investigations on the buckling behavior of perforated unstiffened compression flanges of columns and beams conducted by the authors at the University of Missouri-Rolla. In addition, it is intended to discuss the buckling behavior and crippling strength of beam webs when holes are provided in webs for duct work, piping and for other purposes.

Preliminary design recommendation will also be included in this paper.

Wall Stud Design Criteria

A. Simaan, T. Pekoz, Cornell University

The load carrying capacity of cold-formed steel wall studs is increased very substantially by the bracing action of the wall board material making the use of such studs as load carrying structural members economically feasible. This is a beneficial effect of the wall board that is primarily used for enclosure purposes on one or both sides of the studs. Wall studs are usually of simple or modified I, Z or channel section with web perpendicular to the plane of the wall board. Wall board can be of a variety of materials such as fiber board, pulp board, plywood, gypsum, etc., and is connected to the studs most commonly by screws. The bracing action of the wall board is due to both its shear rigidity, restraining the displacement of the stud in the plane of the wall board, and the resistance it offers to the pure rotation of the stud at the connector.

In the course of this research analytical solutions, including these bracing effects, were obtained. The solutions include prediction of flexural and torsional-flexural buckling loads, as well as determining the effect of initial imperfections of the stud in limiting the adequacy of wall board material as a bracing medium. Solutions are for the cases of wall board on one or both sides of the studs.

The analytical solutions were checked by an experimental program designed to explore different modes of behavior. The correlation is quite satisfactory.

Finally, the solutions were put into a design oriented format, and computer programs to facilitate its use were prepared.

This research was sponsored at Cornell University by the American Iron and Steel Institute. It is expected that the results of this research will be reflected in future design specifications.
Task Group Reports

TASK GROUP 14, HORIZONTALLY CURVED GIRDERS

Acting Chairman, J. L. Durkee, Bethlehem Steel Corporation

Recent Research in Instability of Curved Plate Girders and Box Girders

C. G. Culver, National Bureau of Standards
(presented by B. T. Yen, Lehigh University)

Analytical and some experimental studies were made in the area of instability of component parts of horizontally curved plate girders and box girders. The projects were conducted at the Carnegie-Mellon University under the sponsorship of Pennsylvania Department of Transportation in conjunction with the Consortium of University Research Teams (CURT) Program of the Federal Highway Administration. Elastic and inelastic localized flange buckling, lateral buckling, web bending, web buckling and the behavior of transverse and longitudinal stiffeners in curved plate girders were examined. For example, transverse stiffeners rigidity requirements were developed as a function of the aspect ratio and radius of curvature of the curved web panel. For curved box girders, buckling of stiffened and unstiffened compression flanges were considered. Based on the results of these and other studies, design recommendations were developed for the proportioning of horizontally curved plate girders, curved composite box girders and curved hybrid girders.

TASK GROUP 16, PLATE AND BOX GIRDERS

Chairman, F. D. Sears, Federal Highway Administration
(B. T. Yen, Lehigh University, presiding)

Brief Summary of Research on Steel Box Girders Abroad

B. T. Yen, Lehigh University

The report with the above named title was presented by Dr. Yen.

Failure Tests of Composite Rectangular Box Girder Methods

B. T. Yen, Lehigh University

The report with the above named title was presented by Dr. Yen.
Task Group Reports

TASK GROUP 17, STABILITY OF SHELL-LIKE STRUCTURES

Chairman, K. P. Buchert, University of Missouri-Columbia

Applications of Shells and Shell-Like Structures


A survey of applications of shell and shell-like structures was given. Examples of the applications given included large domed stadiums, roofs for storage of alumina, nuclear reactor containment vessels, underwater oil storage vessels, chemical and refinery plants and hyperbolic paraboloids.

Task Group 17 recommended the following problems be studied:

1. Shell buckling under unequal biaxial stress conditions.
2. Buckling of stiffened shells with unequally spaced stiffeners.
3. Buckling of shell-like structures under non-uniform loads and concentrated loads.
4. Effects of deflections and imperfections on cylindrical and HP shell-like structures.
5. Effects of residual stresses on the buckling of shell-like structures.
6. Effects of material properties on buckling of shell-like structures.
7. Stability under erection.
Shell-like Underwater Oil Storage Structure
(Courtesy Chicago Bridge & Iron Co.)

Shell-like Storage Structure
(Courtesy Chicago Bridge & Iron Co.)
A summary of ongoing research on tubular members was presented. Included were brief discussions on the following projects:

- Plastic Design of Square and Rectangular Members sponsored at McMaster University by CIDECT.

- A study of moment-curvature relationships for D/t ratios ranging from 19 to 100, sponsored at the University of Wisconsin in Milwaukee by API, a steel producer, and a local fabricator under the direction of D. R. Sherman.

- Computer development of $M-\phi$ curves for cylindrical tubes reported by Fowler, Erzurumulu and Toprac and the April 1973 ASCE meeting.

- A full scale test program on a four foot diameter pipe considered for the underground portion of the Alaska pipeline by J. G. Bouwkamp. (This pertains to local wrinkling of pipe walls in buried pressurized pipe with longitudinal expansion loads).

- Critical plastic buckling parameters for tubing in bending under axial tension, reported by J. C. Wilhoit of Rice University at the 1973 Offshore Technology Conference.

An API Task Group headed by Larry Boston has identified several research topics of particular concern, including:

- Strength of Axially Loaded Fabricated and Manufactured Unstiffened Cylindrical Columns

- Interaction between Local and Overall Buckling for Unstiffened Cylindrical Compression Members

- Local Post-Yielding Strength of Unstiffened Cylinders in Flexure

- Interaction of Lateral Pressure and Axial Loadings for Unstiffened Members

Design documents and related information are in preparation by several groups including:

- AWS Structural Welding Committee

- WRC Subcommittee on Welded Tubular Members
The Creep Buckling Problem in High Temperature Equipment Service

M. D. Bernstein, Foster Wheeler Corporation

In the design of equipment in the creep range, possibility of creep buckling has long been recognized. In the last two years specific requirements dealing with this problem have been imposed for nuclear vessels in high temperature service. One particular design problem is the tube or cylinder under external pressure. All such equipment has some initial imperfection from its ideal shape. This ovality of out-of-roundness is often made worse by fabrication, such as tube bending, which distorts the shape and changes wall thickness.

Unlike the Euler buckling of an axially compressed strut, which is sudden, the oval tube under external pressure tends to become more and more oval. The increased ovality raises circumferential bending stress in the wall, enhances the creep strain causing still greater ovality, until creep collapse finally occurs.

The current ASME Code requirements in Code Case 1331-7 specify two safety factors on creep buckling. The first involves time: the calculated collapse time must be ten times the operating life of the vessel. The second involves the deformed shape at the end of service life: under end-of-life conditions a load factor of safety against instantaneous buckling is required. This factor may vary from 2.5 to 3.0.

Actual service conditions present the complex problem of determining the elastic plastic and creep behavior of various shapes when subject to a time varying history of temperature, pressure and mechanical loads. This task is best handled by computer, although certain limited hand analyses are available for simple cylinders.

Dr. J. M. Chern of Foster Wheeler Corporation has written a program for the elastic plastic creep buckling analysis of a long circular cylindrical shell with initial geometric imperfections subject to time varying external pressure, through the wall temperature and either axial load or strain. Agreement of this program with limited experimental results has been good.
Task Group Reports

Another, more generalized program which has creep buckling capabilities is the MARC-CDC program written by Professor P. Marcal of Brown University.

Unfortunately all computer programs for the solution of creep buckling problems are costly because the problem has to be solved over and over again as the shape changes. Special extrapolation methods may assist in shortening the problem.

Ultimate Strength of Concrete-Filled Steel Tubular Beam-Columns

W. F. Chen, Lehigh University

This paper presents a theoretical investigation of the ultimate strength of concrete-filled steel tubular beam-columns. A procedure has been developed by which the ultimate strength of such sections when used as beam-columns may be calculated by hand computation or with only minor computer usage. The procedure is based on assuming an idealized relationship among moment, curvature and thrust in the ultimate state.

It is found that the moment magnification factor given by ACI is a very acceptable and safe method of obtaining the maximum beam-column bending moment given the end moment. Thus, once the maximum end moment is computed by the method discussed in this paper, the maximum moment anywhere in the beam-column may be obtained by using $M_{MC}$ derived directly in the analysis, or conservatively, by using the ACE magnification factor.

In summary, the procedure for calculating the ultimate strength of concrete-filled steel tubular beam-columns described in this paper besides being simple, is both accurate and safe, when compared with existing experimental results.
Analysis of Concrete-Filled Steel Tubular Beam-Columns

W. F. Chen, Lehigh University

The elastic-plastic behavior of pin-ended, concrete-filled steel tubular columns, loaded either symmetrically or unsymmetrically about either of the two axes is studied using the Column Curvature Curve method. Two types of cross section are considered: circular shapes and square shapes. Three types of stress-strain relationship for concrete are studied: (a) uniaxial state of stress; (b) triaxial state of stress, the effect being assumed to increase the ductility only; not the strength; (c) triaxial state of stress, the effect being assumed to increase both the ductility and strength.

Using the corresponding stress-strain curves for concrete, interaction curves relating axial force, end moment, and slenderness ratio are presented for the maximum load carrying capacity of the beam-columns. The results obtained are compared with those from tests reported elsewhere, and good agreement is observed.
CONTRIBUTIONS OF TASK REPORTERS

TASK REPORTER 11, STABILITY OF ALUMINUM STRUCTURAL MEMBERS

J. W. Clark, Aluminum Company of America

Stability of Aluminum Members Under Large Deformations

J. W. Clark, Aluminum Company of America

This title was selected to encompass two topics: limiting width-to-thickness ratios required for flanges of aluminum beams to permit the large deformations associated with plastic design, and large deformations of tubes under axial compression where the tubes are used for energy absorption.

Figure 1 illustrates experimental values of the ratio of critical strain to yield strength strain for aluminum I-beams subjected to bending under concentrated load. The abscissa is the ratio of the clear width of the outstanding flange, $b$, to flange thickness, $t$. The limited results obtained in this investigation to date suggest that it will be possible to adapt design rules developed for plastic design in steel to aluminum alloys.

Aluminum tubes loaded in end compression make efficient energy absorbers for supporting highway guardrails or automobile bumpers. Plastic analysis of the stability of such a tube under large deformations leads to the equation in Fig. 2 for the specific energy absorbed (that is, the energy per unit weight of tube). The quantity $\rho$ in the equation is the density and $f_T$ is the tensile strength, here taken as an approximate measure of the flow stress for large deformations. The equation agrees well with results of both static tests (open symbols) and dynamic tests (closed symbols).

TASK REPORTER 13, LOCAL INELASTIC BUCKLING

L. W. Lu, Lehigh University

Elastic and Inelastic Post Buckling of Strength of Web Plates

S. N. Iyengar and L. W. Lu, Lehigh University

A method of analysis to investigate the behavior of a thin rectangular steel plate in the elastic and inelastic ranges is described. The plate, supported on all of its edges, may be subjected to transverse, in-plane or combined loadings and may have external elastic restraints. The formulation considers the effect of yielding through the thickness, unlike in previous approximate solutions where the plate was considered to be composed of two concentrated stress-carrying layers only. Assuming a number of finite layers in the plate, each layer is treated to be in a state of plane stress. The incremental theory of plasticity is used in deriving the stress-strain relationship in the inelastic range in matrix form.
Task Reporters

The governing differential equations of the elastic large deflection theory of thin plates are suitably modified to account for inelastic behavior when this occurs. The equations are then solved using finite differences and special iterative techniques.

