STRUCTURAL STABILITY RESEARCH COUNCIL

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"SSRC - Link Between Research and Practice"

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TABLE OF CONTENTS

FOREWORD

To recognize its 50th Anniversary, the SSRC has organized a special conference for 20-22 June 1994 with the theme, *"SSRC - Link Between Research and Practice"*. It is appropriate to hold this conference at Lehigh University, which has been the SSRC Headquarters since 1966. The meeting is intended to review the complete scope of SSRC activities and to use the expertise of the entire SSRC membership to develop a vision of stability related research and design in the future. These Proceedings contain the viewpoints of a group of invited speakers on topics that are considered by the SSRC Task Groups and Task Reporters. A volume of post-conference Proceedings will contain the current research papers presented at Task Group meetings, with summary statements from the Workshop sessions at the end of the conference, which will reflect the input from all participants, recorded in a special report.

When the Column Research Council (CRC) was organized in 1944, one of the primary motivations was the need to establish uniformity in column design on the part of all building authorities in the U.S. The CRC was successful in this effort, and in the succeeding years the scope of its activity expanded to include many other stability related problems. As a result of the expanded scope, the name of the organization was changed in 1976 to the Structural Stability Research Council (SSRC) to better reflect its activities. Another change that has evolved over the years, is the international character of the organization. All continents of the world in which the stability of metal structures is both examined and is a feature of design are represented in the current membership.

Considerable change and expansion in both research and design practice has taken place since the inception of the Council and particularly in the last two decades. Much of this has been piecemeal, so that it is not easy to determine overall trends. Problems in applying existing stability design criteria have been created by the use of higher strength materials used in more open framed structures, unique architectural layout, analytical computing capabilities and an information explosion in stability related publications. Present criteria emphasize the design of individual members, but it has become increasingly evident that the member must be considered as part of the total structure. One of the major current difficulties is the trend for increasingly diverse methods of design for stability rather than unification. Specification writing groups have different philosophies of design and are exposed to different literature and data bases in various parts of the world. With an increasingly global construction environment, difficulties are encountered when the design/construction team is international. The structural engineering profession will benefit from the meaningful discussion and attempt to focus the future of stability related research and design that is the intent of this conference.

Numerous people could be recognized for the influence they have had in bringing this conference into fruition. We could go back to Jonathan Jones of Bethlehem Steel and Shortridge Hardesty, the first Chairman of the Council, for their vision in creating an organization devoted to considering structural stability problems. There is a long list of other leaders over the years who have left their imprint, such as Lynn Beedle who served as Director for 23 years and was a strong influence in bringing about the international character. However, directly related to the conference, a special thanks is due to Clarence Miller and the Program Committee as well as the authors who have given their time and effort in the preparation of the papers for these and the post-conference proceedings. Special recognition is also due to the Headquarters staff for the extra effort they had to put forth in the organizational details. For this we thank: Jim Ricles, Director; Lesleigh Federinic, Administrative Assistant; and Diana Walsh, Secretarial Assistant. Finally, we acknowledge and express appreciation to the co-sponsoring organizations:

The Aluminum Association
ATLSS Engineering Research Center
Aluminum Company of America
American Institute of Steel Construction
American Society of Civil Engineers
Federal Highway Administration
National Science Foundation

Hopefully these Proceedings will become an important part of structural engineering literature and will be a force in the development of future stability related research and design criteria.

D.R. Sherman

Donald R. Sherman
Chairman

CIVIL INFRASTRUCTURE SYSTEMS & STRUCTURAL STABILITY RESEARCH

by
K. P. Chong and S. C. Liu
National Science Foundation

Summary

Civil infrastructure Systems (CIS) are those structures and lifelines by which society transports goods, provides clean air and water, controls disease, and conducts commerce. CIS span all these civil systems - their operations, management, and policies which interact with societal demand and the physical world. In the last Century the United States has invested heavily in building its infrastructure but it has rapidly degraded due to neglect, misuse and excessive demand. Many studies conclude that the collapsing infrastructure in U.S. contributes significantly to the current decline in the nation's productivity and its increasing deficit. Intelligent renewal is required in using the limited resources available to renew and retrofit the two trillion dollars of infrastructure in the most cost-effective manner. A research initiative is therefore proposed to underpin this effort. This initiative builds on NSF's past and current research efforts (which have focused on component performance) and will address the seemingly intractable issues of system behavior, deterioration, assessment, renewal, institutional effectiveness and productivity. Structural stability research is an important component of the CIS. Examples of NSF grants on stability research are discussed.

INTRODUCTION

The cost of construction of physical facilities, including structural and geotechnical engineering facilities, constituted about 7.3% of the 1991 GNP employing six millon persons, thus qualifying the construction industry to be one of the largest U.S. industries. The rise and fall of a civilization is ultimately linked to its productivity, global competitiveness , and international cooperation. These in turn are dependent on the life-cycle of its infrastructure which is the underlying, almost invisible base on which the wealth and quality of life a society depend. As the infrastructure deteriorates, civilization unravels and eventually collapse being paralyzed by its inability to transport food, provide clean air and water, control disease, and conduct commerce. In the last century, the United States has invested heavily in building its infrastructure from canals to fiber optic systems, from fresh water to interstate highways and rapid mass transit systems. But the system has rapidly degraded due to age, neglect, misuse and excessive demand. In the U.S. for example, it is reported that 40% of the bridges are structurally deficient and are in need of repair and rehabilitation. More than half of the existing school buildings are over 50 year old and are in poor condition. Corrosion, a common deteriorating physical phenomenon of CIS, cost this country over $250 billion annually (4% of the nation's GNP). The 1992 occurrences of the Chicago flood and Hurricane Andrew, and the 1993 Mississippi Flood illustrated the fragility of these systems and the staggering losses their failures can incur. Many studies conclude that the current decline in the nation's productivity and its increasing deficit are largely the result of a collapsing infrastructure. There is therefore an urgent need to "rebuild America" but the cost is prohibitive and the added burden of the particular debt could cause the total collapse of the US economy.

Instead, intelligent renewal and retrofit is required which will use the limited resources that are available in the most cost-effective manner. The key to this strategy is a research initiative which addresses to parallel issues of implementing existing knowledge and developing appropriate new scientific and engineering knowledge base.

The nation's infrastructure comprises several hundred large complex engineering systems which interact with each other in the ways that are not well understood. Past research has concentrated on the performance and operation of individual components within separate systems. The result has been incremental improvements in selected areas but system performance has not been measurably affected. This "band-aid" approach has been largely ineffective in the overall effort to arrest the decline in the infrastructure.

A new strategy is required that approaches the issue from a systems perspective. In fact the real challenge today is to study the optimal performance of systems rather than individual components. It is time to build on the successful NSF research programs of the past (which have been component-focused) and to address the seemingly intractable issues of system behavior, deterioration, assessment, renewal, institutional effectiveness and productivity.

This initiative therefore has the following goals:

* To stimulate infrastructure renewal (including retrofit) in the U.S. and regain international competitiveness through research excellence and accelerated knowledge transfer.

* To mitigate the barriers to knowledge transfer through improved insight into the information utilization process and the development of innovative strategies that overcome technical, institutional, social and legal impediments.

* To contribute to the increased productivity and industrial competitiveness of the nation by developing new knowledge which will permit the renewal of the U.S. infrastructure such that it will last longer, reach further and provide for increased capacity, while being reliable, safe, cost-effective and environmentally sensitive.

To achieve these goals, this initiative has the following priorities:

* Perform knowledge utilization research through multidisciplinary programs involving engineering, social and scientific disciplines.

* Initiate research programs to address the intellectual challenges associated with implementing existing knowledge and the development of new scientific and engineering

knowledge.

* Develop industry partnerships through proof-of-concept research; support education and research at universities; develop (including re-train) the human resources necessary to renew and maintain current and future infrastructure systems.

BACKGROUND

Civil infrastructure is the physical structure or underlying foundation to a society's wealth and quality of life. Largely hidden and frequently taken for granted, it includes the basic installations and facilities necessary for modern existence and growth. Civil infrastructure therefore includes certain public buildings (such as schools, post offices, police stations, municipal and government offices) and almost all the lifelines. Lifelines are those installations and facilities that transport people, distribute goods and transmit energy and information from place to place. Examples of the major lifelines include electric power systems, gas and liquid fuel systems, telecommunication systems, transportation systems, water and sewage systems. Civil infrastructure systems are usually composed of nodes, components and interconnecting links. For example intersections, bridge and roads make up a highway system. Almost all such systems have been designed component-by-component even though they operate as a system.

NSF has traditionally supported research in infrastructure-related issues, but these efforts have concentrated on component rather than system performance. Current major NSF programs in this area include: Structures, Geomechanics and Building Systems; Mechanics and Materials; Tribology, Earthquake Hazard Mitigation; Natural and Man-made Hazard Mitigation; Fluid, Particulate and Hydraulic Systems; Environmental and Ocean Systems; Electrical and Communications Systems; Social, Behavioral and Economic Sciences, and Computer and Information Science and Engineering; Physics, Chemistry, Biology, Geosciences, Education and other programs.

PROGRAM DESCRIPTION

The proposed Initiative has two basic components: a Research Program and a Knowledge Transfer Program. The Research Program is in turn composed of four critical Elements which directly address the infrastructure problem. These Elements are:

Deterioration Science: A fundamental issue in understanding why constructed facilities decay is to understand better the science of deterioration. A major component of this research will be materials science and mechanics. Other components are studies in failure processes; stability; progressive collapse; risk and reliability; materials processing, fabrication, manufacturing and assembly; corrosion, fatigue and environmental hazards; performance criteria; extension of service life; strength and durability.

Assessment Technologies: A major problem in repairing and upgrading infrastructure is an inability to assess accurately the state of health of a constructed facility. Current methods for assessing constructed facilities are relatively primitive and unreliable, prompting conservative, often costly decisions. Research in this area will focus on nondestructive evaluation, smart materials, damage processes, advanced instrumentation, evaluation of service under extreme events; acceptable risk; interdependence of infrastructure systems; geographical information systems; and social and economic effects.

Renewal Engineering: Constructed facilities - the nation's largest tangible resource - include facilities for transportation, energy, waste collection and treatment, water supply and protection, and for living, working, playing and performing functions of education and government. These physical underpinnings of society need renewal, retrofit, modification, and upgrading, and research will emphasize new design and construction methods. Other important studies include the use of new or modified materials, trenchless technology (e.g., micro-tunneling), batch manufacturing techniques in construction, simulation, innovative repair and modified construction techniques suited to robotics applications. Innovative electrical and communication systems could play a role in intelligent highways. Other research topics include performance criteria and repair strategies; demolition, disposal, and recycling; preservation of national resources; information theory, expert systems, and artificial intelligence; and integrating structural design, processing, and fabrication. Recent Northridge Earthquake demonstrated the effectiveness of retrofitting transportation infrastructure in mitigating damages.

Institutional Effectiveness and Productivity: Currently several significant barriers exist which impede the implementation of research results into practice. The CIS initiative will include a research program to identify the public policy issues and the social, legal and financial constraints which inhibit implementation of new concepts. The program will also promote research to develop innovation methods for linking users and researchers for an effective implementation of new ideas into practice and also to provide a mechanism for a constructive feedback to keep the research program aware of the critical research issues.

Each research element is seen to comprise several subelements selected from various cross-cutting research programs such as: materials science, mechanics, social sciences, structural and geotechnical engineering, fluid mechanics and water resources, environmental engineering, computer and information sciences. Common themes within these subelements include the basic functioning of an advanced society, mitigation of hazards and risk engineering. The proposed initiative is consistent with the traditional role of NSF and seeks excellence, quality and innovation in the development and application of knowledge to infrastructure systems.

The Knowledge Transfer Program is intended to use both traditional and nontraditional methods for accelerating the transfer of research results into practice. Because of the necessity to emphasize system integration and performance of very large scale systems, creative approaches must be developed for sustained knowledge transfer efforts. Examples of nontraditional techniques include the placement of end-users on research teams, the co-sponsorship of proof-of-concept research and demonstration projects, and the development of electronic information services and expert systems. This program will provide a major link between researchers and the practicing community. It will therefore also provide a mechanism for feedback from the community to the researchers and serve to keep the Research Program aware of the most critical issues facing the nation.

It is interesting to note that, shortly after the completion of the NSF development of CIS initiative, President Clinton said in his message on America's technology for economic growth, called for investment in establishing an integrated research program designed to enhance the performance and longevity of the nation's existing infrastructure. Specifically it was mentioned: "...this program would systematically address issues of assessment technology and renewal engineering."

STRUCTURAL STABILITY RESEARCH

Structural stability research is an important component of the Civil Infrastructure Systems initiative since it is closely related to assessment technologies, renewal engineering and especially deterioration science. Over the last couple of decades considerable research has been made in the U. S. and abroad on various aspects of structural stability research. The following examples are some recent and ongoing projects funded by NSF.

* Dynamic Buckle Propagation in Submarine Pipes; J. Carney, Vanderbilt University.

* Second Order Inelastic Analysis of Frames; W. F. Chen, Purdue University.

* Local-global Analysis for Plastic Collapse of Shells; P. Gould, Washington University.

* Space Truss Structural Integrity; E. Murtha-Smith, University of Connecticut.

* Transverse Material Properties Effects on Pultruded Fiber Rienforced Plastic Structures; L. Bank, Catholic University of America.

* Earthquake Stability Problems in Eastern North America; L. Beedle, G. Fox, Lehigh University.

* Corrugated Webs and Panels; M. Elgaaly, Drexel University.

* Ultimate Testing of a Shell Bridge Model; F. Fanous, F. Klaiber, Iowa State University.

* Dynamic Stability of Cable Domes; D. Gasparini, P. Perdikaris, Case Western University.

* Lateral Confinement of Concrete Using High-Strength Fiber Reinforcement; A. Nanni, Pennsylvania State University.

* Post Buckling Behavior and Imperfection Sensitivity of Space Trusses with Multiple Eigenmodes; R. Peek, N. Triantafyllidis, University of Michigan-Ann Arbor.

* Dynamic Behavior of Inflatable Dams; R. Plaut, J. Reddy, D. Walker, Virginia Polytechnic Institute and State University.

* Residual Strength and Repair of Damaged Offshore Structural Steel Platform Members; J.M. Ricles (PYI), Lehigh University.

* Plastic Instability Phenomena with Aspects of Chaos of Fixed-Edge Structures Subjected to Short Pulse Loading; P. Symonds, H. Kolsky, Brown University.

* Detailed Three-Dimensional Simulation of Steel Frame Subassembledges; D. White, Purdue University.

* Dynamic Stability of Structural Degradation; K. Willam, University of Colorado at Boulder.

* Structural Instability Behavior and Material Failure of J-Stiffened Composite Panels; H.Y. Yeh, V. Chen, California State University-Long Beach Foundation.

* NSF/DITAC Workshop on Cold-Formed Steel Structures; W.W. Yu, University of Missouri-Rolla.

BENEFIT

The U.S. has passed through the up-cycle of civilization-building. The spread of railroads, canals, highways, electric power and communication systems across the country served as underpinnings for rapid development and industrialization. The resulting infrastructure served both as the lubricant that sped development of American civilization and as the glue that bound the regions and the citizens of the country together. Now this infrastructure is decaying because of age, deterioration, misuse, and lack of repair, or because it was not designed for current demands.

The total U.S. investment in this infrastructure is estimate to be about $2 trillion. By comparison this initiative is very modest in view of the size, complexity and urgency of the infrastructure renewal problem. If the cost of renewing the infrastructure is reduced by only 1% as a result of this Initiative, this research program will have paid for itself 100 times over.

CONCLUDING REMARKS

Academic and industrial research over the last decade has spurred various advances in new materials, structural systems, structural stability, automated construction, ground enhancement, prefabricated assemblies, electrokinetic geotechnolgy, corrosion inhibition, electro-optical communication, understanding of public decisions, management, location and siting, and public finance, and others Yet barriers still exist to transferring this basic knowledge into civil infrastructure practices. Today, it generally takes 5 to 20 years to move such knowledge from research institutions to the marketplace.

Basic research and the developing of research results, have great potential for prolonging the life and enhancing the capacity of our civil infrastructure systems and for improving our economic productivity and quality of life as well.

This paper reflects the personal views of the authors, not necessarily those of the National Science Foundation.

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Moehle, J. P., ed, "Preliminary Report on the January 17, 1994 Northridge Earthquake" Rept. No.UCB/EERC-94/01, Univ. of California, Berkeley, Jan. 1994.

NSF Workshop on "Public Infrastructure Research Agenda for the Social, Behavioral and Economic Sciences", Dennis Epple (chm), 4/93, Washington, DC.

"Technology for America's Economic Growth, A New Direction to Build Economic Strength", by President Clinton and Vice President Gore, February 22, 1993.

FIFTY YEARS OF RESEARCH ON STRUCTURAL STABILITY
WHAT HAS IT ACCOMPLISHED?

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SUMMARY

The purpose of this paper is to place the other contributions in this volume in an historical perspective by examining the state of structural stability from 1944, when the Council (first " Column Research Council (CRC)", then "Structural Stability Research Council (SSRC)") was founded, to the present time (1994). For illustrative purposes the changes in the research philosophy on the behavior of individual structural members (columns, beams and beam-columns) and structural systems will be discussed.

1. INTRODUCTION

All structural failures are due to some form of instability. The main purpose of the Council is to perform research on instability due to compressive forces acting on metal (steel or aluminum) structures for the benefit of structural design engineers and writers of structural design specifications. As Professor Bruce Johnston, one of the founders of the Council and its chronicler (Johnston, 1981), fondly used to tell (CRC, 1954), research on the stability of structures started in early biblical times when Samson applied a lateral load to two heavily loaded columns. To quote from Judges 16:29-30: "And Samson grasped the two middle pillars upon which the house rested and he leaned his weight upon them ... Then he bowed with all his might; and the house fell upon the lords and upon all the people that were in it." Column research was directed to more peaceful purposes in Greek antiquity, as described by the Roman engineer Vitruvius and recounted by Professor Nicholas Hoff, another early member of the Council (Hoff, 1954). Vitruvius classified columns as being proportioned according to the dimensions of the human leg, distinguishing three column strength categories: The column modeled by the leg of a mature male (Doric), of a mature female (Ionic) and of a young girl (Corinthian). For three thousand years countless columns were constructed by trial and error and by knowledge being passed down from father to son in trade guilds and construction organizations. Understanding of strength was mainly empirical. Leonard Euler (1759) and Engesser and Considere in the last decade of the 19-th Century provided some excellent theoretical insights into, respectively, elastic and inelastic buckling, but by the late 1930-s there really was not much change: columns were being still designed by empirical rules using either the Rankine-Gordon Formula

$$\sigma_{cr} = \frac{\sigma_y}{1 + C_1 \left(L/r\right)^2}$$

the Tetmaier straight line formula

$$\sigma_{cr} = \sigma_y - C_2\left(L/r\right)$$

or the Johnson parabolic formula

$$\sigma_{cr} = \sigma_y - C_3\left(L/r\right)^2$$

where the coefficients C_1, C_2 and C_3 were empirically determined from tests of specific types of columns. Contemporary specifications abounded in examples of such column formulas.

This state of affairs was judged to be highly unsatisfactory to Jonathan Jones, Chief engineer of the Fabricated steel Construction Division of the Bethlehem steel Corporation, who wrote in 1941 (Johnston, 1981):

> "... I urged and do urge that it is a national necessity that as many as possible of the bodies who are interested in writing formulas for steel columns get together in some kind of a central group and carry on the research and analyze the results in a way that is satisfactory to all."