The plate theory and solution are applied to study the behavior of the web in a rolled shape under pure compression, pure bending or combined compression and bending.
Load and Resistance Factor Design

T. V. Galambos, Washington University, St. Louis, Missouri

The report discussed the utilization of results from stability research in the development of Load and Resistance Factor Design (LRFD) criteria for steel buildings. One of the aspects of LRFD is the use of a factor, $\phi$, by which the nominal resistance, $R_n$, of a structural element is multiplied to account for the uncertainties of predicting its capacity. According to one interpretation of first-order probability-based design methodology, the resistance factor $\phi$ is equal to

$$\phi = \frac{R_m}{R_n} \exp (-\alpha \beta V_R) \quad (1)$$

where $R_m$ is the mean and $V_R$ is the coefficient of variation of the resistance, and $\alpha \beta$ is a product obtained by calibration to an existing specification. Values of $R_m$ and $V_R$ were obtained from the literature of CRC directed research for laterally unsupported beams, columns, plate girders, beam-columns, etc., to be used in the computation of $\phi$.

Some representative results are listed below (for $\alpha \beta = 1.65$, which was obtained by calibration against simple compact beams):

1. Unbraced Simply Supported Elastic Wide-Flange Beams Under Uniform Moment

$$R_n = \frac{M_{cr}}{L} \text{ lateral-torsional buckling moment}$$

$$M_{cr} = \frac{\pi E \sqrt{(G/E)JL}}{Y} \sqrt{1 + \frac{\pi^2 L_w}{(G/E)JL^2}}$$

$$R_m = (M_{cr})_{mean} = 1.03R_n; \quad V_R = 0.12; \quad \phi = 0.84$$

2. Axially Loaded Columns

$$R_n = \frac{F_{cr}}{Y} \text{ critical buckling stress}$$

$$F_{cr} = F_{cr}(1 - 0.25 \lambda^2) \quad \text{for} \quad \lambda \leq \sqrt{2}$$

$$F_{cr} = \frac{F_Y}{\lambda^2} \quad \text{for} \quad \lambda \leq \sqrt{2}$$

$$\phi = 0.86 \quad \text{for} \quad \lambda \leq 0.16$$

$$\phi = 0.9 - 0.25\lambda \quad \text{for} \quad 0.16 \leq \lambda \leq 1.0$$

$$\phi = 0.65 \quad \text{for} \quad \lambda \geq 1.0$$
This is to report on the "final draft" status of the CRC Guide. Final draft is defined as a draft approved by the task group, for which no major revisions are required, although minor editorial changes may be needed.

1. Introduction
2. Structural Safety
3. Centrally Loaded Columns

Chapters 1, 2, and 3 were mailed out for final review in March of 1972. Feedback on Chapters 1 and 2 has been minor. Major revision of Chapter 3 was recommended and at a special meeting of Task Group 1 in Pittsburgh, November, 1972, it was decided to present multiple column curves in nondimensionalized form but not to include the incomplete category selection table, nor curves or stress tabulations for specific steels. Chapter 3 is now being revised by the editor in accordance with these recommendations.

4. Local Buckling of Plates
5. Dynamic Load Effects
6. Laterally Unsupported Beams

Chapters 4, 5, and 6 were mailed out for review in late August, 1972. A number of reviews have been received but no drastic modifications have been recommended.

7. Plate Girders

A draft was received from Task Group 16 in November of 1972, but in slightly incomplete form. This chapter requires some further task group work.


A finished draft of this chapter was mailed out in April of 1971. A suggested major revision of the portion on bi-axial bending has been received. In view of the amount of research work now underway on this topic, it is planned to keep this chapter open for further revision as long as possible.
A complete draft of Chapter 9 was submitted by Task Group 13 in March, 1973. Some minor revision prior to task group approval has been suggested. The task group met on May 1 and 2 in Los Angeles and agreed on final revisions.

Drafts as approved by Task Groups 18 and 7, respectively, were submitted to the editor in August and April, respectively, of 1971. Each of these needs some shortening for final draft form.

Chapters 12, 13, and 14 were distributed in final draft form in April of 1972. Some reviews have been received but no major changes are anticipated.

Task Group 4 has put out repeated revised versions of Chapter 15, each in finished form. It is anticipated that the 9th draft, now essentially completed, will be the final version.

Chapters 16, 17, 18, and 19 were distributed in February of 1973. No major changes have been suggested.

General Comment

1. Additional critical reviews of any of the chapters are still very much in order. Special attention by CRC members might also be given to furnish the editor with recent references. References to university reports or dissertations should also be up-dated if a later published version has since become available.

2. Editorial work in Tucson is planned as follows.
   a. Revised version of Chapter 3.
   b. Shortened versions of Chapters 10 and 11.
   c. Distribution of final draft versions of Chapters 9, 10, 11, and 15.
   d. Distribution of Chapters 7 and 8.
CRC Guide Report

e. Distribution of Technical Memoranda.
f. Collation of all suggestions and preparation of complete manuscript -- to be submitted to CRC Executive Committee and Guide advisors for final approval.
g. Submission to publisher.
Progress Report on the Third Edition

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

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9. Thin-walled Metal Construction

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Progress Report on the Third Edition (cont'd.)

10. Tubular Members
11. Tapered Members

Drafts as approved by Task Groups 18 and 7, respectively, were submitted to the editor in August and April, respectively, of 1971. Each of these needs some shortening for final draft form.

12. Columns with Lacing, Battens, or Perforated Cover Plates
13. Mill Building Columns
14. Members with Elastic Lateral Restraints

Chapters 12, 13, and 14 were distributed in final draft form in April of 1972. Some reviews have been received but no major changes are anticipated.

15. Multi-story Frames

Task Group 4 has put out repeated revised versions of Chapter 15, each in finished form. It is anticipated that the 9th draft, now essentially completed, will be the final version.

16. Arches
17. Stiffened Plate Construction
18. Shells and Shell-like Structures
19. Composite Columns

Chapters 16, 17, 18, and 19 were distributed in February of 1973. No major changes have been suggested.

General Comment

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   g. Submission to publisher.
R. W. Haussler, SEASC Past President, B. G. Freeman, CRC Secretary, T. V. Galambos, CRC Chairman, L. S. Beedle, CRC Director
SPECIAL EVENING SESSION

(PANEL DISCUSSION)

FRAME STABILITY UNDER SEISMIC LOADING

MODERATOR: W. A. Milek, American Institute of Steel Construction

PANEL SPEAKERS:

E. J. Teal, A. C. Martin & Associates, Los Angeles
M. H. Mark, Ferver Engineering Company, San Diego
D. R. Strand, Brandow & Johnson Associates, Los Angeles

On the evening of Wednesday, May 2nd, the Column Research Council joined the Structural Engineers Association of Southern California at their regular monthly meeting. The dinner meeting was held at the Rodgers Young Center, where there were more than 300 structural engineers and other professionals in attendance. William A. Milek, AISC Director of Research and Engineering, New York, introduced the three panel speakers, each of whom presented an interesting slide lecture on various aspects of Frame Stability under Seismic Loading.

MR. TEAL: (Stability Considerations in Seismic Frame Design)

Steel frame stability provisions in the code are entirely concerned with column design. For columns which are not part of the seismic frame the allowable axial stress to provide safety against buckling is covered by formulas (1.5-1) and (1.5-2). These formulas include a K multiplier for effective length which is covered by sections 1.8.2 for braced frames and 1.8.3 for unbraced frames. If the frame is considered braced, K may conservatively be considered equal to 1 and the design is simple. If the frame is considered unbraced, the code requires that "the effective length K/L shall be determined by a rational method", and the argument is on. This is an argument, by the way, that only earthquake country designers are generally involved in, since arbitrary bracing feasible in other areas can usually be said to qualify a frame as braced.

The commentary takes up the question of determining the K value by a "rational method". Six cases are illustrated (Figure 1). The first four cases obviously have a K value of 1.0 or less. The last two cases are shown to have a K value of 2.0 because no lateral bracing force is shown at the top. The problem for these two cases is spelled out in the commentary as follows:

"While ordinarily the existence of masonry walls provides enough lateral support for tier building frames to prevent side-sway, the increasing use of light curtain wall construction and wide column spacing, for high-rise structures not provided with
a positive system of diagonal bracing, can create a situation where only the bending stiffness of the frame itself provides this support.

In this case the effective length factor, $K$, for an unbraced length of column, $L$, is dependent upon the amount of bending stiffness provided by the other in-plane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, $KL$ could exceed two or more story heights.

The commentary then goes on to spell out rational procedures for obtaining the $K$ value.

I don't know when there is a real practical application for this procedure, but I am certain that it is not justified with columns supported by drift controlled seismic frames.

(Figure #2) Columns with moment connections at floors (as shown by the left diagram) will resist lateral forces and be bent in an "s" shape. The critical design will not be buckling but will be $P/A + M/S$ stresses at the support. Columns with simple beam connections (as shown by the center diagram) will be supported at the floors, whether the lateral force resisting system is a braced frame or moment resisting frame. The commentary implies the possibility of a condition such as shown in the right diagram. A staying force at the floors far less than that provided for seismic design will not permit this to happen. Actually, this is generally recognized but the code certainly confuses the issue.

The design of columns which are part of a seismic resisting moment frame are covered in the code by formulas (1.6-1a), (1.6-1b) and (1.6-2). (Figure #3) The design is again complicated by code provisions for unbraced frames. This time not only is the $K$ value involved, but also the $C_m$ factor. Although the performance of seismic frames is really controlled by equation (1.6-1b), equation (1.6-1a) will often, if not generally, govern the design when the frame is considered unbraced. Consider the difference between frame design with no lateral force and frame design with lateral force. (Figure #4) Unbraced frames designed without a lateral force will fail by buckling out from under the vertical load when the assumed conditions exist, and the load is increased until the safety factor is used up. This condition is shown in the upper right diagram and is, of course, the same limiting condition assumed for simply connected columns in a braced frame, as shown in the other two top diagrams. Frames designed for significant lateral forces, lower diagrams, will not fail by buckling out from under the vertical load due to high vertical loads and unknown moments. These frames will yield (not fail) when the lateral load is increased sufficiently. They will not fail if the
lateral load is not sustained. The vertical load will not be near the buckling load.

The stability problem with regard to seismic frame columns is therefore not a buckling problem to be solved by \( \text{C}_m \) and \( \text{K} \) values tailored to an unbraced frame assumption. The values for braced frames should be used and the problem should be viewed as a frame problem, not a column problem.