Thus was born the Column Research Council from the desperate cry of a practicing engineer calling for solution to a national problem: how to design steel columns rationally. This paper will high-light some of the research outcomes in the past 50 years since 1944. These results are, indeed, "satisfactory to all".

2. THE CENTRALLY LOADED COLUMN

The history of the centrally loaded column is discussed by Professor Johnston (1983). Since 1944 much of this history was made by researchers associated with the Council. Following is a brief recounting of the evolution of the column design philosophy during the past 50 years. When the Council was formed there was already a considerable body of knowledge on the subject of centrally loaded columns, but little of that was applied by design engineers. The modes of failure and the factors influencing column strength were well understood (Beedle,1951). The proceedings of the Fourth Technical Session of the Council (CRC,1954), appropriately named "The Philosophy of Column Design", include the genesis of most of the developments of the next 40 years. What were the major intellectual problems of the early days? Following are two of the issues that Jonathan Jones, Lynn Beedle, Robert Ketter, Thomas Kavanagh, Harry Hill, John Clark, Ted Higgins and others struggled with in 1954:

a) How complex should a design specification be? After a lot of arguments for "extreme simplicity" versus enough complexity to require at least an MS degree to design steel structures, the participants came up with a compromise which still dominates the Council's thinking today: present the results of research in the most helpful manner for use by designers and specification writers, and to let the latter concern themselves with levels of safety and degrees of simplification.

<u>b) What should be the "basic column strength"?</u> The discussions were strongly affected by the recent (1945) resolution of the "column paradox", (i.e. which is the "true" strength of a centrally loaded column failing in the inelastic range, the *tangent modulus* or the *reduced modulus* strength) by F. R. Shanley (Shanley,1947). As a result the Council came down strongly on the side of the tangent modulus approach. Professor Johnston states that (CRC,1954):

> " Out of the confusion emerges the clear fact that the tangent modulus prediction of the column strength is the logical common denominator by which such strength should be predicted and all other factors tending to lower the strength of an actual column may be thought of as doing so with respect to the tangent modulus estimate as an upper bound"

This statement was a reaffirmation of CRC Technical Memorandum No.1 (issued May, 1952). The tangent modulus theory takes very good account of non-linear material behavior, the shape of the cross-section, the residual stresses and the end restraints, but as to the inconvenient "accidental eccentricities, curvature or lateral loads.... it would seem possible to allow for these as causing an unavoidable scatterband in actual strength with lower limits of such a scatterband to be safely out of range of design loads by virtue of a suitable factor of safety." (Johnston, in CRC, 1954).

Having made a strong argument against the "secant formula" as being "totally empirical", the Council fell into the trap of deemphasizing a crucial portion of the real column strength, namely, the initial imperfections.

However, the tangent modulus theory was a rational solution for a part of the problem, and its adoption spurred on a lot of very useful activity:

- Residual stresses were carefully measured in many types of rolled and welded shapes by researchers at Lehigh university (Huber, Fujita, Tall, Estuar, Nitta and many others) and a solid understanding residual stress effects on column strength emerged.
- The Council standardized the testing of compression coupons, stub columns and columns (Technical Memoranda 2, 3 and 4), as well as the measurement of residual stresses (TM No. 6) and tension testing (TM No.7).
- Many column were carefully tested at Lehigh University, with the relevant data well documented, so that a reliable data base was generated against which future theories could be calibrated.
- A simple "average" column formula was proposed which has served many structural specifications for 40 years.
- The study of the tangent modulus behavior showed that different types of cross sections and manufacturing processes resulted in different strength curves (Beedle, 1960), and out of this came the idea of the *multiple column curve*.

While the tangent modulus concept presented a rational framework for defining the behavior of the *ideal* system, researchers began to turn away from such perfect structures and began to work on structures which were not perfect. By the 1960-s computers had advanced to the point where column with initial crookedness and/or eccentric loads could be efficiently analyzed. Research thus began to repudiate Technical Memorandum No.1. Two key studies, those by Batterman and Johnston at the University of Michigan (Batterman,1967) and Bjorhovde at Lehigh University (Bjorhovde, 1972) demonstrated clearly that full knowledge of the material, cross-sectional and geometrical imperfection parameters,when used in proper non-linear analysis, can predict column strength very accurately, initial imperfections indeed do appreciably reduce the strength of a metal column.

These two seminal studies led eventually to the evolution of the SSRC and the European column curves which are used in many modern structural specifications of the mid-90-s.

Research thus made the Council's TM No.1 totally inadequate by the early 1970-s and therefore TM No.5 was issued in 1981 to come up with a philosophy which was in tune with actual developments. The essence of this memorandum can be summarized by the following quotation from it:

> "Maximum strength, determined by evaluation of those effects that influence significantly the maximum load resisting capacity of a frame, member or element, is the proper basis for the establishment of strength criteria."

This statement has stood the test of the last 15 years, and is likely to continue to be the research philosophy of the Council.

On the subject area of centrally loaded columns the current situation is that the basic intellectual problems have been conceptually solved, and that many of the technical problems have been conquered, including eccentrically loaded initially crooked end-restrained columns, thin walled columns failing by combined lateral-torsional and local buckling, and many others. The theories, the computational methodologies and tools and the test methods exist for the solution of any new type of material combination, cross-sectional shape, manufacturing process and end condition. The new areas of research which suggest themselves are in the use of this advanced methodology for the solution of columns made from wood or from modern composite materials. Another area of research is a more routine but still very necessary task: expanding the experimental and computed data for use in reliability studies of many types of columns, especially concrete-metal composite members.

Thanks mainly to the SSRC-inspired research during the past half century, most aspects of the centrally loaded column problem are for all practical purposes solved. The research has developed the necessary analytical and experimental information so that specification writing groups can select column strength curves of a great enough variety and reliability for most column types used in a design office.

3. THE BEAM-COLUMN

Much of what has been discussed in the previous section of this paper applies equally well to the developments in beams and beam-columns: In 1944 when the Council was founded, there was sufficient theoretical work available for an understanding of the elastic lateral-torsional buckling of beams and of the elastic limit solutions for in-plane beam-columns (Timoshenko, 1936). There were also studies of the inelastic ultimate strength of beam-columns (von Karman, 1910, Chwalla, 1934, Jezek, 1936). This information did not, however, affect the design of steel beams and beam-columns in the US specifications in 1944. Lateral-torsional buckling was checked by a formula which was wrong, and the beam-column interaction equation in use was unacceptably unconservative in the practical slenderness ratio range (CRC, 1954). The situation for these types of members was worse than for centrally loaded columns because there was scarcely any experimental base for buttressing the empirical relations used in design. A 1947 survey paper "Steel Columns, a Survey and Appraisal of Past Works" (Jakkula and Stephenson, 1947) lists only two references reporting tests on beam-columns.

These shortcomings of beam and beam-column design were extensively and brilliantly discussed in "The Philosophy of Column Design" (CRC, 1954) where substantive recommendations were made for the rationalization of beam and beam-column design formulas. In the succeeding 40 years both beam strength and beam-column strength has been extensively studied both under the guidance of the Council and in many other organizations so that in 1994 it can be said that there is hardly a problem which has not been examined many times from different theoretical, numerical and experimental aspects. The beam research is summarized in chap. 5 of the 4th edition of the SSRC "Guide" and the beam-column work is described in Chap. 8 of the same reference (Galambos, 1988). The extent of the literature is truly gigantic, and proven methods exist for solving any new problem which may have been missed previously.

What benefits have resulted from all of this work to the practicing design engineer? The motivation of much of the research was to provide justification for beam-column interaction equations which are " simple and safe". This philosophy still dominates the design criteria for these types of members in almost all of the world's design specifications. On the other hand, the design of beams against lateral-torsional and local buckling is currently (1994) much more in tune with the advanced research results.

Beam-columns are universally designed by "interaction formulas" which connect the centrally loaded column strength at one extreme and the beam strength at the other extreme with a simple empirical relationship. An example is shown in Fig. 1 of the interaction equation in the 1994 specification of the American Institute of Steel Construction (AISC, 1994). Similar equations are used in all the other steel design standards of the world.

Following is a brief description of the evolution of the beam-column interaction equations in order to illustrate that the problem of beam-column behavior is just too diverse to be able to be represented by one simple set of formulas. Figure 2 is the representation of interaction equations for in-plane behavior. The top equation is the original linear interaction equation which was shown to be unconservative in 1954 in the Council's "Philosophy of Column Design". The next two equations show modifications to include the in-plane deformations (P-δ effect) and the plastic strength. The third equation is actually a very accurate representation of experimental results for compact wide-flange beam-columns which are restrained against out-of-plane lateral buckling. The last two equations in Fig. 2 contain modifications for moment gradient; two equations are needed because of the artifice of the moment-gradient factor C_m, leading to the necessity of a dual check for member and cross-section strength. When the member is subjected to reverse curvature bending this method is very conservative. The equations in Fig. 3 contain the adjustments to account for lateral-torsional buckling and for biaxial bending, respectively. These equations are generally very conservative.

The interaction equations have been compared innumerable times to experimental results and to numerical predictions for wide-flange members so that their conservatism is appreciated by specification writers as the price to pay for simplicity. However, the use of these equations for singly symmetric sections (e.g. double angle and Tee sections), composite steel-concrete beam-columns and unsymmetrical shapes (e.g. unequal-leg angles) is not justified by either experimental or theoretical evidence. Different types of beam-columns are just too diverse to be characterized by one simple relationship. Through the 50 years of this Council's work enough tabulated and charted information has been amassed so that the actual strength of almost any practical metal or composite beam or beam-column can readily obtained. Why not use this data directly in the design office, as is done for the proportioning of beam-columns in reinforced concrete codes?

4. THE FRAME

Research on individual structural elements and members has resulted, as we have seen, in well substantiated analytical models so that the accuracy of the simplified design rules can be accounted for in the thinking of specification writers. Specification criteria were originally formulated under the slogan "extreme simplicity with assured safety" (George Winter in CRC, 1954). Considering the availability of modern computers, the same slogan applies today as it did in 1954.Current design specifications are still based on simplified methods. These are much more complex, to be sure, than the standards in 1944, but then we have much more than a slide-rule to work with!

The evaluation of the design of a frame, which is an assembly consisting of structural elements and members, is much more problematical since the behavior models cannot be checked experimentally except for the most simple structures. Following is a discussion of the evolution of ideas about frame behavior and design in the last 50 years within the Council.

Professor Winter, in the article "Compression Members in Trusses and Frames" (*CRC 1954*), summarized the status of frame stability and frame strength criteria in US building design specifications of the time:

> "Present column specifications are usually written either in terms of the actual length, L, regardless of the end conditions, or as in some bridge specifications for truss members, in terms of an effective length, kL, where k depends only on the type of end connection. This procedure is unsafe in some cases and uneconomical in others."

Professor T. C. Kavanagh wrote a few years later (CRC, 1957) in reference to the frame stability problem that

> "... It is a matter of awe that a subject having so fundamental an effect on both the safety and economy of the structure has been so blandly ignored in specifications of the past."

The solution that was put forth by Professor Winter was a modification of the beam-column interaction equation by incorporating the effective length factor in the formulas for P_{ux} and P_{ex} (Fig.2). This scheme, when it was proposed in 1954, was already in use in the German specification DIN4114 (CRC, 1954), and it was adopted by the AISC in its 1961 specification.

This illustrates the time it takes for changes in design standards. Even today there is considerable opposition to the use of the effective length factor in the beam-column interaction equations.

An enormous amount of the Council's research effort and creative thinking was expended on the many aspects of the calculation of the effective length factors, and this work continues still today in 1994. With the exception of the Canadian and the South African steel specifications the effective length concept is used in all other modern specifications in the interaction equations. The 1994 AISC specifications have a set of interaction equations which are very broad in their applicability: they can account for frame instability (elastic or inelastic), second-order bending due to lateral story displacements (P-Δ effect) and due to member bending (P-δ effect), out-of-plane member instability, in-plane member strength and local instability for forces determined by a first order elastic analysis. These equations have been successfully tested against experiments and analytical models for small and modestly sized structures. Can we, however, justify the leap of faith of extrapolating, say, to a building of the type of the Hancock Tower in Chicago? Can we do better than using empirical interaction equations?

At the same time as the Council's researchers worked on the improvement of the simplified design methods, work progressed very rapidly on the determination of the actual strength of frames using rational models. During the 1950-s and 1960-s this work was also part of the concurrent research of developing plastic analysis methods for the design of multi-story buildings, mainly at Lehigh University but also elsewhere (e.g. Manchester, Alberta and Washington (St Louis) Universities). Many landmark experiments were performed and many kinds of analytical models were explored. This was a very creative time for the Council. A concise summary of this work is given in Sec. 16.2 of the 4th edition of the SSRC Guide (Galambos, 1988). Recently, since the beginning of the 90-s, a major campaign has been mounted by SSRC under the leadership of Task Group 29, "Second-Order Inelastic Analysis for Frame Design".

The earlier research on frame behavior had as one of its major objectives the generation of data from physical and analytical-numerical experiments so that empirical design methods could be checked for accuracy and appropriateness. This means that some of the research was focussed on the development of empirical strategies so that forces from a simple first order, or a rudimentary second order, elastic analysis could be safely used in interaction equations in the task of member checking. Force analysis and member checking were thus two separate design steps. This way of design is still the predominant scheme used in modern steel design specifications.

The official position of the Council is stated in Technical Memorandum No. 5 (published in Feb. 1981 and reprinted in Appendix B of the SSRC Guide (Galambos, 1988)), from which the following quote is taken:

"Although the maximum strength of frames and the maximum strength of the components are interdependent,..., it is recognized that in many structures it is not practical to take this interdependence into account rigorously....Therefore, SSRC recommends that, in design practice, the two aspects, stability of individual members and elements of the structure and the stability of the structure as a whole, be considered independently. The proper basis for member design is the maximum strength of the restrained imperfect member. Where appropriate, second-order effects (such as P-Δeffects in frames) determined with due regard for non-linear and non-coexistent response, should be included with the first-order effects among the actions for which the member is to be designed."

This statement essentially reflects the situation as it exists in most of the world's steel design specifications in 1994, but it is not the state-of-research as it exists in 1994. The statement does not appreciate what can be done today. Thus SSRC Technical Memorandum No.5 needs to be brought up-to-date.

The current state-of-the-art is assessed in a book published by the Council (White and Chen, 1993) entitled "Plastic Hinge Methods for Advanced Analysis and Design of Steel Frames". The work leading up to this publication is the culmination of many years of research (mainly at Cornell and Purdue, but also at many other Universities such as Sydney and Minnesota) which was motivated by the conviction that the time is here when advanced designers need no longer rely on empiricism but they can use computer tools to determine the entire load-deformation history of the frame. The preface of this book states this as follows:

"....improvements in methods of non-linear analysis and in the power of personal computers and workstations have placed the comprehensive modeling of primary strength limit states within practical reach. Advanced analysis of large laterally restrained two-dimensional frames has become a practical reality..."

Work is currently in full swing to refine the methods and to produce software for use in design office practice. Research on frames with semi-rigid connections has also advanced to a stage where practical tools exist for application in design. These advances benefit mostly structural configurations where planar behavior is a valid assumption. Research is underway, and needs to be accelerated, on the advanced analysis of three-dimensional frames, frames where lateral instability may be a limit state and frames with steel-concrete composite beam-columns and beams.

One new motivation to this extensive research activity is that for the first time several of the world's structural steel design specifications permit, as an alternate to elastic analysis and member-by-member design using interaction equations, the application of "advanced analysis". The following quotation is from Appendix D of the Australian Standard 4100-1990 (Standards Association of Australia, 1990):

"For a frame comprising members of compact section with full lateral restraint, an advanced analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frame. The analysis shall take into account the relevant material properties, residual stresses, geometrical imperfections, second-order effects, erection procedures and interaction with the foundations....For the strength limit state, it shall be sufficient to satisfy the section capacity requirements..."

Other codes have, or will have, similar provisions permitting under certain conditions the performance of an advanced analysis that bypasses the member stability checks. It is possible, for the first time in the history of this Council, to foresee a practical path which allows the designers to break away from the empirical artifacts of the effective length factor and the interaction equation. Credit must here be given to Professor McGuire of Cornell, who inspired his students and colleagues by his belief that there is surely a better way to design a steel framed structure!

5. CONCLUSIONS

The Structural Stability Research Council was founded because Jonathan Jones cried out in 1941: "...get together in some kind of a central group and carry on the research and analyze the results in a way that is satisfactory to all". I believe we have done just that in the past 50 years and we have been very successful at what we did. It was a good thing to have formed this council separate from the larger organizations such as ASCE and the more promotion oriented groups such as AISC because

- ours is a small intimate group within which communication can be efficient;
- we all know each other not only through our research work but also personally;
- new research findings can be quickly communicated;
- we keep abreast of each other's work in the Annual Technical Sessions;
- there is continuity among the members, some serving a full professional lifetime (e.g. Lynn Beedle, Bruce Johnston, George Winter, Bob Ketter, Geoge Lee and many others that could be named);
- there is an excitement and a camaraderie in our annual Task Committee meetings that cannot be matched in any other technical group.

The impact of the Council has been extensive. The following points can be mentioned:

- Many researchers have drawn inspiration, advice and financial assistance from the Council throughout the 50 years of its existence.
- A remarkable network of international contacts have been established and maintained through the many international conferences sponsored by the Council.
- Structural engineering practice has been enriched by the various major publications, notably the four editions of the SSRC Guide, the "World View", and the proceedings of the Annual Technical Sessions.
- The impact of the Council's work on the world's structural design specifications has been truly phenomenal. I believe that this success is far beyond anything that Jonathan Jones could have imagined.
- Many of the researchers associated with SSRC are University professors, and naturally their knowledge is being transferred to their students.
- Finally, many of the members of the Council have contributed to the philosophical and intellectual underpinnings of modern structural mechanics.

In this paper it was attempted to illustrate the exciting developments of the first 50 years of SSRC, by using as an example the tracing of the evolution of our understanding of the behavior of framed steel structures. Other examples could have been used just as well, such as the research on

- plate and box girders
- cold formed structures
- tubes and shells

In any of these endeavors we can say to Jonathan Jones, "the results are satisfactory to all".

What of the future? I have attempted to relate the developments now taking place in the area of frame design, where it will be possible to rationally design complex frames with semi-rigid joints and geometric initial imperfections in the near future. Following are some other directions that should be explored:

- stability of nonmetallic and metal-nonmetal composite structures;
- stability of thin-walled and/or latticed structures in applications such as structures in outer space, containers, transportation vehicles, etc.;
- dynamic instability of slender structures in unusual settings;
- reliability of complex structural systems which are subject to failure by instability;
- exploring the interactions between instability by compressive phenomena and by tension phenomena (buckling-versus-fracture);
- instability of structural members in contact with soil or rock foundations (piles, cofferdams etc.);

I am sure many additional possibilities could be added. The exciting aspect of all of this is that we now have the theories and the tools to solve any problem that can be formulated. How much more fortunate we are than the founders of this Council who had not many aids in their search for truth! But, they had a belief that the problems in technology posed by instability can be solved and they led the way. May we the same foresight to forge ahead to the next set of problems, thus keeping the Structural Stability Research Council in the forefront for the coming era of technological development.

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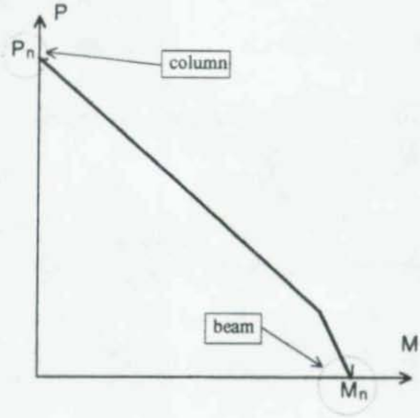

Fig.1 Interaction Diagram (AISC, 1994)

ORIGINAL INTERACTION EQUATION

$$\frac{P_u}{P_{nx}} + \frac{M_{ux}}{M_{yx}} \geq 1.0$$

MODIFICATION TO INCLUDE IN-PLANE DEFLECTIONS
UNIFORM BENDING

$$\frac{P_u}{P_{nx}} + \frac{M_{ux}}{M_{yx}\left(1 - \frac{P_u}{P_{ex}}\right)} \geq 1.0$$

$$\frac{P_u}{P_{nx}} + \frac{M_{ux}}{M_{px}\left(1 - \frac{P_u}{P_{ex}}\right)} \geq 1.0$$

MODIFICATION TO INCLUDE MOMENT GRADIENT

$$\frac{P_u}{P_{nx}} + \frac{M_{ux} C_m}{M_{px}\left(1 - \frac{P_u}{P_{ex}}\right)} \geq 1.0$$

$$\frac{P_u}{P_{yield}} + \frac{M_{ux}}{M_{px}} \leq 1.0$$

Fig. 2 Evolution of the Interaction Equation - In-plane Deformations Only

INTERACTION EQUATIONS FOR X-AXIS BENDING
AND LATERAL-TORSIONAL BUCKLING

$$\frac{P_u}{P_{ny}} + \frac{C_{mx} M_{ux}}{M_{nx}\left(1 - \frac{P_u}{P_{ex}}\right)} \leq 1.0$$

$$\frac{P_u}{P_{yield}} + \frac{M_{ux}}{M_{nx}} \leq 1.0$$

INTERACTION EQUATIONS FOR BIAXIAL BENDING

$$\frac{P_u}{P_{ny}} + \frac{C_{mx} M_{ux}}{M_{nx}\left(1 - \frac{P_u}{P_{ex}}\right)} + \frac{C_{my} M_{uy}}{M_{py}\left(1 - \frac{P_u}{P_{ey}}\right)} \leq 1.0$$

$$\frac{P_u}{P_{yield}} + \frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{py}} \leq 1.0$$

Fig. 3 Interaction Equations for Lateral-torsional Buckling and Biaxial Bending

P_Y, P_E, P_T, K, Δ/L and σ_{RC} IN THEORY AND PRACTICE

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INTRODUCTION

Perhaps for no other structural member than a column, a central loaded column, an axially loaded compression member, has the transition from theory and research to practice been so slow and so difficult. One could argue, with considerable justification, that it has not yet completely taken place.

Progress seems to have been in spurts separated by significant time intervals as is deduced from the historical summary in Bleich (1952). Bleich's treatise was promoted by the Column Research Council. Euler in 1744, 250 years ago, established the concept of an elastic buckling load. The next step was one hundred and five years ago when Engesser in 1889 introduced the tangent-modulus theory for inelastic buckling. Forty seven years ago Shanley (1947) established that for perfectly straight members the tangent-modulus was the appropriate measure of the inelastic buckling load and that the double-modulus load, which had been in vogue for 40 years or so, was a load that was approached only with relatively large lateral deflections.

Under the auspices of the Column Research Council, the Lehigh Group (Beedle and Tall 1960) made a most significant step by demonstrating that the inelastic behaviour, for so long attributed incorrectly to out-of-straightness or inevitable testing eccentricities, was in substantial measure due to the gradual reduction in the effective flexural stiffness, EI, due to the presence of residual stresses. As the effective flexural stiffness depends on the cross sectional shape, the axis of buckling and the residual stress pattern there could exist more inelastic buckling curves than there are structural shapes. One approach is to group shapes with similar inelastic curves together.

What other approaches are there and how best can we take into account the contribution of out-of-straightness as well? Do limit states design principles help us here? What are possible future developments in the humble column curve or do we have to wait another half century from 1960 or from now?

EARLY CONTRIBUTIONS

No doubt early constructors realized that there was an upper limit to the strength of compression members. Wood crushed; cast iron, marvelous in compression was rent asunder (probably with some tensile bending stresses) and, somewhat later in time, some steel members failed in compression at P_Y, the yield load. But this limit, as depicted in Figure 1, scarcely described the behaviour of columns.

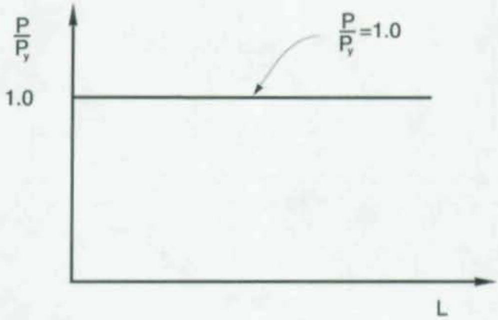

Fig. 1 A primitive column curve

Caught up in the concept of stresses, allowable stresses at that, to ensure the safety of the structures pre-Eulerian engineers did not appreciate the nuances of the buckling problem, the investigation of potential unstable equilibrium between the external loading and the internal response of the structure. Limit states design, were it in vogue, would have at least directed the designers attention to the question: What load does the column have to carry and can it carry that load? Empirical designs were, in fact, the order of the day.

In 1744, Euler pointed to the problem as being one of stability and fifteen years later gave the critical buckling load for a flag-pole column, using today's terminology, as

[1] $$P_{cr} = \frac{\pi^2 EI}{4L^2}$$

In general form for other end conditions then

[2] $$P_E = \frac{\pi^2 EI}{(KL)^2}$$

where KL is an effective length defining a portion of the length between points of zero curvature and having the same relative shape as the entire pin-ended column, and EI is the flexural stiffness.

With this advance, if she were to design in steel, the engineer had a two-part column curve, the Euler curve with a cap at P_Y as shown in Figure 2. But many real columns didn't attain either P_Y or P_E, as shown in Figure 3.

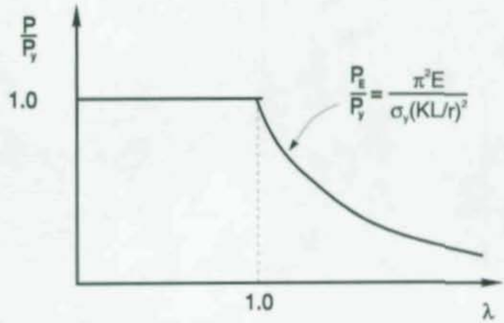

Fig. 2 Post-Eulerian column curve

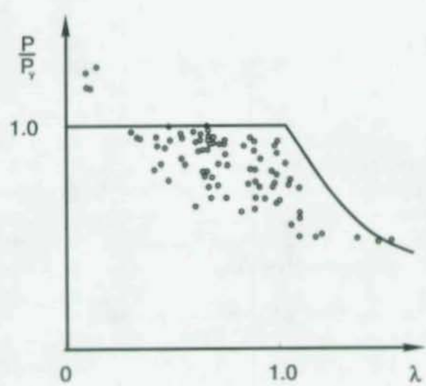

Fig. 3 An elasto-plastic column curve with test results (from Tall 1964)

In 1889, Engenser dealt with inelastic buckling by introducing the tangent modulus in Euler's equation

[3] $$P_T = \frac{\pi^2 E_T I}{(KL)^2}$$

to describe the bifurcation load for an initially straight column buckling inelastically. This incidentally, is given as equation [1] in Beedle and Tall (1960). It is noted that the tangent-modulus curve shown in Figure 4 is not really a continuous curve but represents countless contiguous points each one of which is on an Euler type curve with its own tangent modulus of elasticity and represents the combination of effective length and inelastic buckling load consistent with that tangent modulus.

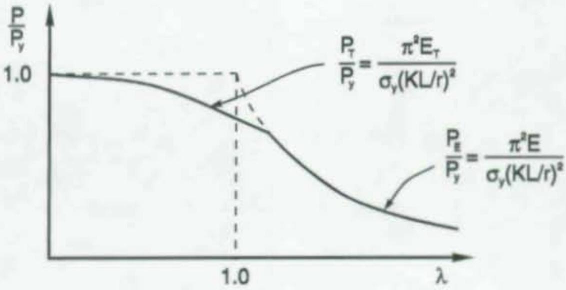

Fig. 4 An elastic-inelastic buckling curve

Two problems arose which hindered further development and lead to a long hiatus. The first was that the inelastic buckling load was more properly defined as the double-modulus or reduced modulus load not the tangent-modulus load. However, it was not until 1947 that Shanley established that, for perfectly straight members, the tangent-modulus load was the appropriate measure of the inelastic buckling load and that the double-modulus load was only approached with relatively large lateral deflections.

These discussions probably obscured the more fundamental problems as to why steel columns, coupons from which displayed essentially elasto-plastic behaviour, failed in the inelastic region at loads less than those given by the yield load, P_y, and only reached the Euler load, P_E, at large slenderness ratios. Even a tangent-modulus load did not describe the test results adequately. It appears that these divergencies between test and theory were rationalized principally on the basis of (i) initial crookedness or out-of-straightness of the columns, (ii) initial end eccentricities if the test results were too low and (iii) initial unforeseen end restraints if the test results were too high - what rationalizations! Geometric imperfections were considered to be the cause of the divergence. Empiricism reigned supreme. Straight lines, parabolas and other curves were used to fit test results that varied considerably. Unfortunately, the behaviour of columns could be modelled, at least over some range of slenderness ratios, by simply selecting an appropriate end eccentricity or alternatively an out-of-straightness.

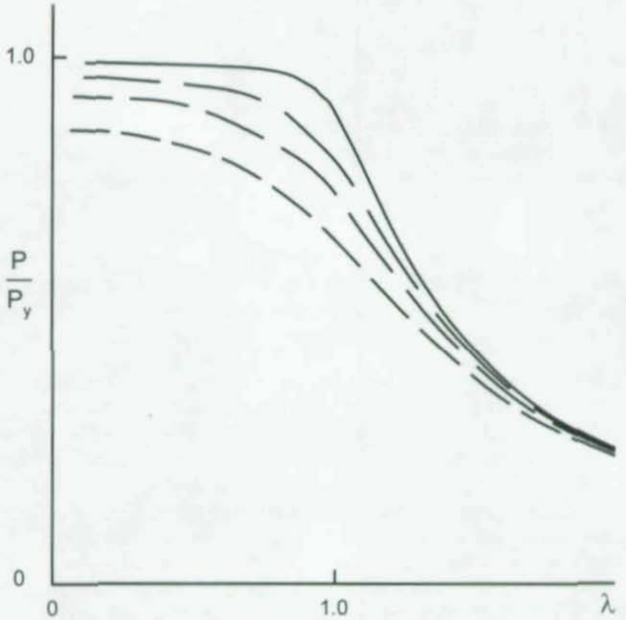

Fig. 5 Some secant curves

The secant formula,

[4] $$\sigma = \frac{P}{A}\left(1 + \frac{ec}{r2} \sec \frac{L}{2r} \sqrt{\frac{P}{EA}}\right)$$

an example of the former with an assumed initial end eccentricity e, also assumes that the modulus of elasticity remains constant up to the yield point. Figure 5 show some secant curves. Bleich (1952) wrote:

"the high regard in which the engineering profession has held .... the secant formula has its origin in the fact that, for small values of the eccentricity, this formula leads to a column curve for axially loaded columns which complies fairly well with values of P/A derived from column tests provided that an "equivalent eccentricity" e in [4] is suitably but arbitrarily selected to agree with these tests".

Although he stated this may have been a happy coincidence, Bleich realized this did not bear any relation to the fundamental aspect of the problem as one of stability. In my view it was an unfortunate coincidence as the fundamental cause of the problem remained obscured.

RESIDUAL STRESSES

Galambos (1968) states that the idea that residual stresses could be the cause of the lower than expected strength of steel columns of intermediate length was put forward in 1888 (See Salmon 1921) but this was not demonstrated until 1951 by Osgood.

Order truly emerged from chaos when research work, at this time, at Lehigh University, (See Beedle and Tall 1960 for a summary) under the guidance of Research Committee A of the Column Research Council, showed quantitatively that the strength of columns was a function of the residual stresses present. As is now well known, when the sum of the applied stress at any point on the cross section plus the residual compressive stress equals the yield strength at that point the stiffness there approaches zero. Thus, the effective moment of inertia determined from the area that has not yielded decreases. This can be considered to be equivalent to a decreased (tangent) modulus of elasticity over the entire area. Obviously however the effective moment of inertia for any stress level depends on which portions of the cross section have yielded and therefore on the pattern of residual stresses and on the axis of buckling. In general then

[5] $$P_{cr} = \frac{\pi^2 E I_e}{L^2}$$

for a straight pin-ended column where I_e is the effective moment of inertia for the axis of buckling. It follows immediately that there could be a host of inelastic buckling curves depending on the structural shapes, the yield strengths, the maximum residual stresses, the residual stress patterns and axes of buckling. As well, the effects of variations in the yield strengths over the cross section and the effects of cold straightening procedures may need to be assessed.

Yet, with these new insights, when the gradual reduction of stiffness is taken as the predictive tool, good correlation is obtained between the test results on central loaded columns and the predictions, as shown in Figure 6 for weak axis and strong axis buckling of a W Shape and also for two tests on annealed columns. The latter two fall near the line $P/P_Y = 1.0$, as would be expected when the cooling residual stresses are removed by annealing.

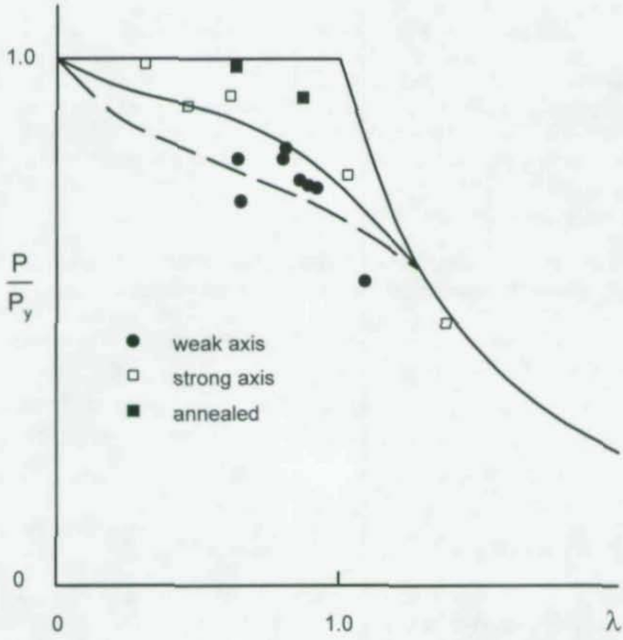

Fig. 6 Test results and buckling curves for a W shape
(from Beedle and Tall 1960)

Beedle and Tall also show that the "CRC curve" (Johnston 1976) initially published in 1960, fitted their data reasonably well in the inelastic region. This curve, simplified design procedures by eliminating consideration of the flexural axis, and is used in the AISC allowable stress specifications (AISC, 1978) with a variable factor of safety.

In summing up the research on the effects of residual stresses, Beedle and Tall (1960) state that any assumption of "the magnitude of the accidental eccentricity or initial curvature must necessarily have been arbitrary since a *considerable portion of the reduction* in column strength is now known to be due to the presence of residual stresses rather than eccentricities and design curves for column strength based on the tangent modulus method modified by the presence of residual stresses, reflect actual conditions rather than a reliance on assumed irregularities".

We do not argue, nor did the Lehigh group, with statements such as those just quoted, *that initial curvature, crookedness or out-of-straightness play no role in the buckling of* columns but there is no doubt that residual stresses play the predominant role and that the work on residual stresses cleared up decades, if not two centuries, of confusion about the inelastic buckling of columns. The work, under the guidance of the Column Research Council, was instrumental in bringing reason to the scene and bringing order from chaos.

INITIAL OUT-OF-STRAIGHTNESS

Galambos (1986) quotes from Technical Memorandum No. 5 of the Structural Stability Research Council to show that its position is that the proper column strength model is one that incorporates both the effects of residual stress and the initial out-of-straightness. Thus, both these effects are to be considered as was done by Bjorhovde (1972) in developing sets of column curves as discussed subsequently.

In most cases the initial out-of-straightness is set at 0.001 of the length, i.e. at about the maximum mill tolerance, although Bjorhovde also used an initial out-of-straightness of the mean value he established of $\frac{1}{1470}$ or 0.00068.

MULTIPLE COLUMN CURVES

The broad dispersion of test results, particularly in the inelastic range, led to the development of multiple column curves. In 1959, German Standard DIN 4114 introduced one curve for tubes and another for all other shapes. The European Convention for Construction Steelwork recommendations (See Sfintesco 1970, 1976 for example) led to code adoptions in several countries. In the ECCS recommendations as shown in Figure 7, structural shapes are assigned to one of five column curves, as appropriate, depending on factors such as cross sectional shape, the proportions of the cross section, the forming process, the weight of the cross section and axis of buckling. The highest curve, a°, is for annealed shapes of high strength steels and the lowest curve, d, is for rolled heavy W and welded heavy H shapes made from universal mill plates.

Bjorhovde (1972) proposed two sets of 3 column curves each, to which various structural shapes could be assigned. The first set based on an initial out-of-straightness of 0.001 of the length are known as SSRC curves 1, 2, 3 and the second set with an initial out-of-straightness of 1/1470 are known as SSRC curves 1P, 2P and 3P.

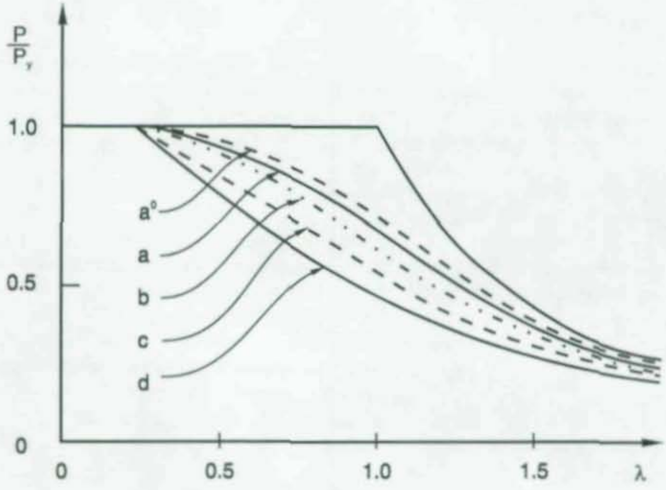

Fig. 7 ECCS Multiple column curves
(from Galambos 1986)

Bjorhovde compared the results of computer numerical analyses of maximum strengths to data from column tests to establish the validity of the former and then derived 112 column curves for members for which the residual-stress distributions are known and assuming a sinusoidal initial out-of-straightness of one of the two prescribed values.

RESEARCH INTO PRACTICE

Some examples of the application to practice of the research results of the SSRC and its forerunner the CRC and of others have already been given. For structural engineering, this application frequently takes the form of the adoption of theoretical and experimental research (and their more useful combinations) in codes, standards and specifications.

No steel specification in the world is without the Euler curve. All column curves, we suggest, have been influenced by the work of the effect of residual stresses on inelastic buckling.

The AISC allowable stress design column curves is the CRC curve of Johnston.

In Europe (Beer and Schultz 1970) the five ECCS multiple column curves have been adopted broadly.

The LRFD specification of the AISC (1986) uses a single curve fitting SSRC curve 2P.