The frame stability problem is a matter of total frame design to resist lateral shears and vertical \( \text{P}\Delta \) moments. (Figure #5) This diagram shows story rotation due to story drift. It also essentially shows joint rotation, if the bending of the columns is neglected. The drift coefficient actually measures the sine of the story rotation angle but, since the angle is small, it also measures the angle in radians. The drift coefficient and other coefficients shown in this and the following figures will be identified by subscript numbers for simple reference. The lateral force moment can be related to the \( \text{P}\Delta \) moment by the ratio between the lateral force coefficient and the drift coefficient. For elastic design it can readily be seen that if the drift coefficient is established, the \( \text{P}\Delta \) effect can be accounted for by an increased lateral force coefficient. Since the ratio should not exceed 10% and the lateral force coefficient is such an approximate quantity, it can also be seen that there is little reason to try to account for the \( \text{P}\Delta \) effect in elastic design.

The frame stability problem therefore resolves into a problem of \( \text{P}\Delta \) effects under large inelastic drifts due to severe earthquake motions. An early study of this problem was made by Dr. Housner. (Figure #6) The energy input into one motion of a SDF system by the earthquake ground motion is related to the Spectral Velocity as \( (W/g)(S_V^2/2) \). The vertical displacement of the load due to frame drift is equal to \( h(1-\cos \theta) \) which is approximately equal (for small angles) to \( h \theta^2/2 \). The energy involved in the \( \text{P}\Delta \) effect is therefore equal to \( Wh \theta^2/2 \). The system shown is stable as long as the yield moment times the drift angle is equal to or greater than the lateral force energy plus the \( \text{P}\Delta \) energy. The base shear yield capacity for stability can therefore be related to Spectral Velocity, \( W \), and \( h \). For instance, for a Spectral Velocity of 2.25 ft. per sec., the critical base shear yield capacity is \( .40W/\sqrt{h} \), for the system shown and the assumptions made. For a height of 16 feet the critical base shear yield shear would be \( .10W \) at a drift angle of 10%, and the lateral displacement for collapse would be 1.6 feet. For a system with more than one yield hinge, the shear constant would be proportionately smaller. Housner also developed a simple approximate formula for the base yield shear to prevent collapse of a MDF system. This involves a number of simplifying assumptions and is exceeding approximate. The period shows up in this formula but it was actually derived by reference to the number of stories which could absorb energy and the number of stories then related to period by the code period formula.

Jennings & Husid in a later investigation used a computer to subject
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the same type of SDF model to actual earthquake motion records and computed the collapse potential. (Figure #7) They found that the potential for collapse was related to the ratio between the yield capacity and the earthquake motion intensity, also the height, and the duration of strong motion. The equation can be put into yield capacity terms similar to the Housner equation. In fact, for a time to collapse of 60 seconds and a Spectral Intensity value equal to the Spectral Velocity value, the yield capacity formula is the same as the Housner formula.

A later investigation reported by Chang-Kuei Sun developed formulas relating yield displacement to collapse displacement and also to small residual displacements. (Figure #8) These displacements are related by a constant which can be shown equal to the ratio between a drift coefficient and the shear coefficient which produces that drift. This, at first seems to relate collapse potential to elastic drift control. However, the equation reduces to a simple statement that collapse occurs when the weight times the yield displacement is equal to the yield moment capacity. The constant derived in this study is kh/mg. This equals kh/W. The stiffness constant "k" can be expressed in terms of base shear C_2W and the displacement at that shear. The height "h" can be expressed in terms of displacement and drift coefficient. W and Δ cancel out and the constant reduces to C_2/C_1, or yield force coefficient over yield drift coefficient. When C_2/C_1 is substituted for C_3 in the stability equation, an equation for the yield force coefficient C_2 can be obtained. It resolves into C_2 = s/h for stability, which simply equates PΔ moment to yield capacity moment.

The important fact that comes out of all three of these analyses is that the frame collapse potential is not related to the elastic PΔ but only to the ratio between earthquake motion intensity and yield capacity of the system and to the height of the system. Stability is only related to columns because the simple systems shown do not include beams, but only a fixed end column. In practice, the hinge is generally designed to form in the girders or panel zone, not in the column.

It seems apparent that the stability problem, which is an inelastic problem, is much less critical for MDF systems than for SDF systems. However, a general solution to the stability problem for MDF systems has not yet been attempted. Computer programs for solving specific frame problems have been available, though they have been limited to small frames. Our office has recently developed a program for solving the inelastic response for very large frames. This program includes provisions for all elements of drift, including column shortening and PΔ effects, a bilinear yield relation and an energy check. An analysis with this kind of program using a wide range of earthquake motions will give a reliable dynamic prediction of the distorted shape of the seismic frame during excursions into the inelastic range. This not only provides assurance against instability but also of adequate drift control. The following figures show a comparison of elastic and inelastic analyses.
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(Figure #9) Section thru SPNB building.

(Figure #10) Typical floor plan and lateral force frame.

(Figure #11) This shows the earthquake motions used in the analyses of the SPNB building and the shears indicated by an elastic dynamic computer analysis. The height in stories is shown at the left with the Plaza as the base of the tower and the 49th floor as the top. Seven earthquake motions were used in the analyses, factored to represent maximum credible events for various fault slips as recommended by consultants Doctors Allen and Housner.

#1 - El Centro
#3 - Union Bank Sq.  x 1.5
#7 - Holiday Inn Orion  x 1.5
#2 & 4 - A1 & A2  x .67
#5 & 6 - B1 & B2  x 1.3

Code level 3000 kips, Girder yield level 9000 kips. Note that the El Centro record (#1) and the San Fernando record (#3) for the Union Bank Plaza near the SPNB site generate almost the same response shear, about 5000 kips. Note also that the elastic yield capacity is about 9000 kips. This is what we call the incipient yield capacity since it corresponds to the shear developed when one part of one member of a frame reaches the specified yield stress.

The greatest shears are developed by 1.5 times the San Fernando Orion Holiday (#7) Inn record and 1.3 times the B1 and B2 simulated records (#5 and #6). These shears are indicated to be well within the inelastic range. An inelastic analysis was therefore made first for 1.5 times the Holiday Inn record. This showed that the fully plastic yield capacity for the given shear distribution was not 9000 kips but 12,000 kips. The response was so nearly elastic that we ran the B1 earthquake with a multiplier of 2.0 to insure that the response would be well into the inelastic range. This well exceeds the intensity of all the motions predicted as credible for the site.

(Figure #12) The shear response for the two selected motions are shown here, the dashed lines showing the elastic shear and the solid lines the inelastic shear. Obviously the elastic shears represent only potential shear if the system remained elastic. Note, however, that the fully plastic shear well exceeds the incipient yield as was noted earlier.

(Figure #13) This shows the elastic O. T. response moments. The incipient yield moment was not plotted, but, since the shear was approximately the same as ground motion #4, the O. T. moment can be assumed to be also given by #4, at about 3.5 x 10^6 ft. kips.

(Figure #14) The O. T. moment response for the two selected motions show that a moment well exceeding the incipient yield moment can be generated by the inelastic response.
Panel Discussion

(Figure #15) This shows the elastic deflection curves. Notice that the deflections due to the B earthquake motions are the greatest, and far exceed the deflection due to the Holiday Inn motion, even though the base shears are similar.

(Figure #16) The comparison between the elastic and inelastic displacements tells an important story. For this building, and probably for most buildings, the extrapolation of elastic drift for motion responses into the inelastic range provides a reasonably good inelastic displacement estimate. For 2.0 times the B1 motion the response is well into the inelastic range but the displacement is actually less than that predicted by the elastic analysis.

(Figure #17) A more significant measure of displacement is the drift coefficient. Note that the El Centro drift does not exceed .4%, and the most severe drift for any of the motions examined is less than 1%. The sharply reduced drift at two locations occur at the mechanical equipment stories which have intermediate framing to reduce the drift.

(Figure #18) The inelastic vs. elastic drift coefficient curves are not quite as favorable as the total displacement curves. However, the difference is not great or cause for concern. Note again that, even for the "incredible" 2.0 times B1, the drift coefficient is not much over 1.5% at any story.

(Figure #19) This shows the absolute acceleration for an elastic response. Notice that there is little or no general amplification, or whipping, up through the building. The absolute floor accelerations remain essentially of the same magnitude as the base acceleration.

(Figure #20) For excursions well into the inelastic range it is seen that the absolute accelerations seem to be greater than those predicted by an elastic analysis. However, the maximum indicated absolute acceleration still did not greatly exceed the base acceleration.

(Figure #21) This figure shows the total energy input to the building by the earthquake ground motion as a function of the time of the strong motion. The B1 record is 30 seconds long, but the motion tapers off after 24 seconds so that is all that was run. For an inelastic analysis of a frame this large, the computer time required for each analysis step is such that the run must be cut down to the essential parts of the motion record. The grey area shows the energy dissipated by the frame. The white area is the energy stored as elastic energy which is released when the motion forces stop.

(Figure #22) This figure shows the breakdown of the dissipated energy according to the assumptions of the analysis. A damping factor of 5% was
Panel Discussion

assumed and it can be seen that this is the most significant part of the dissipated energy. The energy termed as hysteresis represents the inelastic yield energy assumed to take place after a yield section is fully plastic. Of course the two terms are, in most ways, measuring the same energy. The damping measures small local yielding and the hysteresis measures the inelastic energy of fully plastic hinges.

(Figure #23) In order to assure the validity of an inelastic analysis it is necessary to see that all of the energy is accounted for. This figure shows the energy imbalance due to very small errors which creep into the hundreds of thousands of calculations made by the computer. It can be seen that the error is kept to about 2% for most of the run.

(Figure #24) This last slide shows the occurrence of hinges up through the building during the time of the earthquake motion. Note that the height of the blip indicates the degree of inelastic yielding in the story. When the blip reaches the parallel line above its base, all girders in that frame at that story are hinging. In this case that represents 36 hinges. The width of the blip measures the time of hinging. It can be seen that the longest time of any hinge formation is about 1 second. Notice also how few times during the entire ground motion that hinging takes place in any member. It is very interesting and reassuring to see that the hinging involves most of the building in the energy absorption rather than concentrating at one or a few stories.

In conclusion:

It seems apparent that any stability problem we may have is not linked to column design for unbraced frames. We should consider seismic frames to be braced frames for the purpose of column design. It seems also to be apparent that any PΔ problem we have is not an elastic problem to be provided for by an assumed PΔ moment based on a drift assumption. The key to stability design for seismic frames is an adequate yield shear capacity which will absorb the energy from credible earthquake motions and therefore control inelastic drift. For MDF systems we don't know what the lower limit is. We do know that our present code values do not seem to be too far off, but are on the low side. It is indicated that low story ductility factors are not a problem and may be a benefit, but that high story ductility factors may risk inelastic instability. The word ductility, as used here, means simply the factor Spectral Intensity response over yield capacity.