In Canada, the 1994 edition of CSA Standard S16.1-M94 "Limit states design of steel structures" will use all three SSRC curves. Using first order second moment techniques, Kennedy and Gad Aly (1970) showed that, for a reliability index of 3.0, a resistance factor of 0.90 was appropriate for use with SSRC Curve 1 for Class H (hot rolled or cold rolled stress relieved) hollow structural sections and they were assigned to that curve in the 1974 edition of the standard. Other shapes, including welded H shapes with flame-cut edges were assigned to SSRC curve 2.

Subsequent work (Chernenko and Kennedy 1991) has resulted in welded H shapes made from flame-cut plate being raised to SSRC curve 1. This results from both the favorable residual stress pattern and the measured out-of-straightness. In this work some 50 computer simulations, corroborated by documented test results of McFalls and Tall (1969) were used to determine maximum column loads for a range of residual stress patterns, and out-of-straightness and for several values of the slenderness parameter, λ. A procedure was developed to assess the statistical variation of out-of-straightness and residual stresses sequentially in order that resistance factors, ϕ, appropriate to some design equation could be evaluated from

[6] $$\phi = \rho_R \exp(-\beta\alpha V_R)$$

where ρ_R is the bias coefficient of the resistance, V_R is its associated coefficient variation, β is the desired reliability index, taken as 3.0 in the Canadian steel design standard for member behaviour, and $\alpha = 0.55$ is a coefficient of separation as proposed by Galambos.

In addition to assessing the effect of material and geometrical variations on the bias coefficient, ρ, and the coefficient of variation V_R, Chernenko and Kennedy expressed its other constituent part, the test/predicted ratio, ρ_P, as

[7] $$\rho_P = \rho_s \cdot \rho_n \cdot \rho_{ex}$$

In [7] ρ_s is the ratio of the computer simulated strength to that given by the appropriate design equation, evaluated at the mean value of out-of-straightness and average compressive residual stress for a given value of λ. The term ρ_n accounts for the variation due to different residual stress patterns and other minor factors and ρ_{ex} is the mean value of the ratio of the experimental strength to that determined by computer simulation.

Of course with each of the bias coefficients there is an associated coefficient of variation which can be combined, assuming statistical independence, as the square root of the sum of the squares. In fact there are four separate coefficients of variation to be determined , two associated with ρ_s one arising each from the out-of-straightness and residual stress effects and assessed sequentially and the other two associated with ρ_n and ρ_{ex}. Chernenko and Kennedy (1991) describe the procedures used.

Figure 8 shows the variation of the test/predicted ratio for three different values of the slenderness parameter λ as a function of the out-of-straightness and the average compressive residual stress.

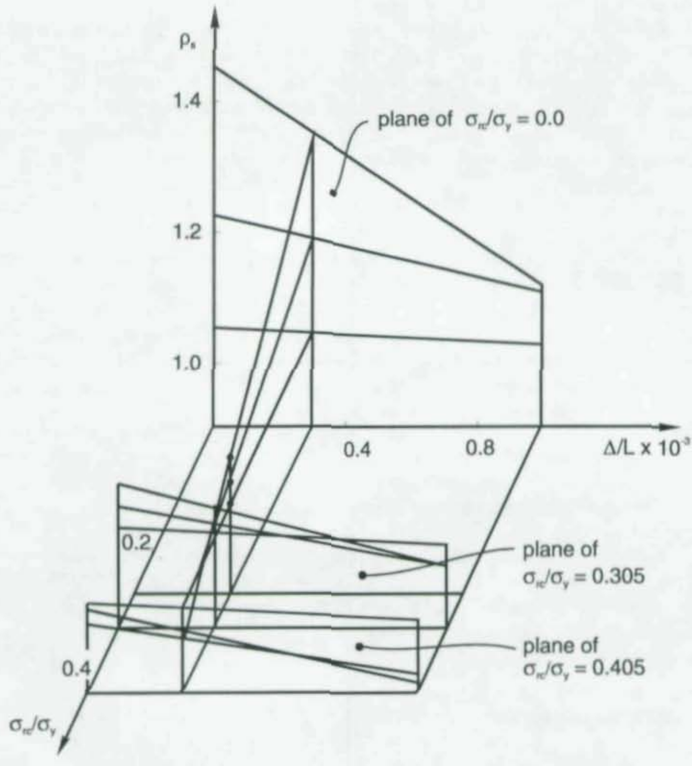

Fig. 8 Test/predicted ratio vs compressive residual stress and out-of-straightness

In the 1994 edition of the Canadian Standard (CSA, 1994), after some years of soul searching, SSRC Curve 3 for "heavy" shapes has been added. There was no rational basis for not doing so. Thus the limit states design standard to be published soon will give all three SSRC curves.

There is no doubt that the Canadian approach has been markedly influenced by the move to limit states design first adopted in the National Building Code of Canada (ACNBC 1975) in 1975. Moreover, since 1990 only limit states design provisions are referenced in the National Building Code and designers are therefore required to use limit states design procedures for the design of steel structures. (This also applies to the design of highway bridges (OHBDC 1991) and fixed offshore structures (CSA 1992). The overriding criteria in establishing load and resistance factors has been to maintain economy and provide as uniform a level of safety as possible without undue complexity and with the desired small probability of failure. In some cases, loads (e.g. snow drifts on stepped roofs) have become greater and in other cases resistances have increased or decreased. The net effect has been structures with more uniform and adequate safety and with increased economy of construction.

The limit states design or LRFD approach does emphasize that no longer is it necessary or desirable to use lower bound curves or to develop curves based on the maximum out-of-straightness or maximum residual compressive stress. What is of concern is the likely or probable combinations of the two. The realistic approach, therefore, is to combine the relative parameters statistically based on their bias coefficients and coefficients of variation to determine resistance factors to give the desired probability against failure as may be measured for example by the reliability index. In fact, Monte Carlo simulations can be used (Kennedy and Baker 1984) to combine probability density functions that have different characteristics.

It follows directly then that by treating more restricted populations, for example welded wide flange shapes made from flame-cut plates as done by Chernenko and Kennedy, the dispersion of the sample as measured by the coefficient of variation is reduced and more favourable resistance factors are obtained for the same degree of reliability.

It is concluded, therefore, that the results of SSRC research, with contributions from others as well, has had a profound effect on the design of columns around the world and apparently, somewhat surprisingly, a greater effect in countries other than the USA. Of course, the use of these column curves has a direct effect on the design of beam-columns as well.

FUTURE DEVELOPMENTS

Where do we go from here? For me, forced to retire as an academician in a few days, I might wish to just remain with a piece of paper, my pencil and my slide rule, or perhaps, make that my pocket calculator. But the extremely powerful desktop computer is here. It is no more difficult to access a column strength for a particular number based on one column curve and which, therefore, by definition must accommodate all the dispersion or scatter we have discussed and, therefore, be more conservative, than it is to have the strength based on one of a number of curves. The data for a particular member could even be customized. Each refinement reduces scatter and leads to a better assessment of the strength, more uniform safety, and more structure for the dollar.

Further statistical analyses are, at the very least, required to maximize safety and economy. These must be based on the statistical measures of the relevant parameters such as compressive residual stresses, residual stress patterns, out-of-straightness, axes of bending and weight of the cross-section and the effects of these combined in the proper statistical way. By treating more restricted populations (not all columns are created equal) safety and economy result.

But perhaps the Australians (SAA 1990) are already ahead of us. They have proposed procedures where, in the analysis of the structure, it is not necessary to assess the behaviour of members under a set of end forces and moments, but the member behaviour, its load-deflection response, is determined as part of the structure (at least, so far, for in-plane effects).

Currently in Canada computer programs are being developed, operational on 486's, that will assess the lateral torsional buckling strength (if that is the mode of failure) of beam columns under arbitrary loading, and lateral and torsional restraint conditions and that take into account distortion of the cross-section if, in fact, that occurs. Different residual stress patterns as appropriate to the section under consideration can be considered.

In my view, with the splendid impetus given by the research work under the aegis of the SSRC there is no reason at all to use "one design curve fits all". Rather, the question is: Should we have multiple column curves with cross-sections assigned to the "nearest" appropriate one or is the eventual step that we take to have no curves at all?

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FRAME ANALYSIS : A BRIEF REVIEW OF THE EUROPEAN TRENDS

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1. INTRODUCTION.

The European trends in performing global analysis of steel frames is the best reflected by the relevant chapter of Eurocode 3 - Part 1.1. [1].

While structural frameworks are basically 3D structures, it is most often allowed to identify a series of - usually parallel - primary bearing frames, that are connected to each other by secondary structural elements in the transverse direction. Thus the frame action develops mainly in the primary direction.

To the author's knowledge, no existing code or standard in Europe gives provisions regarding the design of frameworks as real 3D structures. The design practice still consists in considering the primary frames and analysing each of them as a plane frame. Then, each structural element of a specified frame is checked for its own (bending, compression, compression and bending,...). Should a structural element belong to two plane frames, it shall be checked for biaxial bending and possible compression.

Traditionally the aspects of inelastic material behaviour and second-order effects due to changes in geometry have been accounted for solely in specification provisions for member design; first order elastic global analysis has been used predominantly to estimate the distribution of internal forces.

In European standards, and especially in Eurocode 3 [1], attempts have been recently made with a view to allow for more accurate and realistic estimates of the carrying capacity of frames. Thus, plastic global analysis can be contemplated and whatever the type of global analysis, second-order effects may be accounted for. In addition, frameworks are imperfect by nature; they shall thus be analyzed as imperfect structures.

Any method of second-order inelastic global analysis involves the direct consideration of both member strength and stability. Provided the significant behavioural effects be accounted for properly in the global analysis, separate checks of member design equations are simplified or even unnecessary. In [2], the writers express the idea that "AS 4100 (AISC, 1990) is the only design specification at present which explicitely allows the designer to disregard member capacity checks if an advanced inelastic analysis is employed". The author is willing to stress that Eurocode 3 [1] allows similarly, though he agrees that such an allowance results much more from a careful reading and interpretation than from a limpid writing. It should be well understood that if all the significant in-plane behavioral effects are properly modelled in the global analysis, and if the frame is proportioned to prevent out-of-plane buckling, the conventional interaction equations for member strength need not to be checked.

As a consequence, the more refined the method of global analysis, the less complex the verification of the structural elements.

Present paper is aimed at explaining the whole range of facilities offered by Eurocode 3 for what regards the methods of global analysis and the relevant methods of verification of the structural elements.

2. METHODS OF GLOBAL ANALYSIS: GENERALS.

A given combination of actions of any nature induces a distribution of internal forces and moments, that complies with equilibrium and compatibility conditions. On how to perform the global analysis depends on the assumptions which are made regarding respectively : i) the material stress-strain response and the capacity to stress redistribution, either within the cross-section or between cross-sections, and ii) the effects of the in-plane displacements on the frame response.

Thus, idealisations bear on the rheological behaviour - with the result of an *elastic* or a *plastic* global analysis -, on the one hand, and, on the geometrical behaviour - with the consequence of a *first order* or a *second order* global analysis -, on the other hand.

2.1. Rheological behaviour.

For an *elastic analysis*, the stress-strain behaviour of the steel material is assumed indefinitely linear elastic; the bending-curvature response is similarly linear.

For a *plastic analysis*, the stress-strain relationship is either close to reality - with a knee connecting the yield plateau to the range of linear elastic behaviour - or idealised according to either a bilinear diagram or a model where elastic strains are fully disregarded with regard to the plastic ones. The corresponding bending-curvature response is the one relevant to the modelling of the σ - ε constitutive law . One has thus to distinguish between : i) the *elastic-perfectly plastic* response, advocating the concept of "concentrated" plastic hinge with due account taken of elastic strains, ii) the *elastoplastic* response, involving the concept of plastic zones (in contrast to notional plastic hinges), and ii) the *rigid-plastic* response, where elastic strains are fully disregarded and plastic strains are concentrated into plastic hinges.

2.1.1. Elastic global analysis

Elastic global analysis may be used in all cases, for both first order and second-order elastic analysis.

Because the stress-strain behaviour of the steel material is assumed linear whatever the stress level, the global analysis does not give any consideration to the resistance of cross-sections; the latter must thus be checked separately. The elastic analysis may still be adopted where the resistance of a cross-section is based on plastic resistance.

It is allowed to modify the distribution of bending moments calculated based on a first-order elastic analysis by shedding up to 15 % of the peak calculated moment in any member provided : i) the member concerned has Class 1 (plastic) or Class 2 (compact) cross-section, ii) the internal forces and moments in the frame after redistribution remain in equilibrium with the applied loads, and iii) the member concerned is not prone to either in-plane or out-of-plane buckling. Proceed to such a redistribution is equivalent to a plastic analysis where only a limited plastic redistribution would be allowed and where plastic rotations would be restricted to the rotation capacity of Class 2 sections.

2.1.2. Plastic global analysis

Plastic global analysis may be used only where : i) the material has sufficient ductility, ii) the cross-sections where plastic hinges occur are capable of reaching their respective plastic resistance and exhibiting an available rotation capacity which allows for the formation of the expected collapse plastic mechanism, and iii) the loading is predominantly static.

According to EC3 [1], the conditions for material ductility bear on the ultimate-to-yield stress ratio ($f_u/f_y > 1{,}2$), on the elongation of failure ($\varepsilon_f \geq 15$ %) and on the ultimate-to-yield strain ratio ($\varepsilon_u/\varepsilon_y \geq 20$). Steel grades S235 and S355 may be accepted as complying with these requirements; also grades S42ON and S460N, subject to specifications in compliance with Annex D of EC3 [3], do so.

At present time, any member having Class 1 cross-section and made with a steel grade satisfying above criteria is considered as appropriate for plastic global analysis. An explicit check of the available rotation capacity is thus not required. Some recent research works have however demonstrated that the available rotation capacity is governed by more factors than above ones. Recently it was attempted to express the *available rotation capacity* as a function of a whole set of basic parameters [4,5]; this approach is based on the results of some experiments and numerous numerical simulations. Because the latter are rather difficult to perform -it is indeed necessary to account for strain hardening and to describe the beam behaviour beyond the rotation where the maximum bending resistance is reached -, on the one hand, and because of the few available test results, on the other hand, this item - the available rotation capacity - does undoubtfully require additional investigations; some of them are at present in progress at RWTH Aachen, Germany.

Of course, the available rotation capacity may alternatively be assessed by tests. In such circumstances, the question may arise of how much the partial safety factor to be applied on the measured values.

Above requirement of predominantly static loading is justified by the fact that EC3 does not give any provision regarding shake-down, also termed alternating plasticity. This phenomenon, which involves both yielding and repeated loadings up to the elastoplastic range, should be expressed in terms appropriate to codes or standards for steel buildings and bridges.

When plastic global analysis is used, lateral restraint shall be provided at each cross-section where a plastic hinge may occur.

Based on many comparative computer simulations conducted on calibrating frames [6,7,8], there is some evidence that elastic-perfectly plastic analysis is most often quite acceptable, though it results in a very slight increase of the structural stiffness, compared to an elastoplastic analysis. Such an analysis provides indeed a by far sufficiently accurate estimate of the ultimate loads, whether second order effects be or not accounted for.

Rigid plastic global analysis should understandably not be used for second-order analysis; indeed the P-Δ effects are significantly contributed by the elastic deformations which are fully disregarded in such an analysis.

3. CLASSIFICATION OF STRUCTURES.

Depending on their ability to horizontal displacements, structures are classified as :

- *braced* or *unbraced*
- *sway* or *non sway*.

Sway resistance may be provided by either i) the frame action of the structure properly so called, or ii) by a bracing system.

A plane frame is termed as *braced* when the sway stiffness in its plane is provided by a bracing system, whose rigidity to horizontal loads is such that these are predominantly resisted by the bracing system. (Such a definition applies to multi-storey frames with a bracing system at each storey). For practice purposes, there is a need for the bracing system to reduce horizontal displacements by at least 80 % at each storey. The bracing system may, as for it, be either sway or non sway.

A plane frame is termed as *non sway* if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its storeys. This assumption may be accepted as complied with when, for a given load case, the normalized load ratio V_{Sd}/V_{cr} is not greater than 0.1, where V_{Sd} is the design value of the total vertical load and V_{cr} the elastic critical value for failure in a sway mode. For multi-storey frames with beams connecting each column at each storey level, this criterion writes simply: $(\delta/h)\ (V/H) \leq 0.1$, where V and H are the total vertical and horizontal reactions respectively at the bottom of each storey of depth h, and δ is the horizontal displacement in this storey due to both horizontal and vertical design loads and equivalent horizontal forces substituted for the initial sway imperfection. (δ is computed according to a first order theory).

Above simple criterion for non sway frames is of course established based on the assumption of vertical columns and horizontal beams. It does not apply as such to most single-storey industrial buildings, because of the usual slope of the rafters. Therefore, limits should be given to the latter where this criterion would still be usable.

4. EFFECTS OF DEFORMATIONS.

Internal forces and moments on a plane frame may be determined using either:

a) *first-order* theory

b) *second-order* theory.

According to the *first-order theory*, the internal forces and moments are in equilibrium with the external loads when reference is made to the non-deformed shape of the structure.

In contrast, when use is made of the *second-order theory*, account is taken of the influence of the deformation of the structure and the internal forces are determined accordingly . Because the deflected shape is the consequence of the internal forces distribution, while the latter depends on the deformed configuration of the structure, the process cannot be straightforward and requires an iterative procedure.

Second-order theory may of course be used in all cases. Should it not be necessarily required, then a first-order theory will be used in order to avoid any iterative time and money consuming process. That is allowed when either i) the influence of the deformations on the internal forces distribution- i.e. the second order effects -is negligible, or ii) an indirect allowance for second-order effects can be contemplated, without a significative loss of accuracy compared to direct account of these effects.

According to EC3, second-order effects can be fully disregarded when the frame can be termed as braced or non-sway in accordance with the relevant classification criteria. On when indirect allowance for second-order effects is permitted depends on several factors that are examined further more in detail.

5. EFFECTS OF GEOMETRIC IMPERFECTIONS.

According to EC3 [1], geometric imperfections must be accounted for at one and the same time: i) in the global frame analysis, ii) in the analysis of the bracing system, and iii) in the design of the structural elements. The magnitudes adopted for these imperfections are slightly different from those suggested by ECCS [9].

Frame imperfections must be understood as unavoidable out-of-plumbs of the columns. The out-of-plumb corresponds to an initial sway $\phi = k_c\ k_s\ \phi_o$; ϕ_o is a basic reference sway (ϕ_o = 1/200), while factors k_c and k_s depend respectively on the number of columns per storey and on the number of storeys; both figures are specified in such a way that they are roughly in the range [0.7, 1] for k_c and [0.45, 1] for k_s. An increase in the number of columns (resp. of storeys) results in a decrease of k_c (resp. of k_s); that means that the larger the number of columns (resp. of storeys), the easier the correction of frame imperfections during erection. Some guidance is given on how to compute ϕ when columns do not extend to all the storeys or when floor levels are not connected to all the columns.

External loads - mainly horizontal and vertical - shall thus be applied to a frame fitted with the appropriate initial sway. Performing computations accordingly is not especially convenient. Therefore EC3 allows to replace the initial sway imperfection by a system of equivalent horizontal forces, that shall be applied at each floor level and be proportionate to the vertical loads applied to the structure at that level. These equivalent horizontal forces shall be accounted for in all the load combinations.