The most serious possibility of building instability due to column buckling under seismic loading is the underestimating of overturning loads on columns by the use of code lateral forces which have been factored down from real dynamic response forces. Column stress amplification factors based on column curvature assumptions do result in some added column capacity available for excessive loading, but this is an irrational, and ineffective way to attack the problem.
Panel Discussion

The most serious possibility of instability due to insufficient yield moment capacity is involved in low simple structures which involve few yield hinges and large rotations for small lateral displacements.
\[
\frac{f_a}{F_a} + \frac{C_m}{(1-\frac{f_a}{F_a})}, f_b \leq 1.0 \quad (1.6-1a)
\]

\[
\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad (1.6-1b)
\]

\[
\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad (1.6-2)
\]

**FIGURE 3**

\[
\sin \theta = \frac{\Delta}{h} \quad \theta = \frac{\Delta}{h} = C_1, \text{ DRIFT COEFF.}
\]

\[
M_o = C_2 h W \quad M_a = C_2 W h
\]

**FIGURE 5**

**FIGURE 4**

**SDF SYSTEM**

\[
W = \frac{S_v^2}{g} W + \frac{w}{2} h g' = M_o \theta
\]

\[
M_0 = S_v \sqrt{g \frac{1}{g}} W \quad V_b = \frac{S_v}{\sqrt{g h}} = \frac{S_v}{5.7/h} W
\]

FOR \( S_v = 2.25 \)

\[
V_b = \frac{40}{h} W \quad a_c = -\frac{40}{h}
\]

**MDF SYSTEM**

\[
V_b = \frac{S_v}{\sqrt{g h}} \sqrt{g W} \quad \text{FOR } h = 12.5' \quad V_b = 0.4 W
\]

**FIGURE 6**
\[ t^* = \frac{2000}{h/C_4^2} \]
\[ C_4 = \frac{E}{C_2} \]
\[ C_2 = \frac{t^* E^2}{2000h} \quad C_2 = \frac{E \sqrt{t^*}}{45/h} \]

**FOR** \( t = 60 \) seconds

\[ C_2 = \frac{E}{5.8\sqrt{h}} \]

**FOR** \( E = 2.25 \)

\[ C_2 = \frac{40}{\sqrt{h}}W \]

**FIGURE 7**

**SDF SYSTEM**

\[ h \]

\[ V = C_2W \]

**FOR STABLE CONDITION**

\[ \varepsilon \leq C_3 \varepsilon_y \]

\[ C_3 = \frac{k h}{\rho} = \frac{k h}{W} \]

**FOR SMALL RESIDUAL DISPLACEMENT**

\[ \varepsilon \leq \sqrt{C_3 \varepsilon_y} \]

\[ k = \sqrt{h} - C_1 h \]

\[ \varepsilon_y = \Delta y - C_1 h \]

\[ \frac{k}{C_1} = \frac{C_3}{\rho} \]

\[ \therefore C_3 = \frac{C_3}{\rho} \]

\[ \frac{C_3}{\rho} = \frac{C_2}{C_1} = \text{YIELD FORCE COEFF.} \]

\[ \frac{C_3}{\rho} = \text{YIELD DRIFT COEFF.} \]

**FIGURE 8**

**FIGURE 7**

**FIGURE 8**

**FIGURE 10**
SECURITY PACIFIC NATIONAL BANK
HEADQUARTERS BUILDING
LOS ANGELES, CALIFORNIA
MAXIMUM DYNAMIC RESPONSES

FIGURE 15

SECURITY PACIFIC NATIONAL BANK
HEADQUARTERS BUILDING
LOS ANGELES, CALIFORNIA
MAXIMUM DYNAMIC RESPONSES

FIGURE 16

SECURITY PACIFIC NATIONAL BANK
HEADQUARTERS BUILDING
LOS ANGELES, CALIFORNIA
MAXIMUM DYNAMIC RESPONSES

FIGURE 17

SECURITY PACIFIC NATIONAL BANK
HEADQUARTERS BUILDING
LOS ANGELES, CALIFORNIA
MAXIMUM DYNAMIC RESPONSES

FIGURE 18
SECURITY PACIFIC NATIONAL BANK
HEADQUARTERS BUILDING
LOS ANGELES, CALIFORNIA
MAXIMUM DYNAMIC RESPONSES

FIGURE 19

SPNB H/Q BUILDING
MAX. DYNAMIC RESPONSES

FIGURE 20

TOTAL ENERGY

STRUCTURE: SPNB
DAMPING: 5%
YIELD STRESS: 40 KSI

FIGURE 21
Dissipated Energy

Structure: SPNB
Damping: 5%
Yield Stress: 40 KSI
E/Q: 2.0 x 81

Figure 22

Energy Imbalance

Structure: SPNB
Damping: 5%
Yield Stress: 40 KSI
E/Q: 2.0 x 81

Figure 23

Girder Hinges

Structure: SPNB
Damping: 5%
Yield Stress: 40 KSI
E/Q: 2.0 x 81

Figure 24
Panel Discussion

MR. MARK: (Section 2313 J.1.D and Steel Frames)

As revised this year Section 2313 J.1.D will appear in part in the new edition of the Blue Book as follows:

"All framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load carrying capacity and induced moments due to four times the distortions resulting from the code required lateral forces."


It is certainly an understatement to say that this new section has caused some confusion. This talk is an attempt to shed some light on how the provision affects the design of low and moderate rise steel moment frame buildings.

It should be noted that to satisfy the new code provision:

1. Only the members not part of the lateral force resisting system need to be shown as adequate and then adequate only to carry their vertical loads.
2. For this provision the lateral force system does not need to be shown as adequate to withstand the four times code level distortions. The provision presumes that a properly designed system will have been provided with the ability to safely distort to this extent either by elastic or inelastic behavior. Mr. Teal has given and Mr. Strand will give his thoughts on providing an adequate lateral force resisting system and I note that some of the papers presented at the Column Research Council meetings today also dealt with this subject. I will not.
3. The new provision implies that the distortions should be calculated using the stiffness of the lateral force system only, even though other members not part of the system may actually be resisting drift.

For ordinary steel frame buildings two effects may preclude non-seismic system members from carrying their vertical loads at high drift levels: P-Δ and induced bending.

(Figure #1) This illustrates the P-Δ effect. Due to the drift Δ, a bracing force V_{pΔ} is needed to keep the column from overturning. The term H is the column's height. The bracing force is equal to the column load multiplied by the drift angle θ.

Now if the column were an interior column of this building (Figure #2) the bracing force would have to come from the perimeter frames. The picture shows the floor framing plan of a popular type of steel building. The rigid
Panel Discussion

frames are on the perimeter lines while all interior girders and columns are simply connected. The bracing comes solely from the rigid frames. For instance, Frame A provides all the north-south bracing for the westerly half of the building. Frame A is therefore bracing itself, half of Frames 1 and 7 and half of the interior core columns.

If these moment frames are designed to take card of the P-Δ effects of the non-acting columns, the new code provision is satisfied for P-Δ. The P-Δ effects are included in the design at code force level—not at the 4 times code force level as it is presumed that if the frames can resist the P-Δ effect at the more realistic, higher drift level.

The K factor for the interior columns and for the out-of-plane mode for the frame columns is unity. The bracing provided by the frames makes this factor one.

If this building were altered and all the columns were part of a two-way frame system then the interaction equation (Figure #3) could handle the P-Δ problem be means of the amplification factor. This amplification of the bending term is similar to calculating an additional bending moment in the previously shown column (Figure #1) resulting from the bracing force V_\text{PΔ}.

For the case where all the columns are not part of the frame system the amplification term (Figure #3) is not adequate as small f_a represents the load only the frame column being investigated and does not account for the requirement to provide bracing for other column loads that are not part of the frame system.

This suggests one way of handling the P-Δ problem. The amplification term can be altered so that the equation reads (Figure #4) as shown. The new amplification term now reflects all the column loads tributary to the frame and increases the bending effect more than the AISC equation. ΣP is the sum of all column loads braced by the frame — those within the frame and all others tributary to the frame. ΣP'e is the Euler capacity, with safety factor, of all the frame columns. This new equation is based on the theory that all the columns of the frame buckle in a sidesway mode simultaneously.

A disadvantage of relying only on this equation is that available computer programs must be altered, calculated drifts do not include the secondary drifts from P-Δ, girder bending is not amplified and the bending term of the other AISC interaction equation that must be checked is not amplified.

An alternate method of handling P-Δ overcomes these disadvantages. (Figure #1) This method includes the P-Δ effect as an additional story
shear which can be computed as the drift angle multiplied by the sum of the loads, dead or live, tributary to the frame in question. These P·Δ story shears are added to the code seismic shears and with the possible exception of omitting the amplification factor the analysis is done in the usual way with available computer programs for the frames.

Again it must be remembered that for this building (Figure #2) Frame A is bracing half the building and thus takes 50% of the P·Δ shear.

The drift used for calculating the extra P·Δ story shears could be the limiting drift established for design or if drifts are kept low, 5% to 15% more than the drift for code seismic shears acting alone.

There is some question as to proper frame column K factors for a design incorporating these P·Δ shears and some argument for using an in-plane K factor close to unity. For the usual case where bending governs the frames it probably doesn't matter much. But when the column loads are high in proportion to bending this approach may be unsafe. I suggest for the time being, until better methods become established, that K factors be computed in the usual manner, for instance, using the AISC Monograph.

One item that should be mentioned before leaving the P·Δ effect is that frame sidesway buckling should be checked for this type of structure. If the design lateral forces are low or frame stiffness is low, the entire structure may not possess the code required safety factor for gravity loads acting alone.

(Figure #5) This can be checked by the method illustrated based on the theory that the frames sway as a unit and that moderate gravity moments do not significantly affect sidesway buckling. The sum of the in-plane axial load capacity of the frame columns should be larger than the sum of all column loads tributary to the frame.

Although this condition of sway buckling is unlikely it could be checked at a few levels.

The second effect mentioned by the new code provision is induced bending. The drift level to be considered is four times the drift caused by code forces acting on the resisting system.

(Figure #6) For simply connected girders and columns it is consistent to assume that simple framing connections have sufficient rotation capacity to safely withstand the column rotations without failure and without inducing significant moments in the members.

Some columns could be restrained in a manner that would induce significant bending such as this interior column restrained by a rigid basement.
Panel Discussion

If the column section were compact and laterally braced so that a plastic hinge could form at the ground floor level, no further check is necessary. The column should carry its vertical load.

If the column section were not compact, it is possible that local flange or web buckling could occur. This does not necessarily mean that the column would not carry its vertical load. For instance, even though local flange buckling might occur there may be enough web section (if the web is thick) to carry the vertical load. In any event reinforcing such as web material or flange stiffeners may be added so that the vertical load could be transmitted at this location.

If the column flanges are not wide there is also a possibility that combined lateral-torsional and column buckling could occur due to the induced moments and lack of flange bracing. Although there is some question as to whether this failure mode would be self limiting it should be investigated and appropriate design revisions made if required.

Column splices, especially partial penetration types of minimum size could also be a problem in this region of high moment.

Allowable stresses for the local and lateral-torsional buckling checks could be high as the code requirement is a near ultimate condition. I suggest a 50% increase on normal working stresses which will bring factors of safety in the range of 1.1 to 1.3.

For steel buildings this new code requirement will result in little additional design time. The calculation of P-Δ shears is not time consuming and once done the analysis proceeds in the usual manner. Many firms have been incorporating the P-Δ effects in their designs for some time.

For buildings designed with an adequate drift limit calculating the P-Δ effects will appear to be only increasing the numbers a little. This is true. But if the drifts are too high the effects will be significant and will show up in the analysis.

What the new provision does do is force recognition of the P-Δ problem. Hopefully it will aid in providing designs incorporating adequate frame stiffness - for stability.
OVERTURNING \( M' = \text{RESTORING} \ M' \)
\[ P \Delta = V_{pa} h \]
\[ V_{pa} = P \frac{h}{h} = P \theta \]

\[
\frac{f_a}{F_a} + \frac{C m_\text{f} f_b}{\left(1 - \frac{f_a}{F_e}\right) F_b} \leq 1 \quad \text{(AISC EQ. 1.6-1a)}
\]

AMPLIFICATION FACTOR = \[
\frac{1}{\left(1 - \frac{f_a}{F_e}\right)}
\]

\[
\frac{f_a}{F_a} + \frac{C m_\text{f} f_b}{\left(1 - \frac{\sum P}{\sum P_e}\right) F_b} \leq 1
\]

\( \Sigma P = \text{SUM OF ALL COLUMN LOADS} \)
\( \text{BRACED BY THE FRAME.} \)

\( \Sigma P_e = \Sigma (A F_e) \text{ FOR ALL THE COLUMNS} \)
\( \text{OF THE FRAME ONLY.} \)
CHECK: $\Sigma P_a \geq \Sigma P$

$\Sigma P_a = \Sigma (A F_a)$ of all the frame columns (V thru Y).

$F_a =$ in plane of frame allowable axial stress.

$\Sigma P =$ all the column loads tributary to the frame (U thru Z).

FIGURE # 5

HIGH MOMENT INDUCED AT GROUND FLOOR

FIGURE # 6
Panel Discussion

MR. STRAND: (Role That Connections Play on Frame Stability)

There is no question that the connection required to hold a structure together must be the last to fail after yielding of the members. For structural steel frames the codes are explicit as to requirements.

The Structural Engineers Association of California in their "Blue Book" recommended that each girder moment connection to a column be capable of developing in the girder the full plastic capacity of the girder. The connection has been defined to exclude the whole joint assemble and only consider the attachments to the joint. Further, it is recommended that the members shall comply with the plastic design sections of the code. This will require "compact sections" or conformance to Sections 2.1 through 2.9 of the Commentary to the AISC Specifications.

At the present time only limited testing has been done on steel frames, primarily at Lehigh for monotonic testing and at the University of California for cyclic testing. These tests have been on limited sizes of members and modeled down to small sizes and spans. The larger members have indicated good results but the usable criteria has not been placed into terms readily adaptable to the practicing engineer. Actual tests on the University of California shaking table will be looked at with interest since actual forces will be simulated and bracing of the test frames will be more as an independent system versus the braced stands required for assemblage tests.

The design of a frame girder connection to column (in the strong axis) can readily be done as illustrated in the attached sheets. Based on previous tests the plastic moment of the girder can be developed by the flanges alone. The web of the girders are not designed to be fully developed by the web shear connection, but are designed for the girder shear resulting from the full plastic moment at each end of the beam plus the vertical load acting. Any moment developed by the girder web connection is assumed to be carried by the web connection.

The type of web connection is based on the development criteria desired. The tests thus far indicate good performance from either shear tab web connections to columns or by use of full penetration welds. In all cases the girder flanges had full penetration welds to the column. Further indications are that bolted webs may be preferred with the use of horizontal slotted holes to minimize induced moments into the web connection. The use of welded web connections require consideration to a weld sequence to minimize the locking up of stresses to members within the frame.

The instability of beam members in the frame require consideration to lateral instability of flanges not stayed against reversed loading conditions. A design criteria has yet to be established, but use of compact sections and bracing of compression flanges appears to be best at this time.
Panel Discussion

Instability of columns has not been demonstrated except under very high lateral displacements. Few tests have investigated these large displacements in the weak axis of column. Criteria for instability of columns in the weak axis or with loads in two axis concurrently are lacking. Investigation of strain in the members has been critical under very high distortions but is probably not critical when drift is controlled.

A usable criteria for design of the column panel zone within the girder depth is still to be obtained. The drift of a frame is based on the rotation due to the girder, column and panel zone. The girder and column plastic deformation can be predicted reasonably well but panel zone drift is much more complex.

Weak panel zones initiate lateral torsional buckling since the plastic hinge forms first in this zone. This distortion can cause buckling in the panel zone web or local kinks in flanges with resultant weld cracks. In the opposite sense, strong panel zones place all the joint rotation into the beams and columns and may place too much demand on the welds of the beam connection. A balanced design wherein all components have general yielding simultaneously appears to be appropriate.

The design of the panel zone for stiffener requirements is met by a "working stress" criteria with arbitrary equations in AISC. The shear stress transfer through this area is also based on this same principal. Whether this criteria is applicable under cyclic seismic conditions is not fully established. With plastic hinges in the girders and columns, the shear induced into this joint would require strengthening of most column webs in this zone.

Interaction of vertical load P-Δ effects, and shear stress through the panel zone are now becoming known and the transfer to the column flanges needs further study. Equations proposed for design are based on general yielding and are adequate only if the induced distortions are not too great. Where this area is too rigid, local buckling may be transferred outside of the panel zone area.

A means of assuring good panel zone design is by use of boundary members around the area. These boundaries can be provided by the column flanges and the addition of stiffeners which then may be analyzed as a form of rigid frame under shear deformation.

The design of the stiffeners based on the AISC Specs equation 1.15-4 appears to be adequate for development of forces that may be induced by girder plastic moments. It has been recommended that the stiffener weld be adequate to develop the full yield value of the plate and further that the plate be proportioned to meet AISC width to thickness ratios. The load level at which column webs are required to be reinforced has not been
identified. Reasonable member sizes, possibly controlled by stiffener criteria, may be quite adequate for normal loading conditions.

Inelastic shear buckling has occurred under cyclic loading at high strains, but normal elastic shear buckling does not occur under most conditions due to the column web dimensions and by the beam connection normal to the column in actual structures. A large reserve strength of shear yield to shear ultimate strength has been demonstrated thus far in tests. Buckling does not appear to reduce the strength of this zone, probably by strain-hardening effects.

From this information it may be assumed that an adequate design criteria may be obtained by:

1. Developing the full flange capacity of the girders.
2. Designing the girder web connection based on plastic end moments and vertical load moments acting together on the girder.
3. Using reasonable column sizes with respect to the girders such that shear stresses through the column web are less than yield. The axial load relation proposed should not exceed: 
\[(P/Py)^2 + (V/Vy)^2 = 1\]
Design should be based on the elastic (working) stresses that may be developed in the members as opposed to plastic design data.
4. The above relations are adequate even when stiffeners are required although more distortions could be expected through the panel zone.
5. Stability of the frame need not be checked unless these criteria are exceeded.

Further testing is still required to confirm the following:

1. Weak axis bending in columns.
2. Instability of the weak axis.
3. Concurrent loadings in both axis of a column.
4. Tests on higher strength steels.
5. Plastic design criteria for panel zones.
6. Stiffener design and details of welding.
7. Doubler plate requirements for column panel zones.
8. Fatigue design requirements of members and welds.
9. The P-\(\Delta\) effect on the panel zone capacity or design.
The Council holds an annual meeting for the purpose of reporting activities, election of members and officers, and presentation of the budget for the following year. The 1973 Annual Meeting was held on May 3rd in conjunction with the Technical Session at the Los Angeles Hilton, Los Angeles, California.

The minutes of the 1973 Annual Meeting are as follows:

CALL TO ORDER

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

The Chairman introduced himself and welcomed the members and friends. He introduced the Director, Dr. L. S. Beedle, and the Secretary, Dr. B. G. Freeman.

The minutes of the 1972 Annual Meeting (March 22, 1972 at the Conrad Hilton Hotel, Chicago, Illinois) are included in the 1972 Proceedings, which will be available in the very near future.

REPORT OF ACTIVITIES

The Director gave a few highlights of the technical work of the Council over the past year. The extent of the work and the progress of the various task groups is illustrated by the presentations made at the Technical Session. Task Group 12 has a new chairman, A. L. Johnson, and Task Group 14 has a new chairman, A. P. Cole.

Further progress is noted in that Task Group 9, Curved Compression Members, has completed its work and asked to be discharged.

CRC GUIDE

The Director reported that the preparation of the Third Edition of CRC's "Guide to Design Criteria for Metal Compression Members", under the editorship of Dr. Bruce G. Johnston, has grown to be a larger task than originally anticipated. A detailed progress report was presented by Dr. Johnston at the Technical Session. Johnston urged that anyone knowing of bibliographic material not already included please furnish this information.

PARTICIPATING ORGANIZATIONS

The Director thanked all participating organizations for their continued support of the Council's work. He remarked that one of our most faithful participating organizations, the Structural Engineers Association of Southern California (our host here in Los Angeles) has been a contributing member for each of the past 28 years!
The Institution of Structural Engineers and the Institution of Civil Engineers, both of London, England, have been invited to join as participating organizations.

Invitations have also been extended to The Institution of Engineers, Australia (Sydney), National Research Council of Canada, and the Japan Society of Civil Engineers.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director, including the proposed budget for the fiscal year 1973-74.

**BUDGET 1973-74:**

<p>| | |</p>
<table>
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The motion that the budget be approved (L.S. Beedle, J.W. Clark) was carried.

EXECUTIVE COMMITTEE

**ELECTION OF OFFICERS:** The Nominating Committee, chaired by P. B. Cooper, had earlier nominated the present Chairman and Vice Chairman, Prof. Galambos and Prof. Winter, each for a one-year extension of their present terms. The Secretary reported the results of a balloting of the voting members of the Council (Representatives of Participating Organizations and Members-at-Large). Both Prof. Galambos and Prof. Winter were reelected.

**ELECTION OF MEMBERS TO FILL VACANCIES CAUSED BY EXPIRING TERMS:** The Nominating Committee nominated two incumbents, J.S.B. Iffland and W. A. Milek, along with Charles Birsteil, for three-year terms on the Executive Committee. Ira Hooper declined nomination due to pressures of work. The motion (B.G. Johnston, J. Springfield) that the three nominees be elected was carried unanimously.

**MEMBER APPOINTED BY CHAIRMAN:** The Chairman reported that L.K. Irwin last week resigned from the Executive Committee. The Chairman appointed L. A. Boston to fill the vacancy effective immediately.
MEMBERS-AT-LARGE

CANVASSING OF EXISTING MEMBERS: The Secretary reported the response to the three-year canvassing of Members-at-Large.

ELECTION OF NEW MEMBERS-AT-LARGE: The following persons have been nominated by the Executive Committee:

- Prof. S. O. Asplund, Chalmers University of Technology, Gothenburg
- Dr. R. Bjorhovde, American Institute of Steel Construction
- Prof. D. A. DaDeppo, University of Arizona
- Dr. J. H. Daniels, Lehigh University
- Mr. W. E. Edwards, Bethlehem Steel Corporation
- Mr. G. S. Fan, Bethlehem Steel Corporation
- Dr. M. P. Gaus, National Science Foundation
- Prof. S. C. Goel, University of Michigan
- Mr. R. G. Kline, U. S. Steel Corporation
- Mr. P. M. Marshall, Shell Oil Company
- Mr. C. W. Pinkham, S. B. Barnes & Associates
- Mr. F. D. Sears, Federal Highway Administration
- Mr. D. Sfintesco, C T I C M, Paris
- Dr. W. P. Vann, Texas Technological University
- Dr. C. K. Wang, University of Wisconsin

The motion that all nominees be elected as Members-at-Large (J. Clark, E. Gaylord) was carried unanimously.