In the analysis of the *bracing system* that provides lateral stability, the effects of imperfections shall be allowed for by means of an equivalent geometric imperfection of the members to be restrained, in the form of an initial bow $e_o = k_r\ L/500$, where $L/500$ is the basic reference imperfection and k_r a factor that depends on the number of members to be restrained. This initial bow may also be replaced, for sake of conveniency, by an equivalent stabilizing force.

Member imperfection consists in an initial bow imperfection of magnitude e_o, that includes not only the initial unavoidable geometric out-of-straightness but also the effects of residual stresses and of material heterogeneity. Normally member imperfection need not be accounted for in the global frame analysis. It does only for any compression element of sway frames that is: i) end restrained, and ii) subject to an axial force exceeding 25 % of the critical buckling load of this element in pin-ended conditions (The latter condition results in a reduced slenderness larger than $0.5\ (Af_y/N)^{0.5}$); then a second-order global frame analysis must be performed. Thus, most often, the global frame analysis may be performed with the sole frame imperfections and the compression elements have to be checked afterwards with respect to buckling. To do so, of course a second-order analysis of the member fitted with the member imperfection may be carried out; preferably, the effects of imperfections on member design shall be implicitely incorporated by using the appropriate buckling formulae. Should the member imperfection be explicitely accounted for in a second-order analysis, its magnitude depends on many factors, (slenderness, buckling axis, section type, class of section, relevant buckling curve and method of global analysis).

In current frames, rare are the circumstances where element imperfections must be explicitely accounted for in the global analysis.

6. SELECTION OF THE METHOD OF ANALYSIS.

Any structure with a bracing system such that the criterion of a *braced* structure is fulfilled may be analyzed according to a first order elastic or plastic method. The stability of the frame is controlled by checking the capacity of the structural elements which are subject to compression or to compression and bending, and using in this respect the system lengths as buckling lengths.

The *bracing system* properly has to be considered as a structure and analyzed by either a first or a second order theory, according as the frame is sway or non sway; due attention shall be paid to initial imperfections or to the equivalent horizontal forces.

A *non sway* unbraced structure may be studied as a *braced* structure, i.e. by using a 1st order method of analysis. Initial imperfections must be accounted for; for instance, the equivalent horizontal forces shall be superimposed to the external lateral loads.

For a *sway* unbraced structure, the global analysis must be conducted according to a second order theory, which may be either elastic or plastic.

How to account indirectly for second order effects is examined herebelow. The major relevant background is an ECCS publication [9], from which most of the EC3 provisions for global analysis are inspired.

The suggested methods of analysis are summarized in figure 1.

6.1. Elastic analysis.

Without any reservation, second order effects may be accounted for by performing a second order analysis. Unless the latter be conducted not only with frame imperfections but also with element imperfections, the stability check for the compression members is required; it is conducted by using the buckling lengths deduced from the *non-sway mode*; indeed the amplification due to second order effects is yet accounted for in the determination of the internal forces and has not to affect the resistance furthermore.

As an alternative, a first order analysis can be performed but the sway moments, i.e. those due to both external and equivalent horizontal loads must be amplified by a specified factor, according to the proximity of the bifurcation critical vertical resultant :

a) if $V_{Sd}/V_{cr} \equiv 1/\alpha_{cr} \leq 0.25$, the amplification factor is $(1 - (V_{Sd}/V_{cr}))^{-1}$ and the buckling lengths are taken as those corresponding to a *non-sway mode* ;

or :

b) in any case, the amplification factor may be taken equal to 1.2 for the verification of the beams and of the beam-to-column connections, provided the buckling lengths are those corresponding to a *sway mode*.

The value of α_{cr} for multi-storey buildings is given as the minimum of the values $(H/\delta)(h/V)$ computed respectively for each storey, where the symbols have the same meaning as in Section 3.

When an elastic global analysis is performed, checks in addition to those relevant for the structural elements are required: they are dealing with the resistance of the cross-sections and the possible out-of-plane buckling of the framework.

6.2. Plastic global analysis.

When plastic global analysis is used, allowance shall be made for the second-order effects in the sway mode by performing a second order elastic-plastic analysis.

However a first-order rigid-plastic analysis with indirect allowance for second-order effects (by amplifying the sway moments) can be used in the following cases provided $\alpha_{cr} < 0.2$:

a) frames with one or two storeys where either no hinge develops in the columns or the columns where plastic hinges occur are such that $\bar{\lambda} \leq 0.32\ (Af_y\ /N_{Sd})^{0.5}$; the latter condition is equivalent to $\varepsilon \equiv l\ \sqrt{N_{Sd}\ /EI} \leq 1$, where l is the member length.

b) frames with fixed bases, in which the sway failure mode involves plastic hinges locations in the columns at the fixed bases only.

The in-plane buckling lengths used for member design are those relevant for the *non-sway mode*; due allowance shall be made for the effects of plastic hinges.

It is usually necessary to check the resistance of the sections ; indeed the analysis is generally conducted without accounting for the interaction between compression, bending and shear.

Out-of-plane buckling of the members shall be verified too.

7. STRUCTURAL MODELLING OF CONNECTIONS.

With a view to perform a structural analysis, it is of primary importance to model the behaviour of the connections. That means the assumptions made regarding the structural response of the connections shall be consistent with the expectable behaviour of the latter. Usually conventional analysis and design of steel frames are carried out under the assumption that the beam-to-column connections are either fully rigid or ideally pinned. Of course such idealisations simplify drastically the analysis and the design procedures; however they require appropriate constructional detailing to ensure either full slope continuity between the adjoining members or no transfer of gravity moments from beams to columns. The validity of these assumptions may be questionable when the real connection response is intermediate between the fully rigid and ideally pinned cases.

The influence of so-called semi-rigid connections on the structural response results not only in influencing the distribution of internal forces but also in an increase of the frame drift and accordingly of the second-order effects.

Considering that a nearly pinned connection is more than a real hinge, some benefit can be drawn from the effects of practical restraint on a more economical beam design, on the one hand, and on the stability of the columns, on the other hand. In contrast, to make a joint fully rigid requires complex joint detailing, involving the use of thick end plates and of web column stiffeners. At present it is a paramount importance to save money and accordingly to simplify the joint detailing, with the result of possibly no more rigid joints. Last, some connection arrangements are such that the joints have really a semi-rigid behaviour.

Eurocode 3 has introduced provisions to allow designers for considering the behaviour of any type of connection in the design of structural frames. Connections are respectively classified i) by *strength*, and ii) by *rigidity*.

Regarding the rigidity, the connection is considered either as *nominally pinned*, as *rigid* or as *semi-rigid*.

Regarding the resistance, it is distinguished between the *full strength connection* and the *partial strength connection* according as the design resistance of the connection does or not exceed the design strength of the member connected.

Classification by rotational stiffness is governed by the rotation stiffness S_j of the connection, which shall be taken as the secant stiffness associated to the maximum design moment resistance of the connection.

The distinction between rigid and semi-rigid joints is first a matter of connection rotational stiffness. In this respect the classification boundaries are at present different according as the frame is braced or unbraced. Possibly there could be no objective reason to maintain such a distinction; some research work is in progress on this querry.

The connection behavioural response, in terms of moment - relative rotation, is actually nonlinear. To account for such a real behaviour in the global analysis is not convenient at all. Therefore, this response is idealised as elastic-perfectly plastic, similarly to what is done for member response. It is thus characterized by an initial rotational stiffness - that is a secant stiffness with respect to the real M-ϕ curve - and by a design bending resistance.

How to assess both characteristics of the connection behavioural response is depicted in Annex J [10] of Eurocode 3. Presently a revised Annex J [11], commonly termed Annex JJ, is being drafted based on the experimental and theoretical results obtained during the last decade; it should hopefully be substituted for Annex J. On base of this very valuable document, some actions have yet been undertaken - in which the department of the author is deeply involved - with a view to prepare design guidelines and design aids devoted to the designers. The latter will allow for a simple estimate of both joint stiffness and resistance on base of the sole geometric characteristics of the joint and of the mechanical properties of the material. Not only design procedures will be offered for most of the connection types, that will be applicable to any joint of this type, but also design tables will provide the designers with calculated values of joint rigidity and strength for a series of specified combinations of joint data and shapes.

Annex JJ and the subsequent documents especially aimed at practice purposes, will constitute, to the author's knowledge, the more extended and updated information on the subject. However, there are still items which are not yet fully or satisfactorily understood because of the lack of information. One of them is the rotation capacity of joints, which is of paramount importance for plastic analysis or when the frame has to resist seismic actions. To improve the design provisions in this respect requires experimental and theoretical investigations, which could reasonably be expected to start soon.

8. INTERACTION EQUATIONS FOR BEAM-COLUMNS.

As far as the global analysis is performed in such a way that stability is not directly and fully accounted for, it is necessary to check the members using the interaction equations in beam-column design. These equations provide a convenient means by which members subject to combined axial force and bending can be designed. In this process, the framework is first analyzed and the diagrams of internal forces in the members are drawn. The axial force and bending moments acting on a specified member are checked against the capacity of this member using interaction equations. The design is satisfactory when the interaction equation is satisfied.

Because most beam-columns do no exist as isolated members but as an integral part of a frame, the design of beam-columns in frames must account for the interaction amongst adjacent framing members. That is done by means of an effective length factor. Annex E [12] of Eurocode 3 provides practical methods of determining the effective buckling length factor for a compression member; these are based on Wood's charts, where the distribution factors are computed based on the response of a subassemblage composed of the continuous column - i.e. the specified compression member fitted with its respective extends to the adjacent upper and lower storeys - and the beams adjoining both compression member ends. According to [12], above charts may be substituted by analytical expressions (obtained from curve fitting) which provide slightly conservative estimates of the effective length factor. There is one chart for braced frames and another one for unbraced frames.

Account for P-Δ effect requires a second-order analysis which entails an iteration process. Aimed at design practice, simplified approaches have been developed.

The *member instability effect* is currently accounted for by using the relevant European buckling curves for the assessment of the buckling resistance involved in the axial force component of the interaction equations.

Regarding the *frame instability effect*, three situations may arise when checking the beam-column design:

a) This effect may be accepted as negligible and the bending moments are those obtained from the 1st order global analysis . The buckling length is the one relevant for the non-sway mode (braced).

b) The sway moments are factored by means of a single moment multiplyer - for instance, 1.2 -. Because of this rough approach, the buckling length shall be taken as the one relevant for the sway mode (unbraced); that means that the frame instability effects is reflected through both bending and axial force components of the interaction equations.

c) The sway moments are factored by a moment amplification factor $(1 - V_{Sd}/V_{cr})$, which characterizes the proximity of instability according to a sway mode. Accordingly the buckling length is taken as the one for the non-sway mode (braced); that means that the frame instability effect is brought on the bending component solely.

According to Eurocode 3 [3], the check of beam-columns with Class 4 cross-sections is slightly different from that of members with Class 1, 2 or 3, as a result of the effective cross-section to be considered for Class 4 sections. Then attention shall be paid to additional bending moments which are due to the axial force acting eccentrically because of the shift of the relevant centroïdal axis when Class 4 section is subject to uniform compression;

The maximum bending moment in the member does generally not occur at midspan. Instead of determining the exact location and magnitude of the maximum moment, use is made of the concept of *equivalent bending moment*. The latter allows to base the design of a beam-column subject to any combination of end moments (and possible moments due to lateral loads) on an equivalent beam-column subject to a pair of equal and opposite end moments.

In the more general format, the interaction equations for beam-columns according to EC3 [1] write:

$$\frac{N_{Sd}}{N_{ud,z}} + \frac{k_{LT}\,(M_{Sd,y} + N_{Sd}\,e_{Ny})}{\chi_{LT}\,M_{ud,y}} + \frac{k_z\,(M_{Sd,z} + N_{Sd}\,e_{Nz})}{M_{ud,z}} \leq 1$$

$$\frac{N_{Sd}}{\min\,(N_{ud,y},\,N_{ud,z})} + \frac{k_y\,(M_{Sd,y} + N_{Sd}\,e_{Ny})}{M_{ud,y}} + \frac{k_z\,(M_{Sd,z} + N_{Sd}\,e_{Nz})}{M_{ud,z}} \leq 1$$

They apply when lateral torsional buckling is respectively likely or not likely to appear. The numerators involve the design values of the axial N_{Sd} and of the peak bending moments $M_{Sd,y}$ and $M_{Sd,z}$ about both principal axes of the section. The denominators involve the design values of the buckling resistance $N_{ud,y}$ or $N_{ud,z}$ - for flexural buckling about y-y or z-z respectively - and of the bending resistances $M_{ud,y}$ and $M_{ud,z}$ about the relevant axes. Effects of additional moments due to shift e_N of the relevant centroïdal axis have, as said above, to be accounted for when Class 4 sections only. The reduction factor χ_{LT} for lateral torsional buckling should basically be different from the one χ for column buckling. However it has been demonstrated [13] that: i) the analytical expression of χ_{LT} can be taken, with a quite sufficient accuracy, similar to the one for χ, ii) values of χ_{LT} are deduced from European column buckling curves *a* and *c* according as the section is respectively rolled or welded.
The moment amplification factors k_y and k_z are obtained through expressions that are more complex than the current ones used in AISC-LRFD standard and in most European national

codes. They still depend on the moment distribution along the member but also on the member slenderness, on the shape factor of the cross-section and on the magnitude of the design axial force normalised with respect to the ultimate buckling load. The amplification factor is computed according to a process which involves the computation of secondary factors, which may be either positive or negative, and can be therefore source of misuses and errors; besides the physical background is, to the author's opinion, hidden by formal mathematical aspect.

The moment amplification factor k_{LT} for lateral torsional buckling is obtained in a way similar but not identical to k_y and k_z. That means the distribution of bending moments does not influence in a very same way both the loading and the resistance. In the past, European practice was generally to make this effect of moment distribution only reflected either on the resistance or - as most often - on the loading.

In Eurocode 3, the formulae are written for the general case of axial compression and biaxial bending. When combined compression and uniaxial bending, while lateral torsional buckling being not a potential failure mode, EC3 requires to use the minimum value χ_{min} irrespectively of whether or not the member is bent in the same plane as the critical column buckling plane. In contrast, the German code [14], though referring to the same formulae as EC3, specifies that χ_{min} shall reduce to the reduction factor for column buckling in the plane where the member is bent. This German practice - which, to the author's opinion, is by far more consistent - means that the interaction formula to be applied (when lateral buckling is prevented) controls the failure due to an excess of yielding in the plane of bending.

It has been shown [15] that, despite their higher sophistication, the EC3 interaction equations for beam-columns, compared to other standards, do not provide a significantly better agreement with bench marks obtained, for instance, from numerical simulations. Sometimes they can even lead to unduly conservative results. In this respect, the author feels allowed to regret that ECCS work [16] has not more inspired the drafting panel of Eurocode 3. Indeed the ECCS approach is less complex in its format, reflects the physical phenomena in a more comprehensive way and does not provide less accurate results than the provisions of Eurocode 3.

9. CONCLUSIONS.

The most updated European approach for frame analysis is the one described in Eurocode 3 - Part 1.1. Allowance is made as well for plastic as for elastic global analysis. This standard offers a wide range of possible methods going from the full second-order global analysis to simplified approaches for braced and unbraced frames ; of course the applicability of each of them is subordinated to the compliance with some criteria. The higher the level of sophistication of the method for global analysis, the more simple the subsequent checks to be made. The designer can thus balance the complexity either on the analysis or on the member (and section) checks, according to the available computational tools and means.

The presentation of Eurocode 3 should still be improved in some respects :

a) By presenting the available methods of analysis in a more perceivable way than the present written specifications. For instance, a layout, similar to the one of figure 1 of present paper, would be very useful to designers.

b) Without it be always clearly stated, some approximates suggested in EC3 are only valid for frames having horizontal beams. They are undoubtfully valid when slightly sloped rafters but there is no statement regarding the range of applicability.

c) Interaction formulae are established for members of uniform section; there is an urgent need for extending such formulae to fabricated tapered members, the use of which spreads out, mainly in industrial buildings.

d) Interaction formulae of EC3 should come back to a more simple and practicable format.

e) As far as plastic design is concerned, the rotation capacity is of primary importance; there is much work to be undertaken with a view to get a better knowledge in this field.

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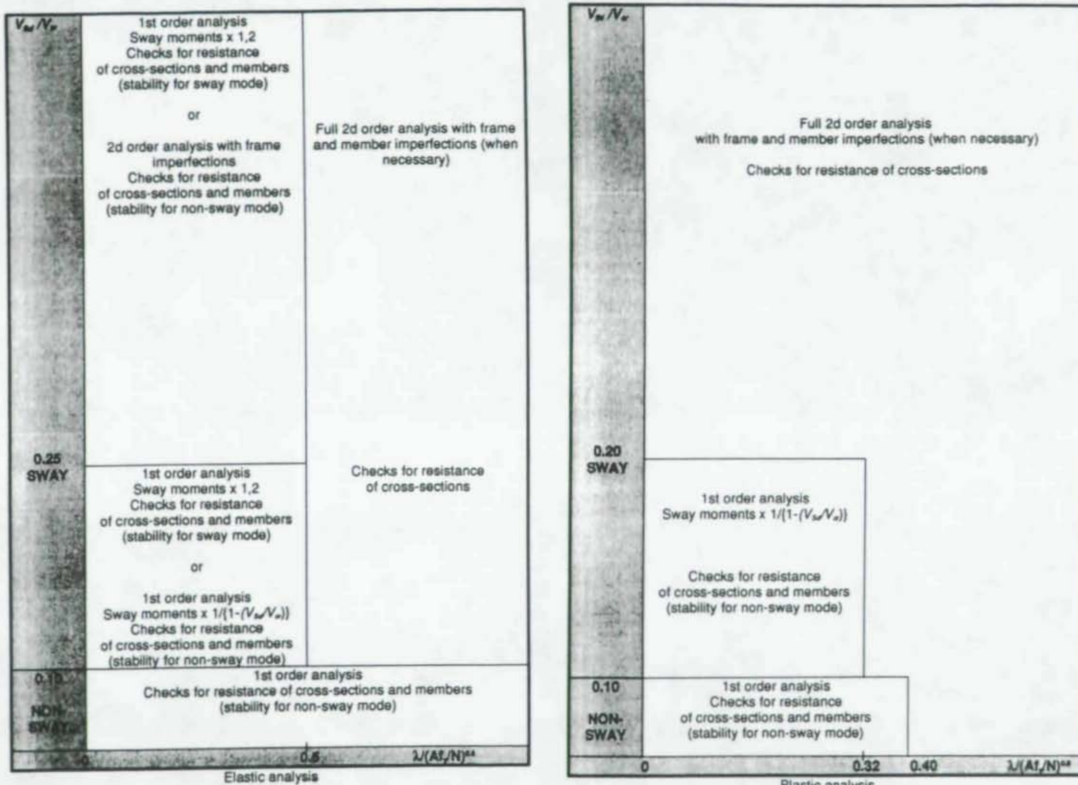

Figure 1 - Selection of a method for global analysis.

TUBULAR MEMBERS — LARGE AND SMALL

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ABSTRACT

Members of circular, hollow cross-section have a long history of use and wide application in civil engineering structures. In the smaller sizes, usually called hollow structural sections, improvements in connection design have greatly increased their use. Design methods for these members are reasonably well-defined. Design rules for manufactured or fabricated pipe of the kind used for transmission of fluids such as water, gas, or oil are not quite so well established, however. Likewise, there are still areas in the design of fabricated steel tubes of the large diameters used for stacks, masts, conveyor galleries, or in offshore structures that need further study. A overview of the state-of-the art in these several areas is presented.