NEXT ANNUAL MEETING

The Chairman announced that the next Annual Meeting of the Council will be held in Houston in the spring of 1974.

ADJOURNMENT

The Chairman expressed thanks to the members present and to Don Wiltse, Executive Secretary of the Structural Engineers Association of Southern California, and his secretary, Patricia Lindsey, for their help in making the entire conference so successful.

At 12:15 p.m. the motion (L. Boston, E. Gaylord) that the Annual Meeting be adjourned was carried. The Chairman reminded the Executive Committee to reconvene for a short meeting.
Program of Technical Session and Annual Meeting

Wednesday, May 2, 1973

8:00 a.m. - Registration

8:30 a.m. - MORNING SESSION

Presiding: B. G. Johnston, University of Arizona

INTRODUCTION

T. V. Galambos, Chairman, CRC

TASK GROUP REPORTS

Task Group 1 - Centrally Loaded Columns
Chairman, J. A. Gilligan, U.S. Steel (L. Tall, Lehigh Univ. presiding)
Considerations and Consensus of TG-1 with respect to Multiple Column Curves
W. A. Milek, AISC
Comparison of U.S. and European Approaches....
R. Bjorhovde, AISC (presented by L. Tall, Lehigh University)
Future Research on Centrally Loaded Columns
L. Tall, Lehigh University

Task Group 3 - Ultimate Strength of Columns with Biaxially Eccentric Loads
Chairman, J. Springfield, Carruthers and Wallace, Ltd.
Design Criteria for Steel H-Columns Under Biaxially Eccentric Load
W. F. Chen, Lehigh University
Ultimate Strength of Columns Under Biaxially Eccentric Load
E. H. Gaylord, University of Illinois
Hybrid Computer Analysis for Load-Carrying Capacity of Biaxially-Loaded Steel Beam-Column
T. M. Baseheart, University of Cincinnati, and
B. C. Ringo, University of Cincinnati, and
J. F. McDonough, University of Cincinnati
Design Recommendations for Beam-Columns,
J. Springfield, Carruthers and Wallace, Ltd.

10:20 a.m. - BREAK

Task Group 4 - Frame Stability and Effective Column Length
Chairman, J. S. B. Iffland, Praeger-Kavanagh-Waterbury
Ultimate Strength of Frames Designed by the Allowable-Stress Method
L. W. Lu, Lehigh University
Ultimate Load Capacity of Plane Multi-Story Steel Rigid Frames
S. Liapunov, New York University
Task Group 7 - Tapered Members

Chairman, A. Amirikian, Navy BUDOCKS (G.C. Lee, State Univ. of New York presiding)

Approximate Analysis of Tall Buildings by Non-prismatic, Thin-walled Beam Theory

G. C. Lee, State University of New York

12:00 Noon - LUNCH

1:15 p.m. - AFTERNOON SESSION

Presiding: T. V. Galambos, Washington University

Task Group 8 - Dynamic Instability

Chairman, D. A. DaDeppo, University of Arizona

Response of a Column Subjected to Periodic Axial Force

D. Krajcinovic, Argonne National Laboratory

Dynamic Stability of Yielding Structures

S. C. Goel, University of Michigan, and
Y. L. Wang, University of Michigan

Nonplanar, Nonlinear Oscillations of a Beam-Column

R. A. Scott, University of Michigan, and
C. H. Ho, University of Michigan, and
J. C. Eisley, University of Michigan

Task Group 9 - Curved Compression Members

Chairman, W. J. Austin, Rice University

Buckling of Shallow Arches

W. J. Austin, Rice University

Task Group 11 - European Column Studies

Chairman, D. Sfintesco, C.T.I.C.M.,
Vice Chairman, W. A. Milek, AISC

Summary of Colloquium on Column Strength, Paris, November 1972

W. A. Milek, AISC

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

Chairman, G. F. Fox, Howard, Needles, Tammen & Bergendoff
(A.L. Johnson, AISI presiding)

Status of Quest for Steel Properties in Inelastic Range

A. L. Johnson, AISI

3:00 p.m. - BREAK

Task Group 13 - Thin Walled Metal Construction

Chairman, S. J. Errera, Bethlehem Steel Corporation

Design Criteria for Interaction of Local and Overall Buckling

T. Pekoz, Cornell University, and
J. DeWolf, Cornell University, and
G. Winter, Cornell University
Buckling Behavior of Perforated Unstiffened Compression Elements and Beam Webs

C. S. Davis, Ford Motor Company, and
W. W. Yu, University of Missouri-Rolla

Wall Stud Design Criteria

T. Pekoz, Cornell University, and
A. Simaan, formerly Cornell University

Task Group 14 - Horizontally Curved Girders

Acting Chairman, J. L. Durkee, Bethlehem Steel Corp.

Recent Research in Instability of Curved Plate Girders and Box Girders

C. G. Culver, National Bureau of Standards
(presented by B. Yen, Lehigh University)

4:30 p.m. - ADJOURN

8:00 p.m. - EVENING SESSION

PANEL DISCUSSION

Frame Stability Under Seismic Loading

Moderator: W. A. Milek, AISC, New York

Panel Members:

E. J. Teal, A. C. Martin and Associates, Los Angeles
M. H. Mark, Ferver Engineering Co., San Diego
D. R. Strand, Brandow & Johnson Associates, Los Angeles

THURSDAY, May 3, 1973

8:00 a.m. - MORNING SESSION

Presiding: D. Wiltse, Structural Engineers Association of Southern California

Task Group 16 - Plate and Box Girders

Chairman, F. D. Sears, Federal Highway Admin. (B. T. Yen, Lehigh Univ. presiding)

Brief Summary of Research on Steel Box Girders Abroad

B. T. Yen, Lehigh University

Failure Tests of Composite Rectangular Box Girder Models

B. T. Yen, Lehigh University

Task Group 17 - Stability of Shell-Like Structures

Chairman, K. P. Buchert, University of Missouri - Columbia

Applications of Shells and Shell-like Structures

C. D. Miller, Chicago Bridge & Iron Co., and
J. O. Crooker, Butler Manufacturing Co., and
S. Bucksbaum, American Bridge, United States Steel, and
K. P. Buchert, University of Missouri - Columbia
Task Group 18 - Tubular Members
Chairman, A. L. Johnson, AISI
Current Research of Design Developments in Tubular Members
A. L. Johnson, AISI
The Creep Buckling Problem in High Temperature Equipment Service
M. D. Bernstein, Foster Wheeler Corporation
Ultimate Strength of Concrete-Filled Steel Tubular Beam-Columns
W. F. Chen, Lehigh University

Task Group 20 - Composite Members
Chairman, S. H. Iyengar, Skidmore, Owings & Merrill
Analysis of Concrete-Filled Steel Tubular Beam-Columns
W. F. Chen, Lehigh University

10:00 a.m. - BREAK

10:20 a.m. - TASK REPORTERS
Task Reporter 11 - Stability of Aluminum Structural Members
Stability of Aluminum members With Large Deformations
J. W. Clark, Aluminum Company of America
Task Reporter 13 - Local Inelastic Buckling
Elastic and Inelastic Post Buckling Strength of Web Plates
L. W. Lu, Lehigh University

RESEARCH REPORTS
Stability Problems in Load Factor Design
T. V. Galambos, Washington University, St. Louis

CRC GUIDE
Chairman, E. H. Gaylord, University of Illinois
Progress Report on the Third Edition
B. G. Johnston, University of Arizona

11:30 a.m. - CRC ANNUAL BUSINESS MEETING
# ANNUAL MEETING & TECHNICAL SESSION ATTENDANCE

<table>
<thead>
<tr>
<th>Participant</th>
<th>Affiliation</th>
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<tbody>
<tr>
<td>W. J. Austin</td>
<td>Rice University</td>
</tr>
<tr>
<td>T. M. Basehart</td>
<td>University of Cincinnati</td>
</tr>
<tr>
<td>L. S. Beedle</td>
<td>Lehigh University</td>
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<tr>
<td>M. D. Bernstein</td>
<td>Foster Wheeler Corporation</td>
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<tr>
<td>L. A. Boston</td>
<td>J. Ray McDermott &amp; Co., Inc.</td>
</tr>
<tr>
<td>K. P. Buchert</td>
<td>University of Missouri-Columbia</td>
</tr>
<tr>
<td>S. Bucksbaum</td>
<td>American Bridge, U. S. Steel</td>
</tr>
<tr>
<td>W. F. Chen</td>
<td>Lehigh University</td>
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<tr>
<td>J. W. Clark</td>
<td>ALCOA Laboratories</td>
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<tr>
<td>R. G. Clary</td>
<td>McDonnell Douglas Automation</td>
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<td>J. O. Crooker</td>
<td>Butler Manufacturing</td>
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<td>C. S. Davis</td>
<td>Ford Motor Company</td>
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<td>T. Dembie</td>
<td>Dominion Bridge Company</td>
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<tr>
<td>R. C. Edgerton</td>
<td>National Research Council</td>
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<tr>
<td>S. J. Errera</td>
<td>Bethlehem Steel Corporation</td>
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<tr>
<td>L. E. Escalante</td>
<td>L. A. Department of Water &amp; Power</td>
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<tr>
<td>R. L. Foley</td>
<td>Structural Engineers Assoc. of So. Calif.</td>
</tr>
<tr>
<td>G. F. Fox</td>
<td>Howard, Needles, Tammen &amp; Bergendoff</td>
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<td>B. G. Freeman</td>
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<td>T. V. Galambos</td>
<td>Washington University</td>
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<td>E. H. Gaylord</td>
<td>University of Illinois</td>
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<tr>
<td>M. I. Gilmore</td>
<td>Canadian Inst. of Steel Construction</td>
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<tr>
<td>S. C. Goel</td>
<td>University of Michigan</td>
</tr>
<tr>
<td>D. H. Hall</td>
<td>Bethlehem Steel Corporation</td>
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<td>W. C. Hansell</td>
<td>Bethlehem Steel Corporation</td>
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<td>S. I. Hart</td>
<td>Parsons</td>
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<tr>
<td>R. W. Haussler</td>
<td>Consulting Structural Engineer</td>
</tr>
<tr>
<td>J. S. Iffland</td>
<td>URS/Madigan-Praeger, Inc.</td>
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<tr>
<td>A. L. Johnson</td>
<td>American Iron &amp; Steel Institute</td>
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<td>D. L. Johnson</td>
<td>Butler Manufacturing Company</td>
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<td>B. G. Johnston</td>
<td>University of Arizona</td>
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<tr>
<td>R. F. Joyce</td>
<td>William L. Travis &amp; Associates</td>
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<td>D. Krajcinovic</td>
<td>Argonne National Laboratory</td>
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<td>E. B. Lee</td>
<td>Student, UCLA</td>
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<td>G. C. Lee</td>
<td>State University of New York</td>
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<tr>
<td>S. Liapunov</td>
<td>New York University</td>
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<tr>
<td>R. B. Linderman</td>
<td>Bechtal Corporation</td>
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<tr>
<td>L. W. Lu</td>
<td>Lehigh University</td>
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J. W. Marsh  
R. M. Meith  
W. A. Milek  
R. M. Morford  
S. Negrea  
F. J. Palmer  
E. G. Paulet  
T. Pekoz  
C. W. Pinkham  
W. Porusk  
J. O. Robb  
L. P. Schrooten  
R. A. Scott  
J. Shiroma  
W. H. Smith  
J. Springfield  
E. O. Stephenson  
L. Tall  
J. B. Williams  
R. A. Williams  
D. Wiltse  
B. T. Yen  
W. W. Yu

American Inst. of Steel Construction  
Chevron Oil Company  
American Inst. of Steel Construction  
Joy Mfg., Western Precipitation Div.  
Joy Mfg., Western Precipitation Div.  
American Inst. of Steel Construction  
Federal Highway Administration  
Cornell University  
S. B. Barnes & Associates  
Structural Engineer  
City of L.A.-Building & Safety  
Airesearch - Garrett Corporation  
University of Michigan  
Airesearch - Garrett Corporation  
American Iron & Steel Institute  
Carruthers & Wallace  
American Iron & Steel Institute  
Lehigh University  
Jack D. Gillam & Associates  
Holmes & Narver, Inc.  
Structural Engineers Assoc. of So. Calif.  
Lehigh University  
University of Missouri-Rolla
List of Publications

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

American Petroleum Institute
CARBON MANGANESE STEEL PLATE FOR OFF-SHORE PLATFORM TUBULAR JOINTS,
Official Publication API Spec 2H, 1973

Bjorhovde, R., Brozzetti, J., Alpsten, G. A., Tall, L.
RESIDUAL STRESSES IN THICK WELDED PLATES, Welding Journal, 1972

Bjorhovde, R.
DETERMINISTIC AND PROBABILISTIC APPROACHES TO THE STRENGTH OF STEEL COLUMNS, Dissertation for Ph.D. degree, Lehigh University, 1972

Chen, W. F., Chen, C. H.
ANALYSIS OF CONCRETE-FILLED STEEL TUBULAR BEAM-COLUMNS,
IABSE, Volume 33-II, Zurich, 1973

Column Research Council
TASK GROUP 3 MEETING, Toronto, Ontario, 1970

Ellis, J. S.
ULTIMATE STRENGTH DESIGN OF HOLLOW STRUCTURAL STEEL COLUMNS SUBJECT TO BIAXIAL BENDING IN THREE-DIMENSIONAL NON-SWAY MULTI-STORY FRAMES, Final Report, 1972

Engineers Council for Professional Development
40TH ANNUAL REPORT, YEAR ENDING SEPT 1972, New York, New York, 1972

Gere, J. M.
BIBLIOGRAPHICAL STUDY OF S. P. TIMOSHENKO, (Unbound 49 pages), 1972

Hayashi, T., ed.
COLLECTED PAPERS OF TSU YOSHI HAYASHI, Hokuto Publishing Company, Tokyo, 1973

Hoglund, T.
BEHAVIOR AND STRENGTH OF THE WEB OF THIN PLATE I-GIRDERS WITH AND WITHOUT WEB HOLES, Stockholm, 1970

Johnston, B. G.
REMARKS ON STEEL COLUMNS, Paper presented at the open session of New Zealand and Australia Regional Conference of Planning & Design of Tall Buildings, 1973
Lee, G. C., Morrell, M. L., Ketter, R. L.
   DESIGN OF TAPERED MEMBERS, WRC Bulletin, 1972

Liapunov, S.
   ULTIMATE LOAD STUDIES OF PLANE MULTISTORY STEEL RIGID FRAMES,
   Ph.D. dissertation at New York University, Bronx, New York, 1973

Nagarajarao, N. R., Marek, P., Tall, L.

Neubauer, L. W.
   FULL-SIZE STUD TESTS CONFIRM SUPERIOR STRENGTH OF SQUARE-END WOOD
   COLUMNS, ASAE Paper No. 70-408

Tebedge, N., Chen, W. F., Tall, L.
   EXPERIMENTAL STUDIES ON COLUMN STRENGTH OF EUROPEAN HEAVY SHAPES,
   Fritz Engineering Laboratory Report No. 351.7, 1972

Tebedge, N., Chen, W. F., Tall, L.
   ON THE BEHAVIOUR OF HEAVY STEEL COLUMN, Fritz Engineering Labora-
   tory Report No. 337.33, 1972

Vogel, U.
   ULTIMATE LOAD TESTS OF FIXED BOS-SECTIONAL FLOOR COLUMNS SUBJECTED
   TO BENDING ABOUT ITS TWO PRINCIBLE AXIS, Die Ban Hecknik, Berlin,
   1970
CRC CHRONOLOGY (1971 - 1973)

25-26 Mar 71 - Council members took part in IABSE Colloquium on Design of Plate and Box Girders for Ultimate Strength, London

1 Oct 71 - CRC Secretaryship was transferred from F. Van der Woude to G. W. Schulz

24 Jan 72 - Executive Committee met at U.S. Steel Building, Pittsburgh

20-22 Mar 72 - Executive Committee Meeting, Technical Session & Annual Meeting in Chicago

1 Aug 72 - CRC Secretaryship was transferred from G. W. Schulz to B. G. Freeman

13-14 Nov 72 - Executive Committee Meeting in New York City

23-24 Nov 72 - CRC co-sponsored (with ECCS, IABSE & CRC Japan) Colloquium on Column Strength, Paris

3 Mar 73 - First "Chairman's Meeting" took place at The University of Arizona, Tucson

1-3 May 73 - Executive Committee Meeting, Technical Session & Annual Meeting at Los Angeles Hilton and Evening Session with Structural Engineers Association of Southern California, Los Angeles

14 Sep 73 - CRC was represented at meeting of ECCS Commission 8, Lisbon

3-4 Oct 73 - Executive Committee Meeting in Philadelphia

22-24 Oct 73 - CRC Co-sponsored Second Specialty Conference on Cold-Formed Steel Structures, Rolla
# Finance

## BALANCE at Beginning of Period

### INCOME

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### Contributions

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### Guide Royalties

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### Interest

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<thead>
<tr>
<th>Fiscal Year 10/1/72 - 9/30/73</th>
<th>Fiscal Year 10/73 - 9/74</th>
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<tr>
<td>$ 100.00</td>
<td>2.27</td>
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### TOTAL INCOME

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<tr>
<td>$23,275.00</td>
<td>$15,631.58</td>
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## EXPENDITURES

### Technical Services (Headquarters)

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<th>Fiscal Year 10/1/72 - 9/30/73</th>
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<tr>
<td>Director</td>
<td>1,500.00</td>
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<tr>
<td>CRC Secretary</td>
<td>1,616.50</td>
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<tr>
<td>Services, computer</td>
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<td>Supply, phone, mailing</td>
<td>699.04</td>
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<td>Travel</td>
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<td><strong>Total Tech. Services</strong></td>
<td><strong>$ 8,500.00</strong></td>
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<tr>
<td>TG 9 - Rice University, Houston</td>
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<td>Exploratory Research</td>
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<td><strong>$ 700.00</strong></td>
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### Guide

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<tr>
<td>Contract on Revision</td>
<td>4,800.00(e)</td>
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<tr>
<td>Travel and Expense</td>
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### Purchase of Technical Papers

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<td>$ 300.00</td>
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### United Engineering Trustees

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<tr>
<td>$ 100.00</td>
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### Annual Meeting and Proceedings

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<td>$ 500.00</td>
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### Contingencies

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## BALANCE at End of Period

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<td>$ 7,775.00</td>
<td>$12,028.06(h)</td>
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(a) Total Contributions
(b) Contributions from Aluminum Association
(c) Contributions from AISI
(d) Contributions from AISC
(e) Contributions from CISC
(f) Contributions from ICBO
(g) Contributions from Frank M. Masters
(h) Contributions from NSF
(i) Contributions from SEAONC
(j) Contributions from SEAOSC
(k) Contributions from SESA

---

[Note: The table and text are formatted to clearly show the financial information, including income sources and expenditures, for two fiscal years.]
**EXPLANATORY NOTES**

(a) Depositories (as of 9/30/72)

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**TOTAL DEPOSITS**

$9,703.83

(b) Including $1,500 as support to revision of CRC Guide.

(c) $6,000 to be applied for in support of 1973 Annual Meeting. It was thought that a $7,000 continuation would be requested for the CRC Guide, but, in fact, this proposal was not made.

(d)* $6,000 to be applied for in support of 1974 Annual Meeting plus $8,300, which is approximately 70% of total $11,900 grant applied for in continuation of CRC Guide revision (21 month period beginning July 1, 1973 or on approval of grant).

(e) $1,500 from AISC and $5,500 from the $7,000 NSF grant described above.

(f)* 70% of the $11,900 NSF grant is budgeted for the first 12 months of the 21 months.

(g) $4,700 for 1972-73 and $1,300 for 1973-74.

(h) Depositories (as of 9/30/73)

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**TOTAL DEPOSITS**

$12,028.06

* The $11,900 proposal was revised and now requests $6,895.00 over an 18-month period to start when approved.
Register

OFFICERS

Chairman       T. V. Galambos
Vice Chairman   G. Winter
Secretary       B. G. Freeman

EXECUTIVE COMMITTEE

T. V. Galambos (74)
G. Winter (74)
L. S. Beedle (Director)
C. Birnstiel (76)
L. A. Boston (74)
J. W. Clark (75)
T. Dembie (74)
J. L. Durkee *
G. F. Fox (75)
E. H. Gaylord **
J. A. Gilligan (74)
T. R. Higgins (Technical Consultant)
J. S. B. Iffland (76)
B. G. Johnston (75)
W. A. Milek, Jr. (76)

* Past Vice Chairman
** Past Chairman

STANDING & AD HOC COMMITTEES

A. Committee on the Guide to Design Criteria for Metal Compression

Members

E. H. Gaylord, Chmn.  S. J. Errera  A. L. Johnson
A. M. Amirkian  G. F. Fox  B. G. Johnston
A. J. Austin  T. V. Galambos  R. G. Kline
L. S. Beedle  J. A. Gilligan  C. F. Scheffey
K. P. Buchert  G. Haaijer  F. D. Sears
J. W. Clark  R. L. Haenel  D. Sfintesco
A. P. Cole  T. R. Higgins  J. Springfield
D. A. DaDeppo  J. S. Iffland  L. Tall
J. L. Durkee  S. H. Iyengar  G. Winter

B. Committee on Finance

L. S. Beedle, Chairman
T. V. Galambos
B. G. Freeman

C. Ad Hoc Committee on Research

T. V. Galambos, Chairman
Members of Executive Committee
TASK GROUPS

Task Group 1 - Centrally Loaded Columns

J. A. Gilligan, Chairman* R. R. Graham L. Plofker
L. S. Beedle D. H. Hall L. D. Sandvig
R. Bjorhovde A. F. Kirstein L. Tall
M. P. Gaus W. A. Milek, Jr. E. G. Paulet
J. E. Goldberg

Task Group 1 is concerned with the strength of centrally loaded columns as influenced by geometrical properties of the column cross section, mechanical properties of the material in the column and variables associated with the manufacture and fabrication of columns.