1. INTRODUCTION

Members of tubular cross-section have been used for a very long time in civil engineering applications. The Forth Road Bridge in Scotland (Figure 1) and the Eads Bridge in St. Louis U.S.A. are examples of landmark structures that were fabricated using tubular members. In such examples, the tubular members were very large. They were, therefore, plated structures and this means that they required a great deal of labor to fabricate. Moreover, even when smaller sizes were used in this era, connection of the tubular members usually presented difficulties. With the

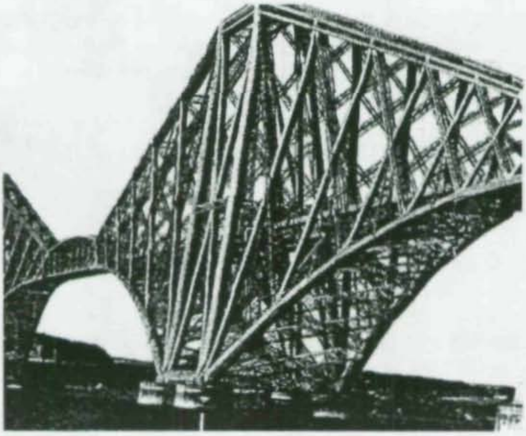

Figure 1 Forth Bridge

entry of hot-rolled steel I-shapes into the market, the use of tubular members went into decline. More recently, however, tubular members have regained popularity. In the case of what we call hollow structural shapes, efficient manufacturing processes and development of better connection details have led to a great increase in their use. They are aesthetically attractive, reduce the amount of surface exposed to corrosion, and in some instances offer advantages in structural capacity.

When large diameter tubes with relatively small wall thicknesses are required, the modern-day equivalent of the Forth Bridge, for example, fabrication is now by welding, not riveting, as was the case in the early structures. These members offer good structural capacity for their size and their circular shape is often aerodynamically advantageous in such applications as masts and offshore platforms. In the sizes required for such large tubes, ring stiffeners are almost always present and longitudinal stiffeners may also be required.

Intermediate between the relatively small sizes of the hollow structural shapes and the size of the members that are seen in offshore structures is the size range that is suitable for pipelines. Pipelines are not usually considered to be a structural element because they are not used in civil engineering applications such as buildings and bridges. However, the problems associated with their structural capacity are fundamentally the same as those for members in the other categories mentioned. Structural engineering research has the potential to contribute significantly to the resolution of the problems faced by the engineer responsible for the design of a pipeline.

The descriptions necessary for identifying the structural capacity of a tubular member should be independent of the arbitrary divisions that have been given above. However, they must necessarily reflect such important parameters as the residual stress pattern, level of imperfections, presence of stiffeners, and so on. Inevitably, the research in each area has tended to be done by specialists in that specific application, and the amount of continuity between the areas, while improving, is still weak.

The purpose of this paper is to present an overview of the stability problems associated with each of the tubular products described, give indications of the level of understanding of the structural behavior, and to identify particular areas of weakness and where structural engineering researchers can make a contribution.

2. HOLLOW STRUCTURAL SHAPES

Hollow structural shapes, otherwise called structural steel tubes, are manufactured as circular, square, or rectangular. Circular tubes are usually denoted as CHS and rectangular or square tubes as HSS. In each category manufacturing can be done in different ways, the overall distinction being hot-forming versus cold-forming. There are further manufacturing differences within each category. Hot-formed tubes can be seamless or formed with a continuous longitudinal seam that is groove-welded in the manufacturing process. A variation is to cold-form a sheet and seam weld it longitudinally with an automatic continuous welding process. Cold-formed tubes are manufactured with similar variations.

The manufacturing process affects the structural performance principally through the resultant residual stress characteristics, straightness, and variations in thickness. A good description of these and other features of hollow structural shapes is given in Reference 1.

In common with most structural members, the designer of a hollow structural shape must consider its member capacity in tension or in compression, local buckling strength of the cross-section, design of connections, and, when necessary, its fatigue characteristics. Tension member capacity is usually straightforward. If the type of connection employed does not connect all of the cross-section, then the principal complication is to properly identify the effect of shear lag. Capacity as a compression member likewise follows the conventional rules for column design and principally is affected by the residual stress pattern for the particular method of manufacturing used. The design rules for compression members in most specifications distinguish between hot-formed and cold-formed hollow structural members. It is considered that the design rules as presently constituted adequately reflect the member capacity in compression, particularly when two column curves are used, so that the cold-formed and hot-formed hollow structural shapes can be differentiated.

Connection design is an important feature of any successful design, but it probably requires more attention when dealing with hollow structural shapes. The connection details must deal with either tube-to-tube connections or tube to I-shaped (or other) sections. A great deal of investigatory work in these areas has been done in the past several decades. The extensive research work reported in technical papers and articles is now being followed by handbooks and other sources of information more suitable for designers [2, 3]. Presentation of information concerning fatigue of hollow structural members (controlled by the connection details in virtually all cases) is also becoming easier for designers to use [2, 4].

Local buckling of hollow structural sections generally follows the same philosophy as for local buckling of other cross-sections, that is, the axially compressed section must not fail by local buckling before it reaches its yield load capacity. This is obviously very conservative since the vast majority of columns are governed by buckling in either the inelastic or elastic range.

Consider the case of circular hollow structural sections. In North American practice, the basis of the limit for cross-section proportions (D/t) comes from an assessment of international test data done by the American Iron and Steel Institute in 1953 [5]. Using the non-dimensional parameter $\alpha = (E/F_Y)/(D/t)$, it was proposed by AISI that, for values of $\alpha \geq 2.23$, the critical local buckling stress be given by:

$$\sigma_{cr} = (0.0379\,\alpha + 0.667)\,\sigma_Y \tag{1}$$

In Equation 1, the value of α that will allow σ_{cr} to reach σ_Y is 8.79. Using this value and setting the modulus of elasticity at 29,000 ksi gives the limiting cross-section parameter $D/t \leq 3300/F_Y$ that is used in North American specifications. (Using SI units, the number 3300 becomes 23 000). It is interesting to note that the material constant used in this local buckling rule, F_Y, is in contrast to that used for most other structural sections, namely, $\sqrt{F_Y}$. More will be said about this later.

Similar limits on wall slenderness are required for square and rectangular hollow structural sections. In this case, most specification rules for local buckling of HSS sections derive from an investigation of local buckling based on the concept of effective width [1]. The classical elastic plate buckling equations are used as the starting point, but modifications for inelastic behavior, residual stress patterns and levels, and imperfections must be included. As summarized by Sherman [1], the result is

$$b_e/t \leq 1.91\sqrt{E/F_{max}}\left[1 - 0.381\sqrt{E/F_{max}}/(b/t)\right] \tag{2}$$

in which b_e is the effective plate width and F_{max} is the stress under the applied load. If the effective plate width, b_e, is set equal to the physical width of the plate, b, and F_{max} is set equal to the yield stress, F_Y, the result is:

$$b/t \leq 1.40\sqrt{E/F_Y} \tag{3}$$

If Equation 3 is used, the criterion again is that local buckling must not occur prior to attainment of the yield capacity of the cross-section. Substitution of the appropriate value of E in either the SI or US Customary units systems gives values for the b/t limit close to those currently given in North American specifications.

It is clear from a review of the background material (see Reference 1, for example), that the local buckling rules for both circular and rectangular or square hollow structural shapes could be improved. However, it is unlikely that such a review would result in significant changes to those limits already in force, and the effort may not be cost-effective. Overriding what is likely to be marginal improvements to the limits on cross-sectional slenderness is the decision by specification writers that buckling must not take place before the overall capacity of the member is reached. If this should change and local buckling and overall member strength are linked, then further examination of the data is probably warranted.

3. PIPE

An area of continuing research interest is that of the flexural capacity of members of circular hollow cross-section. Since a member of circular cross-section is seldom used as a flexural member in building construction, the main area of application is in their role as pipe, or, as the size increases, as stacks, masts, conveyor galleries, and so on. In this section, the flexural behavior of pipe will be discussed first. An extension into cases where the pipe is under combined loading—bending plus axial load plus internal pressure—will also be included.

There are extensive test data describing the bending strength of pipes. The majority of this data is from relatively stocky pipe, such as would be used for water mains, but some is from fabricated steel pipe of much larger size. Also, the data come from a variety of manufacturing styles—electric resistance welded, hot-rolled seamless, and fabricated. Thus, the residual stress patterns and level of initial imperfections will be different.

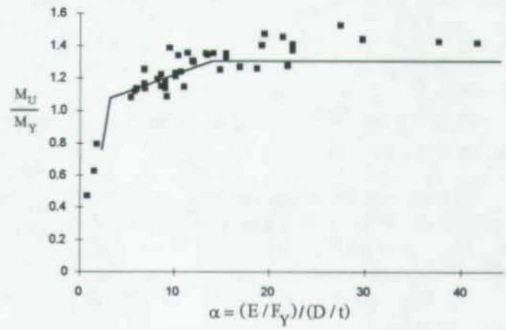

Figure 2 Pipe Bending Tests: Parameter α

Figure 2 shows the results of 42 tests from five different sources [6, 7, 8, 9, 10]. In this figure, the non-dimension moment, M_u/M_Y, is plotted against the same non-dimensional material and geometry parameter used in the development of the local buckling rule for hollow structural sections, namely, $\alpha = (E/F_Y)/(D/t)$. Also shown in the figure is the equation proposed by Sherman [1] to describe the maximum bending moment that can be carried by a pipe before it starts to unload by local buckling. Given that the shape factor for these pipes is very close to 1.30 for all cases, it can be observed that many of the stocky pipes can reach the plastic moment capacity. (In this figure and in Figure 3, which will be discussed following, the stocky pipe are those that plot to the right and slender pipe are those that plot to the left.)

The equations proposed by Sherman (which are given in terms of the plastic moment capacity rather than the yield moment capacity) are as follows:

$$M_u/M_p = 1 \qquad \text{for } \alpha \geq 14 \tag{4a}$$

$$M_u/M_p = (0.775 + 0.016\,\alpha) \qquad \text{for } 3.2 \leq \alpha \leq 14 \tag{4b}$$

$$M_u/M_p = 0.331\,\alpha/(Z/S) \qquad \text{for } 2.29 < \alpha \leq 3.2 \tag{4c}$$

Notice that these equations do not cover the region $\alpha < 2.29$, which is the region of slender cylinders. Thus, the three test specimens shown at the extreme left-hand side of data in Figure 2 are not covered by Equation 4. The plate buckling parameter D/t is much higher for these specimens than for the other data. (These three cylinders had values of α of 1.8, 1.5. and 0.8, had diameters of 1.53, 1.53, and 1.76 m, and the yield strengths were 376 MPa, 306 MPa, and 706 MPa). Appropriately, Equation 4 does not purport to cover the region where these test data lie, but the design equation does cover a fairly large region where there are no test data. In the writer's opinion, designers should not use Equation 4(c), and should only use Equation 4(b) for values of $\alpha > 5$.

A local buckling limit for cylinders in flexure can be developed from Equation 4. For example, for a cylinder to just reach M_Y, use of Equation 4(c) gives a value of $\alpha = 3.02$ for this case, and a corresponding local buckling limit $D/t \leq 9600/F_Y$ (using US Customary Units). For example, the Load and Resistance Factor Design Specification of the American Institute of Steel Construction [11] cites $D/t \leq 8970/F_Y$ for this case.

The same data as used in Figure 2 are shown again in Figure 3, except that this time the moment capacity is plotted against a parameter $\gamma = (E/F_Y)^{0.5}\,(t/R)^{1.5}$. A least-squares curve fitting shows a proposed design equation that uses this parameter [12]. Both the design curve shown in Figure 2 and that shown in Figure 3 fit the data about equally well for stocky cylinders (right-hand region of the plots). However, the design equation shown in Figure 3 captures the data for the three slender cylinders whereas the design curve in Figure 2 does not. It is anticipated that use of this design parameter will also give better results in the region where there presently are no test data.

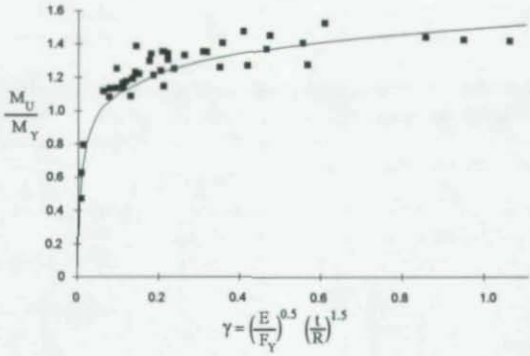

Figure 3 Pipe Bending Tests: Parameter γ

Both the parameter $\alpha = (E/F_Y)/(D/t)$ and the parameter $\gamma = (E/F_Y)^{0.5} (t/R)^{1.5}$ have some justification in theory. The former derives from the classical plate buckling equations. The latter was developed by Edlund [13] and is based on a panel strip theory for plate buckling. For the user of a specification or design rules, it really doesn't matter which is used as long as it is an adequate representation of the ultimate limit state being described. There does not seem to be any significant difference between the two models for the case of stocky cylinders, but the γ-form seems to provide better results for slender cylinders and for steels with high yield strengths.

When a pipe is used to carry fluids under pressure, such as is the case for pipelines carrying oil or natural gas, for example, more information than just the moment capacity based on local buckling is required. In this case, regulatory agencies typically place a limit on the maximum permissible circumferential stress. For example, the CSA specification [14] for design of pipelines requires that the maximum circumferential stress be limited to 80% of the specified minimum yield strength of the pipe material and the maximum longitudinal tensile strain be limited to 0.50%. The latter requirement is intended to protect against tensile fracture.

Work on the pipeline problem is going on at several locations throughout the world. A program being conducted by Murray [15] and his co-workers is typical. Both analytical studies and physical testing have been carried out. Test conditions are much more challenging than those for flexural testing of pipe, as described earlier. The pipeline tests must be able to accommodate axial load (both active and passive), bending, and internal pressure.

Figure 4 Tests of Twelve Inch Diameter Line Pipe

Figure 4 shows three 12–inch diameter pipe of 0.25 inch wall thickness. These were tested under conditions of constant axial load, variable uniform moment, and different cases of constant internal pressure. From right to left in the figure, these were zero internal pressure, one-half of the maximum design pressure, and maximum design pressure. (In this case, the maximum design pressure corresponded to that needed to produce a circumferential stress in the pipe of 72% of the specified minimum yield stress.) The zero pressure and maximum pressure cases refer, respectively, to conditions immediately upstream or immediately downstream of a pumping station. The figure also shows that a girth weld was located at mid-length of the test specimens.

In the unpressurized test, the local buckle forms in a diamond shape and no other wave or local distortion is observed at other locations. In the case of the pressurized tests, examination during the test reveals that several waves form, but, as the test progresses, one becomes dominant and the others damp out because the moment decreases after wrinkling starts.

In the tests described, the strain at which wrinkling first started was about 0.66%, which is significantly greater than the limit of 0.50% now used. A companion series, in which the test segment did not contain a girth weld, showed that strains can be as large as 1.1% before wrinkling takes place. The limit identified is associated with the onset of wrinkling, and it appears that once wrinkling starts, it acts as a kind of "fuse". Significantly, this means that, after wrinkling, strains on the tension side increase only slightly with increasing curvature. If this hypothesis is borne out by the physical testing and the analytical studies that are part of the continuing program, it means that the design limit might simply be the wrinkle size that limits the passage of the inspection equipment that pipeline operators move through the pipe periodically.

Analysis of the physical tests and parametric studies associated with the tests described (a total of 12 pipes of both 20 inch and 12 inch diameter have been tested) are now underway. The analysis is giving good results as compared with the physical tests, but the effort required to adequately predict the strength and deformation of a given pipe is considerable.

Work on this topic is continuing, both at the University of Alberta and elsewhere. It is illustrative of the class of problem to which structural engineers can make a contribution.

4. LARGE DIAMETER FABRICATED STEEL TUBES

Large diameter fabricated steel tubes are made by passing plate through rollers to form a circular shape and then making a longitudinal seam weld. Relatively short lengths result, and it is then necessary to take these "cans" and join them end-to-end using groove welds. The resulting tube (Figure 5) is used for conveyor galleries, stacks, masts, and so on. Ring stiffeners are almost always present, and longitudinal stiffeners may be present. If the loading is axial, the longitudinal stiffeners would be placed uniformly around the circumference, of course. If the loading is primarily flexural, the longitudinal stiffeners would be placed only in the compression zone. When used as a beam, typical diameters are in the order of 1.5 to 4 m and spans can range up to about 60 m. The issues are similar to those encountered in plate girder design—bending strength, shear strength, strength under combined bending and shear, and capacity of details need to be investigated.

The axial compressive strength of large diameter tubes, with or without longitudinal stiffeners has been investigated at a number of locations in recent years. Much of this work has been associated with the need to design structures for the offshore oil industry. Because of the large volume of work in this area, it will not be reviewed herein. Interested readers are directed to the summary paper by Harding and Dowling [16], where there is a good review of the topic and an extensive reference list.

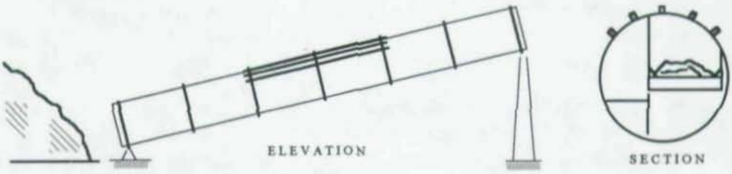

Figure 5 Large-Diameter Fabricated Steel Cylinder Used as a Conveyor Gallery

The problems associated with flexural capacity and shear capacity of large diameter tubes, including, as a benchmark, compressive capacity of large diameter fabricated unstiffened steel cylinders, are currently under investigation [12, 17, 18, 19, 20]. There are strong similarities between the elements required for plate girder design and those required for a large diameter fabricated steel cylinder. In each case, the designer needs information concerning the flexural strength, shear strength, interaction between shear and flexure, design of transverse and longitudinal stiffeners, and local problems such as reaction or load details.

The buckling strength of longitudinally unstiffened, fabricated steel tubes that are loaded axially is affected by dimensional parameters (length, radius, and thickness), material properties (modulus of elasticity and Poisson's ratio), and material and geometric nonlinearities. Current North American practice is to use the rules provided by the ASME code [21], which are a modification of buckling strength statements proposed by Miller [22]. They represent the classical buckling stress statement modified empirically by factors to account for imperfections and material nonlinearity. In the ASME code, no distinction is made between axially loaded and flexurally loaded members, that is, the capacity of a longitudinally unstiffened cylinder is taken as the same in compression and in bending.

A European source, ECCS [23] goes beyond this simplified approach and, in their manual on shell stability, higher buckling stresses are permitted in bending than in compression. This recommendation is supported by a substantial data base for small-scale cylinders failing elastically in either compression or flexure. The basis of the ECCS approach is the application of an empirical capacity reduction factor to the classical buckling equation followed by a further empirically-based reduction to account for inelastic behavior and residual stress effects should the elastic buckling stress exceed the assumed proportional limit of 50% σ_Y. The equations given in the ECCS document make a distinction between aerospace-quality tubes and fabricated tubes, and place an upper limit on the geometrical imperfections that can be present.

The work of Kulak et al. [12] shows that the test data on axially loaded cylinders from a wide variety of sources can be well represented by the function $\gamma = (E/\sigma_{YS})^{0.5} (t/R)^{1.5}$, which was used earlier (Figure 3) in the representation of local buckling of tubes in flexure. The result is shown in Figure 6. Details of the test data can be obtained from Reference 12, but they come from Wilson 1937 (□), Wilson and Newmark 1933 (○), Ostapenko et al. 1976, 1977, 1980 (◊), and Stephens et al. 1983 (♦ & •). The predictor curve shown in Figure 6 is as follows (where σ_U and σ_{YS} are the ultimate and static yield strengths, respectively):

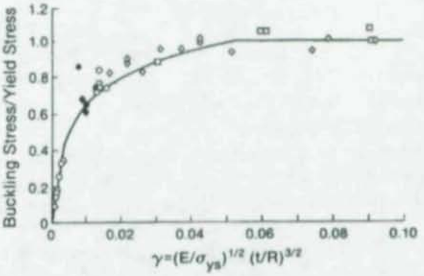

Figure 6 Proposed Local Buckling Strength Curve and Test Data – Axial Compression

$$\frac{\sigma_U}{\sigma_{YS}} = 119.3\gamma \qquad \text{for } \gamma \le 0.0036 \qquad 5(a)$$

$$\frac{\sigma_U}{\sigma_{YS}} = 1.625 + 0.489 \log \gamma \qquad \text{for } 0.0036 < \gamma < 0.0527 \qquad 5(b)$$

$$\frac{\sigma_U}{\sigma_{YS}} = 1.0 \qquad \text{for } \gamma \ge 0.0527 \qquad 5(c)$$

The flexural capacity of large diameter thin-walled fabricated steel cylinders that do not have longitudinal stiffeners is not much different from the corresponding axial load case. The first two parts of Equation 5 cover the region of slender cylinders. However, stocky cylinders can have considerably more capacity in flexure than they do in axial compression. As a consequence, Kulak et al. [12] suggested that Equations 5(a) and 5(b) be used for both axial compression and flexure, but that Equation 5(c) be replaced by Equation 6 for the bending case. (In fact, the predictor curve shown in Figure 3 is a three-part curve consisting of Equations 5(a) and 5(b) and Equation 6). Equation 6 is:

$$\frac{\sigma_U}{\sigma_{YS}} = 1.497 + 0.389 \log \gamma \qquad \text{for } \gamma \ge 0.0527 \qquad (6)$$

Some tests of large diameter thin-walled fabricated steel cylinders that have longitudinal stiffeners have been done at the University of Alberta [17]. Failure can be by shell buckling, in which shell plating buckles like an unstiffened cylinder between stiffeners. In this case, the longitudinal stiffeners remain essentially straight. A second possible failure mode is that of general buckling, in which case the shell and longitudinal stiffeners buckle together. This happens when the stiffeners are closely spaced. Lastly, failure may be initiated by buckling of the stiffeners themselves. The latter can easily be avoided by providing stiffeners with high torsional resistance.

Altogether, the problem is a complex one, although rules suitable for design are now being formulated [17].

Studies done on large diameter fabricated steel tubes that use only ring stiffeners show that shear force is carried in a manner very similar to that seen in thin-web plate girders. Before buckling, behavior is linearly elastic up to a proportional load level. Following this, the response then becomes gradually nonlinear until buckling takes place. The ultimate load can be reasonably estimated by an interaction equation proposed by Galletly and Blachut [24] or by a predictive formula provided by Obaia et al. [19] The finite element method can also be used to provide a *good estimate of the failure load and to simulate the deformed shape* [19].

After buckling, the carrying capacity of the two large-scale specimens tested by Obaia et al. dropped about 20%, after which it remained stable [18]. This suggests a significant secondary post-buckling capacity. The stability can be ascribed to the development of the tension field that develops in the direction of the diagonal buckles that form in the region between ring stiffeners.

In all cases examined, both from experimental data and from numerical analyses, the load versus deflection curves indicate that the post-buckling behavior is stable and that it occurs at a load level that is not greatly less than the ultimate strength. Furthermore, as shown by Roman and Elwi [25], although the ultimate strength is not much affected by the level of initial imperfections it is significantly influenced by the fabrication residual stresses. Those researchers also found that the postbuckling behavior is not affected by either initial imperfections or residual stresses. Therefore, it seems reasonable to use the postbuckling load, rather than the ultimate load, as a reliable and conservative basis for design. This has led to the development of a truss model for predicting the shear-carrying capacity of fabricated steel cylinders stiffened only by ring stiffeners.

The K-truss model developed in Reference 20 showed that the shear capacity is not very dependent upon the material parameter, E/σ_Y, but is significantly affected by the spacing of the circumferential stiffeners relative to the radius of the cylinder (R/L). Consequently, a practical value of $E/\sigma_Y = 600$ was selected as the basis of the nomograph shown in Figure 7. The figure shows values of V_{cr}/V_Y versus R/L for various values of R/t. The shear capacity curves are bounded by the shear yield capacity (i.e., $V_{cr}/V_Y = 1.0$) and by a prediction of failure of the cylinder by buckling due to bending. The latter, which has been calculated according to Equation 5, occurs for small values of R/L.

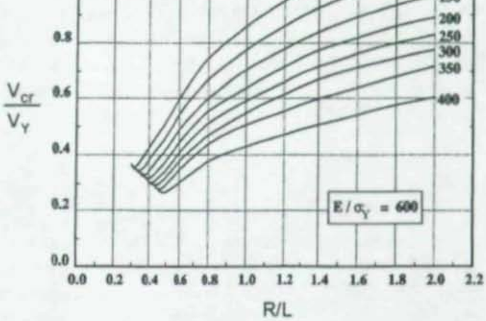

Figure 7 Nomograph for Shear Capacity of Large Diameter Tubes with Ring Stiffeners

When examining the results depicted in Figure 7, it should also be noted that failure by shear buckling is elastic–plastic in all cases. Purely elastic failure would only take place at values of R/L less than those necessary to cause failure of the cylinder by buckling due to bending. Examination of elastic buckling was found to be necessary only for R/t ratios larger than 400.

5. SUMMARY

Tubular members encompass a broad range of manufactured and fabricated elements. They may be square, rectangular, or circular in cross-section in the smaller sizes, and are usually made as a manufactured product in these cases. In the larger sizes, they are a fabricated product that is made using plate which has been rolled to the required size and shape. These sections are almost always circular in cross-section.

There is likewise a broad range of loading conditions. Similar to the standard structural sections, tubular members can be used as beams and columns or as the component members in trusses. They are also used for the storage of liquids (tanks) and for the transmission of fluids (pipelines). In these cases, the design process must include recognition of the internal pressure.

The design rules for the class of tubular members we call hollow structural shapes are well-established for their application as beams, columns, beam-columns, or as tension members. The rules for local buckling appear to offer the opportunity for refinement, but it is anticipated that the economy to be gained from more exact rules will be relatively small.

Pipe that is used for bending or is used to transmit fluids under relatively low pressures can be designed with confidence using the information that is currently available. Here again, there is scope for improvement in the local buckling rules, however. The behavior of pipe that is under relatively high pressure in addition to other loading conditions is not nearly so well understood. Availability of the modern methods of analysis (e.g., non-linear numerical formulations) in combination with physical tests of relatively large diameter pipe is improving the situation, but this is an area in which more work needs to be done.

Large diameter fabricated steel cylinders having relatively thin walls are used in a variety of applications. In the case of the type of member used in offshore platforms, which means that both circumferential and longitudinal stiffeners are usually present, there is a great deal of research information available. While more research is always helpful, it appears that assimilation and synthesis of the data into design rules is the most advantageous direction at the present time. None of this work has been reviewed in this paper.

The same type of product, that is, thin-walled large diameter fabricated steel cylinders, is seeing more use in industrial applications (stacks, masts, conveyor galleries, etc.). The flexural and shear strength of a cylinder that contains only ring stiffeners can be determined with a reasonable degree of accuracy. The interaction of these two has not been examined, however, nor has the design of the ring stiffeners themselves been adequately explored. The same problems in cylinders that use both ring stiffeners and longitudinal stiffeners (over a part of their cross-section) are currently under investigation.

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INFLUENCE OF CONSTRUCTIONAL DETAILS ON THE LOAD CARRYING CAPACITY OF BEAMS

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Abstract

Most specifications define rules for unsupported beams with simple supports. Both assumptions are very rare in practical design. Different types of constuctional details are discussed with regard to the load carrying capacity of beams.

1. General

Beam design is affected significantly if lateral torsional buckling can occur and must be taken into account. For an easy treatment the beam is usually assumed as an unsupported beam with simple supports. This means, that the beam is considered as an isolated member which can rotate freely about its longitudinal axis. At the supports lateral deflection and twist are prevented but no resistance is provided against either lateral bending or warping.
Most specifications define rules for unsupported beams. These rules vary in a wide range [1]. As examples the rules in the new German stability code DIN 18 800 part 2 [2] and for Eurocode 3 [3] are specified by eq. (1).

$$M_x \,/\, (r\, M_{px}) \le 1 \tag{1}$$

where
M_x maximum bending moment about strong axis under factored loads,
M_{px} full plastic moment,
r reduction factor.

This reduction factor in defined differently in both codes. In DIN 18 800 [2]:

$$r = \kappa_M = \left(1 \,/\, \bar{\lambda}_M^{2n}\right)^{1/n} \qquad \bar{\lambda}_M > 0{,}4 \tag{2}$$

$\bar{\lambda}_M = \sqrt{M_{px} \,/\, M_{cr}}$ nondimensional slenderness

M_{cr} elastic critical lateral torsional buckling moment

n = system factor, for distributed loads or moment gradient, = 2,5 rolled sections, = 2,0 welded sections - additional factor 0,8 for constant moment distribution

In EC 3 [3] :

$r = \chi_M =$ reduction factor (instead of κ_M), flexural buckling curve a for rolled sections, flexural buckling curve c for welded sections - using $\bar{\lambda}_M$, called χ_{LT}

Different statical systems like single span beams, continuous beams or cantilever can be taken into account by determination of the proper corresponding elastic critical moment M_{cr}. For the time being very little information is available whether the design curve depends also significantly on the statical system or other constructional details which can affect the support conditions. Therefore in all specifications the same design curve is used for different statical systems.

Actual conditions in constructions do not fulfil in any case these assumptions and must therefore be accounted for.

2. Influence of restraint by adjacent members

In reality unsupported beams as assumed in codes are very rare in practical design. All loads are introduced by neighbouring seperate members and their stiffness can be taken into account in much situations.

Adjacent members are normally present as individual members like cross beams or as floor elements like sheeting. The restraining effect of cross beams is mainly caused by its bending stiffness and to some extent by the joints. Contrary to this for sheeting commonly two restraining effects are present:

i) the horzontal deflection of the upper cord of the beam is prevented by the shear stiffness R of the sheeting, if fasteners between sheeting and beam are present,

ii) the bending resistance of the sheeting in combination with local deformations partly prevents the twisting of the beam.

Effect of horizontal restraints

If the entire compression flange of an I-beam is laterally fixed no lateral torsional buckling can occur. But in continuous beams the moment distribution is such that negative moments at the supports can lead to lateral torsional buckling although the strength of such a beam is much higher than for a laterally unsupported beam. This effect can also be seen by fig. 1, where values for the lateral torsional buckling parameter k are shown which were calculated by a computer program. The elastic critical lateral torsional buckling moment is then calculated by eq. (3).

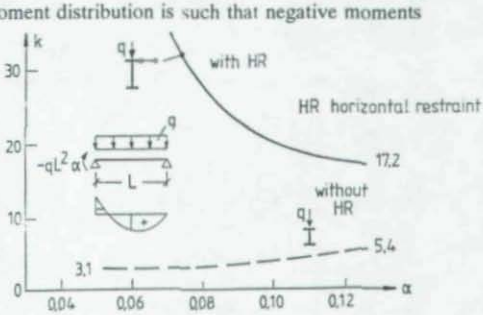


Fig. 1 Lateral torsional buckling parameter k for beams with and without horizontal restraint

$$M_{cr} = (k \,/\, L) \sqrt{GI_t \, EI_z} \tag{3}$$

where

I_t	torsion constant,
I_z	moment of inertial weak axis.

If floor elements are present the condition should be known if the upper flange of a beam can be assumed as horizontally fixed or not. In [2] a value for the neccessary shear stiffness R is given by eq. (4).

$$R \geq \left(EI_\omega \, \pi^2 \,/\, L^2 + GI_t + EI_y \, \pi^2 \, 0.25\,h^2 \,/\, L^2\right) 70 \,/\, h^2 \tag{4}$$

where

I_ω	warping constant,
h	depth of the beam,
70	medium value for different types of moment distribution.

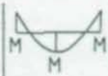
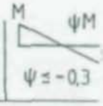
The effective shear stiffness R_e of trapezoidally corrugated sheeting can be calculated by the method of *Bryan* and must then be compared to eq. (4).

Effect of torsional restraints

A simplified stability check requires a minimum stiffness of the torsional restraint coefficient c_θ by eq. (5).

$$c_\theta \geq \left(M_{px}^2 \,/\, EI_z\right) k_\theta \tag{5}$$

Table 1 Factor k_θ for different moment distributions

top flange	moment distribution				
	 M	 M M	 M M M	 M	 M ψM ψ ≤ -0,3
free	4,0	3,5	3,5	1,6	1,0
restraint	0	0,12	0,23	1,0	0,7

The factor k_θ depends on the moment distribution and the fact whether the upper flange of the beam is horizontally restraint or not. Design values for k_θ can be seen from table 1, [2]. The values for c_θ were determined with regard to the demand that the load carrying capacity should reach 95 % of M_{cr} if c_θ is taken into consideration. A further extension of the load carrying capacity would need an extensive enlargement of c_θ which seems not to be neccessary.

The effective torsional restraint coefficient c_θ depends on the rigidity of the joint between the beam which should be stabilized and the structural element which acts as an adjacent member. For trapezoidally sheeting three deformation components should be taken into account as shown by eq. (6). For cross beams as adjacent members the influence of $c_{\theta A}$ may be negligable.

$$1/c_\theta = 1/c_{\theta M} + 1/c_{\theta A} + 1/c_{\theta P} \quad [kNm/m] \tag{6}$$

where

- $c_{\theta M}$ theoretical value with regard to the stiffness of the adjacent member only = $4\,E\,I_a/L_a$ for continuous sheeting, I_a, L_a effective moment of inertia, span of the adjacent member,
- $c_{\theta A}$ rigidity of connection, dependent on no. of screws, position of sheeting (inverted or not), sheet thickness, diameter of washers, thermal insulation,
- $c_{\theta P}$ distorsion of the beam investigated = $5770/(d/t_w^3 + 0.5\,b_f/t_f^3)$, dimensions in [cm].

The rigidity of connection is most important especially for trapezoidally sheeting. To account for different width of beams, it is useful to calculate the effective value by eq. (7).

$$c_{\theta A} = c_{\theta A}'(b_f / 100)^2 \qquad if \quad b_f \le 125\ mm \tag{7a}$$

$$c_{\theta A} = c_{\theta A}'(b_f / 100)1.25 \qquad if \quad 125 < b_f \le 200\ mm \tag{7b}$$

Values for $c_{\theta A}$ can be obtained by tests only. It was shown that a segment of a realistic constructed roof section may be used in tests for different types of loading and constructional details. Test configuration and conducting of the tests were described earlier, [5], [6].
Important parameters which influence the results significantly are ([4], [5], [6]):

- type of the roofing skin (e.g. depth and width of the trapezoidally sheeting, plate thickness),
- location and distance of the fasteners (ridge, trough, fasteners in every ($b=b_r$) or alternate ($e=2b_r$) ridge/trough),
- type of fasteners (normally selftapping screws diameter 6.3 mm),
- roofing construction (with or without intervening thermal insulation, type of thermal insulation),
- type of loading (applied live load, uplift loading by windload),
- magnitude of loading.

An example for one type of construction which was investigated in tests is shown in fig.2, an example for a moment-twist curve measured during the test is shown in fig. 3. From fig. 3 it can be seen that the stiffness related to a small value ϑ can be much higher than for the value $\vartheta = 0.1$ which was taken as an unfavourable reference value. All design values given in [2] and [4] are based on this unfavourable assumption of $\vartheta = 0.1$.

Design values are given in DIN 18 800 [2] and in literature, e.g. [4]. Values in [5] are not yet evaluated on the basis of eq. (7). Part 1.3 (Appendix A) to Eurocode 3 [3] mentions the torsional restraint coefficient too but offers a sligthly different test configuration and gives the same design values. Some values from [2] and [4] are given in table 2.

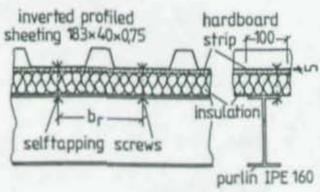


Fig. 2. Example for constructions investigated in tests

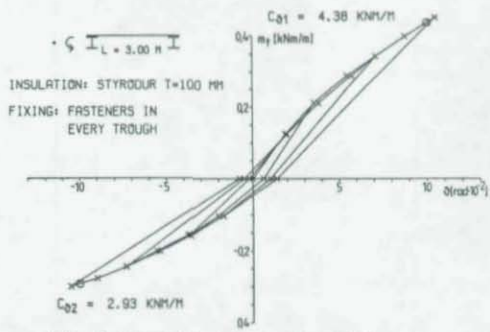


Fig. 3. Moment-twist curve from test

If the simplified stability check of eq. (5) is not used, the positive influence of the torsional restraint can be accounted for in another way too. In this situation the elastic critical moment M_{cr} can be calculated taking into account c_θ by calculating a torsional constant I_t^* instead of I_t using eq. (8).

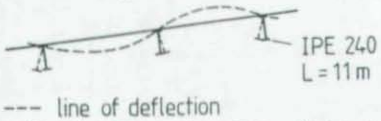
$$I_t^* = I_t + c_\theta\, L^2 \,/\, (G\, \pi^2) \tag{8}$$

This value I_t^* must then be introduced in a suitable formula, e.g eq. (3), or in a computer program.

Table 2. Torsional restraint coefficients for trapezoidally corrugated inverted sheeting, fasteners selftapping screws diameter 6.3 mm, applied live load

type of connection			$c_{\theta A}$
without thermal insulation:			
bottom chord,	$e = b_r$,	diam. 22 washer	3.1
bottom chord,	$e = 2b_r$,	diam. 22 washer	2.0
top chord,	$e = b_r$,		10.0
top chord,	$e = 2b_r$,		5.2
with thermal insulation 60 mm Styrodur 3000S:			
bottom chord,	$e = b_r$,	diam. 22 washer	4.7
bottom chord,	$e = 2b_r$,	diam. 22 washer	2.9
top chord,	$e = b_r$,		5.0
top chord,	$e = 2b_r$,		3.2

Numerical example



A roof is investigated with purlins of IPE 240 section as continuous beams, span L = 11 m, loaded by uniformly distributed load. The trapezoidally sheeting 40x183x0.75 mm (I_{ef} = 21.6 cm^4/m) is fastened at the bottom chord, $e=b_r$.

$$R = (21000 \cdot 0.0003739\ \pi^2/11^2 + 8100 \cdot 0.00129 + 21000 \cdot 0.0284\ \pi^2\ 0.25 \cdot 0.24^2/11^2)\ 70/0.24^2 \qquad = 14300\ \text{kN}$$

It must be shown additionally that the effective value R_e is greater than R.

$$c_{\theta M} = 4 \cdot 21000 \cdot 0.00216 / 3.0 \qquad = 60.5\ \text{kNm/m}$$

$$c_{\theta P} = 5770/(24/0.62^3 + 0.5 \cdot 12/0.98^3) \qquad = 53.9\ \text{kNm/m}$$

from table 2: $c_{\theta A}'$ = 3.1 kNm/m

eq. (7a): $c_{\theta A} = 3.1\ (120/100)^2$ = 4.46 kNm/m

eq. (6):

$$1/c_\theta = 1/60.5 + 1/53.9 + 1/4.46 \qquad c_\theta = 3.86\ \text{kNm/m}$$

eq. (5) and table 1:

$$c_\theta \geq (88.0^2 /(21000 \cdot 0.0284))\ 0.23 \qquad = 2.99\ \text{kNm/m}$$

Because the minimum stiffness requirement is fulfilled no further check with regard to lateral torsional buckling is neccessary.