Task Group 3 - Ultimate Strength of Column With Biaxially Eccentric Load

J. Springfield, Chairman E. H. Gaylord* Z. Razzaq
T. M. Baseheart L. W. Lu B. C. Ringo
W. F. Chen C. Marsh G. Rupley
T. Dembie J. F. McDonough S. Vinnakota

This task group is concerned with investigating the behavior of columns subjected to biaxial bending, with a view of developing rational design procedures based on the ultimate strength of such members.

Task Group 4 - Frame Stability and Effective Column Length

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

The purpose of this task group is to investigate the stability of building frames, including effective column length aspects. It will work in close contact with Task Groups 10 and 15.

Task Group 6 - Test Methods for Compression Members

L. Tall, Chairman J. W. Clark B. G. Johnston
C. K. Yu, Vice Chairman E. W. Gradt B. M. McNamee
L. S. Beedle R. A. Hechtman H. H. Tung
C. Birnstiel* T. R. Higgins

This task group is concerned with the development of technical memoranda on experimental methods and techniques of testing structural members subject to buckling, including the analysis of the data of the test. It is also the purpose of the group to organize and conduct technical sessions and symposia on test methods to facilitate exchange of information on new testing procedures.

* Executive Committee Contact Member
Task Group 7 - Tapered Members (Joint Task Group with WRC)

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

This task group, a joint task group with Welding Research Council, is concerned with research leading to the development of design procedures for tapered structural members and frames made of such members.

Task Group 8 - Dynamic Instability

D. A. DaDeppo, Chairman  I. K. McIvor
B. G. Johnston*  J. C. Simonis
D. Krajcinovic

The goal of the work of this task group is to make design recommendations regarding the load carrying capacity of columns and other compression members subjected to dynamic loading. To this end, the available information in field will be correlated and the areas in which further research effort is required will be identified.

Task Group 10 - Laterally Unsupported Restrained Beam-Columns

T. V. Galambos, Chairman*  G. C. Lee  W. A. Milek, Jr.
J. A. Gilligan  L. W. Lu  M. Ojalvo

This task group is concerned with the study of design methods for wide-flange beam-columns subjected to strong axis bending and unbraced against out-of-plane deformations. The study consists of experimental and analytical investigations of the behavior of beam-and-column assemblages where the columns are laterally unrestrained. The final purpose is the development of improved design rules for such members.

Task Group 11 - European Column Studies

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

The purpose of this task group is to examine the strength of centrally loaded steel columns with particular reference to a statistical approach to tests and interpretation of data. Through collaboration with Subcommittee 8 of the European Convention of Constructional Steel Work, the task group will provide guidance to experimental and theoretical studies in the United States of the heavier European rolled shapes.

* Executive Committee Contact Member
Task Group 12 - Mechanical Properties of Steel in Inelastic Range

A. L. Johnson, Chairman  J. J. Healey  M. Shinozuka
G. A. Alpsten  L. W. Lu  W. J. Wilkes
G. F. Fox*

The purpose of the task group is to obtain data on the mechanical properties of steel in the inelastic range of particular importance to stability solutions. Among other things this would include determination of the average value and variation of the following: yield stress level, strain hardening modulus, magnitude of strain at initial strain hardening, and, for materials without a well defined yield point, yield strength, tangent modulus and secant modulus.

Task Group 13 - Thin-Walled Metal Construction

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

The purpose of this task group is to digest the literature on thin-walled metal construction, as it relates to stability, and to draft a chapter for the third edition of the CRC Guide. Materials of interest include carbon steels, alloy steels, stainless steels and aluminum alloys. The effects of various manufacturing and fabrication processes shall be considered.

Task Group 14 - Horizontally Curved Girders

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

The purpose of this task group is to explore the stability problems which occur in horizontally curved girders, both during erection and in the completed structure, the effects of rolling and fabrication practice on these problems, and criteria for adequate bracing.

Task Group 15 - Laterally Unsupported Beams

R. L. Haenel, Chairman  T. V. Galambos*  A. J. Hartmann
P. F. Adams  W. Hansell  J. A. Yura

The purpose of this task group is to study the stability of laterally unsupported beams and the bracing requirements for such beams in both the elastic and inelastic ranges with emphasis on beams in framed structures. The research should lead to a design procedure for such members.

* Executive Committee Contact Member
Task Group 16 - Plate and Box Girders

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

This task group is concerned with the stability and strength of plate girders. A considerable amount of work on the behavior and load carrying capacity of plate girders is underway in this and other countries. The purposes of the task group are to facilitate exchange of information among these investigators, to encourage preparation of reports relevant to design specifications, and to assist in revising the chapter on plate girders in the CRC Guide.

Task Group 17 - Stability of Shell-Like Structures

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

The purpose of this task group is to prepare a chapter for the CRC Guide, summarizing design information on the stability of civil engineering shell-type structures.

Task Group 18 - Tubular Members

A. L. Johnson, Chairman  D. W. Fowler  R. M. Meith
M. D. Bernstein  R. R. Graham  C. D. Miller
L. A. Boston*  J. R. Lloyd  R. L. Rolf
A. Chajes  J. N. Macadam  D. R. Sherman
N. W. Edwards  P. W. Marshall  S. Stadnyckyj
G. S. Fan

The purpose of this task group is to prepare a chapter for the Guide. This chapter will summarize design information on cylindrical tubes and shells. The task group will also study other aspects of the stability of tubular members of various cross-sectional shapes.

Task Group 20 - Composite Members

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

The purpose of this task group is to study design methods, vis-a-vis research results, examine existing codes, and survey industry to determine the relative importance of various types of composite columns.

* Executive Committee Contact Member
TASK REPORTERS

Task Reporter 11 - Stability of Aluminum Structural Members

J. W. Clark, Aluminum Company of America

Task Reporter 13 - Local Inelastic Buckling

L. W. Lu, Lehigh University

Task Reporter 14 - Fire Effects on Structural Stability

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

Task Reporter 15 - Curved Compression Members

W. J. Austin, Rice University

Task Reporter 16 - Stiffened Plate Structures

R. G. Kline - U. S. Steel Corporation
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<th>Organization</th>
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<td>C. E. Baird, Secretary</td>
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<td>C. M. Duke, President</td>
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<td>J. A. Zecca, Secretary</td>
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<td>A. Dailey</td>
<td>D. Sfintesco,</td>
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<td>Tech. Sec. General</td>
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<tr>
<td>General Services Administration</td>
<td>J. E. Bihr</td>
<td>E. B. Evers,</td>
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<tr>
<td>Institution of Engineers, Australia</td>
<td>M. Stein</td>
<td>Admin. Sec. General</td>
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<td>International Conference of Building Officials</td>
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<td>A. F. Sampson,</td>
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<td></td>
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<td>Administrator</td>
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<td>International Nickel Company, Inc.</td>
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<td>R. D. Henderson,</td>
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<td>J. E. Bihr,</td>
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<td>National Aeronautics &amp; Space Administration</td>
<td>F. A. Petersen</td>
<td>Technical Director</td>
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<td>W. S. Mounce</td>
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By-Laws*

PURPOSES

The general purposes of the Column Research Council shall be:

1. To maintain a forum where problems relating to the design and behavior of columns and other compression elements in metal structures can be presented for evaluation and pertinent structural research problems proposed for investigation.

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

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

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

5. To study the application of the results of research projects to the design of compression elements; to develop comprehensive and consistent design formulas and rules, and to promote their adoption by specification-writing bodies.

*Revised: August 21, 1947; October 1, 1948; November 1, 1949; August 15, 1951; May 20, 1955; October 1, 1960; May 7, 1962; May 21, 1965; and May 31, 1968.
Membership

The membership of the Council shall consist of the Representatives of the Participating Organizations and a variable number of Members-at-Large.

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

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

Every three years the Chairman of the Council shall check with each Member-at-Large to determine whether he wishes to continue his membership.

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

Meetings

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

Fiscal Year

The fiscal year shall begin on October 1.

Duties

1. To establish policies and rules.

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

3. To maintain and operate a central office for the administration of the work of the Council, and for the maintenance of its records.
4. To prepare an annual budget.

5. To issue annual reports.

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

**Officers**

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

2. The terms of office of the Chairman and Vice Chairman shall begin on October 1st and shall continue for 3 years. They shall be eligible for immediate re-election for only one term of one year. In the event of a vacancy in the office of Chairman or Vice Chairman, a successor shall be appointed by the Executive Committee to serve for the remainder of the unexpired term.

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

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

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

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

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

Executive Committee

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

2. The Executive Committee shall transact the business of the Council and shall have the following specific responsibilities and duties:
   (a) To direct financial and business management for the Council, including the preparation of a tentative annual budget.
   (b) To review and approve proposed research projects and Contracts.
   (c) To appoint nominating committee.
   (d) To appoint chairman of committees and task groups, and approve committee and task group members.
   (e) To review reports and manuscripts.
   (f) To advise Council on proposed research projects.
To prepare program for Council meeting.
To correlate and give general supervision to research projects.
To refer inquiries relating to design practice to the Committee on Recommended Practice for definition, evaluation, and suggestions for task group assignment.

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

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

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

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

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

Contracts

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

Standing and Special Committees

1. The standing committees shall be a Committee on Finance and a Committee on the "Guide to Design Criteria for Metal Compression Members". There shall be such special committees as may be approved by the Council.
2. Standing and special committees and their chairmen, shall be appointed by, and responsible to, the Executive Committee. They shall be named at a regular annual meeting of the Council, shall take office upon appointment, shall serve for three years, and shall be eligible for immediate reappointment. Vacancies shall be filled in the same manner as regular appointments except that such appointees will complete the term of office vacated.

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

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

Research Committees and Task Groups

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

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

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

4. The duties of a research committee or task group shall be:
   (a) To review proposed research projects within its field, and to render opinions as to their suitability;
   (b) To make recommendations as to needed research in its field;
   (c) To give active guidance to research programs within its field, in which connection research committees or task groups are empowered to change details of programs within budget limitations;
   (d) To make recommendations as to the time when a project within its field should be temporarily discontinued, or terminated;
   (e) At the request of the Executive Committee to prepare summary reports covering results of research projects and/or existing knowledge on specific topics.
5. Each project handled by a research committee or task group shall be of definite scope and objective.

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

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

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

Revision of By-Laws

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

I. OUTLINE OF ROUTE OF A RESEARCH PROJECT FOR CONSIDERATION OF THE COLUMN RESEARCH COUNCIL

Projects are to be considered under three classifications:

(1) Projects originating within the Column Research Council.

(2) Those originating outside the Column Research Council or resulting from work at some institution and pertaining to general program of study approved by Column Research Council.

(3) Extensions of existing CRC 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.

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 finding 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.
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 Column 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 Column Research Council.

2. Subject to approval of the Column Research Council, the Publication 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 Column Research Council should make their own arrangements for publications or 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.