Parameter studies

A continuous beam is investigated subjected to uniformly distribulted load, [1]. In order to calculate the moment capacity the design curve from DIN 18 800 is used [2]. For comparison the design curve of the British specification BS 5400 is taken into consideration, using the same elastic critical lateral torsional buckling Moment M_{cr}. The results are shown in fig. 4.

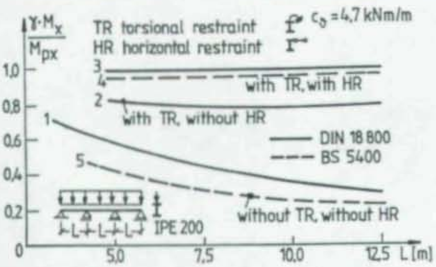


Fig. 4. Influence of restraints

From this fig. the following conclusions may be drawn:

i) compared to the beam without restraints (curve 1) the moment capacity increases significantly if the torsional restraint (curve 2) is taken into account,
ii) if additionally the horizontal restraint of the top flange is taken into consideration (curve 3) the moment capacity reaches the full plastic moment,
iii) the dependency of the moment capacity from the beam length becomes nearly unimportant if restraints are taken into account (curves 2 and 3),
iv) the differences between design curves of different specifications (curves 3 and 4, 1 and 5) decrease when restraints are taken into consideration.

3. Influence of end plates

3.1 General

In calculating the critical moment M_{cr} as shown before it is usually assumed that the end conditions correspond to simple support in the lateral plan. Thus lateral deflections and twist are prevented but no resistance is provided with regard to either lateral bending or warping. But practical constructional details frequently produce a stabilizing effect to the beam due to real support conditions.
End plate connections, often used in practice, see fig. 5, give an elastic warping restraint, which depend mainly on the thickness of the welded end plate.

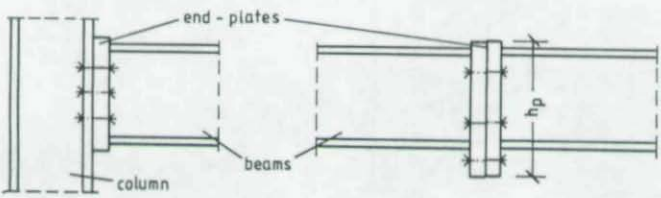


Fig 5. End plate connection

The effect of the end plate is taken into account by calculating the corresponding elastic critical moment $\bar{M}_{cr}$, see eq. (9). In addition to the solution already known the warping constant I_ω is replaced by I_ω / b_0^2 , see eq. (10).

$$\bar{M}_{cr} = \zeta \, N_{cr,z} \left(\sqrt{\bar{c}^2 + 0{,}25 \, z_p^2} + 0{,}5 \, z_p \right) \tag{9}$$

$$\bar{c}^2 = \frac{I_\omega / \beta_0^2 + 0{,}039 \, I_T \, L^2}{I_z} \tag{10}$$

$$\beta_0^2 = 1 - \frac{0{,}5}{1 + \dfrac{2\,EI_\omega}{c_\omega\, L}} \tag{11}$$

$$c_\omega = \frac{1}{3}\, G\, t_p^3\, b\, \left(h - t_p\right) \tag{12}$$

$$N_{cr,z} = \frac{\pi^2\, EI_z}{L^2} \tag{13}$$

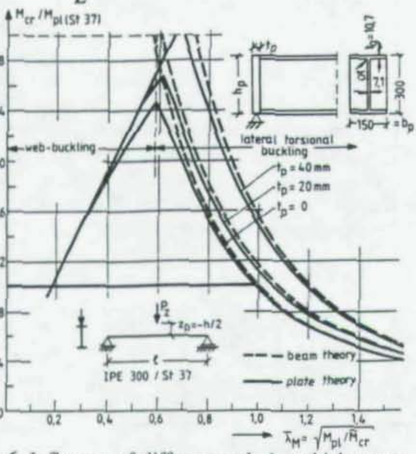


Fig. 6. Influence of different end plate thicknesses

From fig. 6 the influence of different end plate thicknesses can be seen, concerning the elastic critical moment. It may be recognized, that in the *region of small non-dimensional slendernesses* $\bar{\lambda}_M$ the web buckling of the profile governs. Therefore an additional check must be made with regard to the shear capacity of the web.

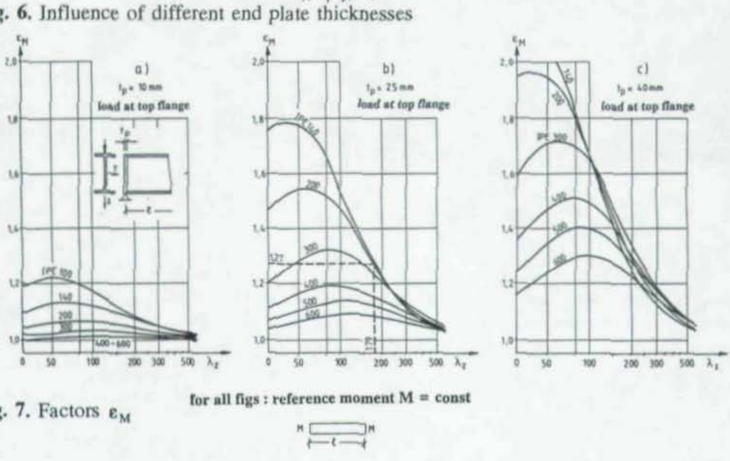


Fig. 7. Factors ε_M

for all figs : reference moment M = const

Other results are given by fig. 7, which may directly be used for design. The factor ε_M is given which is a multiplier for the elastic critical moment M_{cr} given e.g. by eq. (3). The factor ε_M is related to the elastic critical moment M_{cr} for constant moment distribution. Fig. 6 illustrates the following:

- the positive effect increases with increasing plate thicknesses,
- the positive effect is greater for the smaller profiles as IPE 140 than for higher profiles like IPE 600,
- the positive effect has a maximum in the region between $\lambda_z = 50$ and $\lambda_z = 100$.

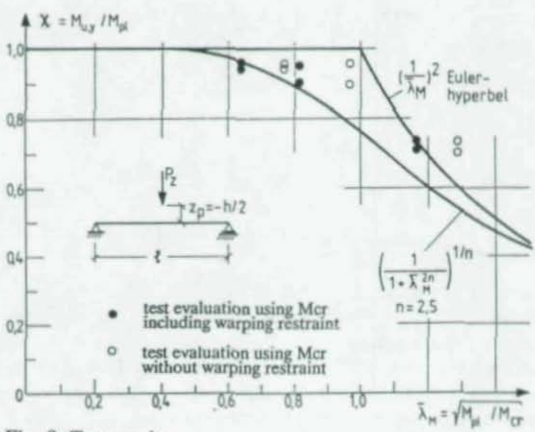
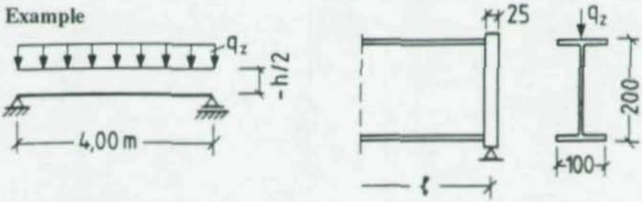


Fig. 8. Test results

In addition ultimate load calculations were done. There results fit quite well the assumed lateral torsional buckling curve of DIN 18 800 and EC 3. Furthermore tests were carried out. The results can be seen from fig. 8 and they fit sufficiently the ultimate load curve. Therefore the same procedure as for simple supported beams may be used for beams with end plates.

3.2 Example



IPE 200, end-plates $t_p = 25$ mm

$I_z = 142\ cm^4$; $I_t = 7{,}02\ cm^4$; $I_\omega = 12990\ cm^6$

$$c_\omega = 8100 \cdot 2{,}5^3 \cdot 10(20-0{,}85)/3 = 8{,}08\,kNm^3$$

$$\beta_0 = 1-0{,}5/(1+2 \cdot 2{,}1 \cdot 1{,}299/(8{,}08 \cdot 4{,}0)) = 0{,}572$$

From fig. 4.3-3: $\lambda_z = 400/2{,}27 = 179$; $\varepsilon_M = 1{,}27$

$$M_{cr} = 1{,}27\,\frac{\pi}{4{,}0}\,\sqrt{21000\cdot 0{,}0142\cdot 8100\cdot 0{,}000702} = 41{,}1\ kNm$$

$$M_{pl,k} = 52{,}6\ kNm$$

Due to EC 3 [3]:

$$\bar{\lambda}_{LT} = \sqrt{52{,}6/41{,}1} = 1{,}13 \quad ; \quad \chi_{LT} = 0{,}5757$$

$$M_{b,Rd} = 0{,}5757\cdot 52{,}6/1{,}1 = 27{,}5\ kNm$$

For comparison: without taking end plates into account:

$$M_{cr} = 32{,}3\ kNm \quad ; \quad M_{b,Rd} = 23{,}0\ kNm \qquad \Delta \sim 19\ \%$$

4. End notched beams

Floor beams in steel framed structures are often notched at the ends. The load carrying capacity may be reduced with regard to some effects:

- *the reduced cross section in the notched area has to carry the shear force and the* corresponding bending moment,
- if the web in the notched area is slender local buckling may occur,
- if the notched area is long the elastic lateral torsional buckling moment is affected by the reduced stiffness in this area,
- normally an interaction between local buckling and lateral torsional buckling is present.

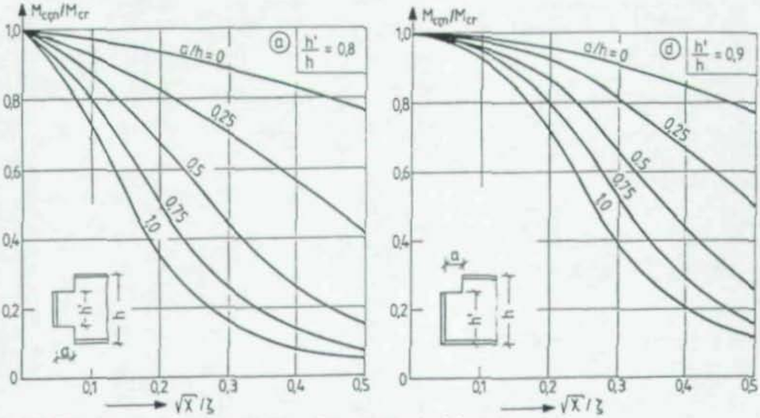


Fig. 9. Elastic critical moments $M_{cr,n}$ for end notched beams

Calculations on the basis of an energy solution show the great influence of the notched ends, see fig. 9. It can be seen that the elastic critical moment $M_{cr,n}$ for the notched beam can be significantly smaller than the usually used value M_{cr} without taking the notched ends into accout.

Some tests show this effect too, see fig. 10.

A proposal for the design demands:
- check of the plastic capacity in the notched area,
- check for lateral torsional buckling taking into account the reduced elastic critical moment $M_{cr,n}$.

Other results were given by *Cheng, Yura* and *Snell* [12], [13].

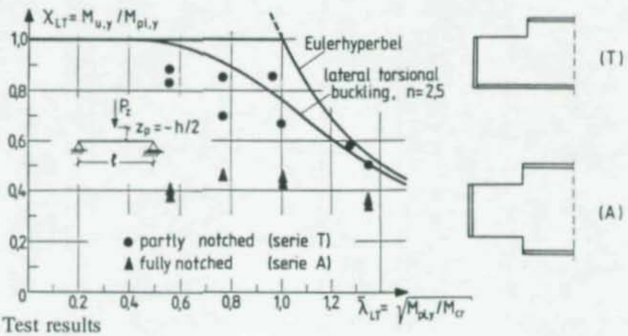


Fig. 10. Test results

5. End plates with reduced depth

In building structures, connections are commonly used which provide ease of design, fabrication and erection. In Europe end plate connections are widely used in beam to column attachment

If the end plate have a reduced depth simple support as usually assumed for in calculating the elastic critical lateral torsional buckling moment is not present.
Theoretical investigations were carried out on the basis of an energy solution taking into account plate deformations in the support area. The elastic critical moment ist then calculated by eq. (14), [9].

$$M_{cr,r} = K\, M_{cr} \tag{14}$$

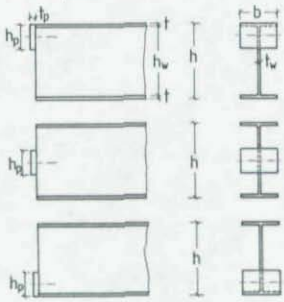
whereby

$$K_p = 1 - m\, W \tag{15a}$$

$$W = \left[EI_\omega \; / \; \left(L^2 \; GI_t \right) \right]^{1/2} \tag{15b}$$

$$\bar{K} = 1{,}76 \; (1 - K_p) \left(\frac{h_p}{h_w} - \frac{1}{3} \right) + K_p \tag{15c}$$

Information about the notation and some factors can be seen from fig. 11. The factor K_p may be chosen as 1.0 if the actual value of W is smaller than the limiting value W_L.



		level load application top $-\frac{h}{2}$	centre 0	bottom $+\frac{h}{2}$
	$\bar{a}$ =			
	m	1,0	0,6	0,25
	W_L	0,10	0,14	0,30
	m	1,25	1,0	0,8
	W_L	0,06	0,07	0,10
	m	1,5	1,25	1,0
	W_L	0,10	0,11	0,12

Fig. 11. Notation and factors for calculation

Additional test results showed the applicability of this method.

6. Conclusion

Some aspects of beam design with regard to lateral torsional buckling are discussed. Compared to the usual assumption of an isolated member with simple support some constructional details may increase the load carrying capacity, others may decrease it. In order to have a design close to riality the actual constructional details should be taken into consideration.

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LATERALLY UNSUPPORTED BEAMS

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1. INTRODUCTION

A beam which is bent in its stiff principal plane may buckle out of this plane by deflecting laterally and twisting, as shown in Fig. 1, especially when there are insufficient lateral restraints. While this subject has been studied for more than 100 years and an extensive body of literature has developed, there still remain many challenging problems that require solution. This paper makes a brief assessment of the present state of knowledge, and then outlines a number of areas where research is incomplete.

The present state of knowledge of the important influences on the elastic buckling of laterally unsupported beams is first summarised, followed by theoretical and experimental research on the strengths of real beams, and then by methods of designing against lateral buckling.

The areas where knowledge is incomplete are outlined under a number of headings, including cross-section types, materials, structural forms, combined loading, analysis, experiment, design, and education.

The purpose of a survey such as this is not only to draw attention to areas which need further research, but also to provoke other researchers to draw attention to completed research work and to unexplored research areas that have been omitted from this paper.

2. PRESENT STATE OF KNOWLEDGE

2.1 ELASTIC BUCKLING

2.1.1 General

The elastic flexural-torsional buckling of beams has been extensively studied. The first treatments were published by Michell (1899) and Prandtl (1899), who considered the lateral buckling of beams of narrow rectangular cross-section. Their work was extended in 1905 by Timoshenko (1953a, b) to include the effects of warping torsion in I-section beams.

Subsequent work in 1929 by Wagner (1936) and later work by others led to the development of a general theory of flexural-torsional buckling, as stated by Timoshenko (1953c) and Vlasov (1961), and incorporated in the textbooks of Timoshenko and Gere (1961) and Bleich (1952).

Specific studies of flexural-torsional buckling were made by many researchers, but prior to the 1960s, these were limited by the necessity to make extensive calculations by hand. Some of these are referred to in a survey by Lee (1960).

This situation changed dramatically with the advent of the modern digital computer, and the 1960s saw an explosion in the amount of published research. As a result, the focus of research moved from the flexural-torsional buckling of isolated members under various loading conditions to the effects of end and intermediate restraints (Trahair and Nethercot, 1974) exerted on a member of a rigid-jointed frame as a result of its continuity with adjacent members and braces. Many of these studies are summarised in the survey of the Column Research Committee of Japan (1971) and in a recent textbook by Trahair (1993a).

The extension of the general finite element method of structural analysis (Zienkiewicz and Taylor, 1989, 1991) to flexural-torsional buckling problems by Barsoum and Gallagher (1970) saw a further change, in that it was no longer necessary to publish comprehensive results of elastic flexural-torsional buckling studies, since almost any particular situation could now be analysed using a general purpose computer program. This development is similar to that which occurred for the in-plane analysis of plane rigid-jointed frames, in which the tabulations of solutions used in the 1930s were replaced by general purpose plane frame computer analysis programs.

Many of the developments of the theory of flexural-torsional buckling have been made by extensions of the previously accepted theories, as expressed either by the differential equations of elastic bending and torsion or by the energy equation for buckling (Bazant and Cedolin, 1991). Not all of these extensions have received general acceptance, and so a number of attempts have been made through the 1980s to produce a generally acceptable theory of flexural-torsional buckling. Such a general theory which is based on the use of the second-order relationships between the deformations and strains that take place during bending and torsion, the concept of the total potential, and the principles of virtual work and equilibrium, and of conservation of energy during buckling was published by Trahair (1993a).

It can now be said that the influences of cross-section, length, moment distribution, load height, restraints and supports are well understood, and that the elastic buckling of any member and most frames can be accurately predicted, either by recourse to summaries of reported solutions, or by the use of general purpose computer programs.

2.1.2 Bracing

There have been many studies of the effects of intermediate restraints on lateral buckling, and some of these are summarised by Trahair and Nethercot (1984) and Trahair (1993a). These intermediate restraints are provided by bracing which may be elastic or effectively rigid, continuous or discrete, and may restrain lateral deflections u, twist rotations ϕ, lateral rotations u', or warping displacements proportional to ϕ', as indicated in Fig. 2 (Trahair, 1979).

2.1.3 Interaction of local and flexural-torsional buckling

Elastic local buckling of a very thin compression flange may significantly reduce the resistance of a beam to flexural-torsional buckling (Cherry, 1960). In the case of uniform bending, local buckles appear along the whole length of the compression flange, and even though local failure is postponed by the flange post-local buckling resistance, the flexural and torsional stiffnesses of the flange are reduced, so that the effective out-of-plane rigidities EI_y, GJ, and EI_w of the beam are also reduced along the whole length of the beam (Bradford and Hancock, 1984). The reduced resistance of the beam to flexural-torsional buckling can be predicted by transforming it into a monosymmetric beam which has these reduced rigidities.

The interaction between local and flexural-torsional buckling for other than uniform bending has been studied (Wang, Yost, and Tien, 1977). However, flange local buckling is then confined to the high moment regions, and its effect on flexural-torsional buckling is reduced.

When the elastic local buckling load of the compression flange is close to the elastic flexural-torsional buckling load, the strength may be reduced by imperfection sensitivity effects. Some small reductions were reported by Menken, Groot, and Stellenberg (1991) for thin-flange beams in uniform bending. In practice, very thin flanges and uniform bending rarely occur, and it is unlikely that significant strength reductions will occur.

However, it is not uncommon to use welded beams with flanges of intermediate slenderness, so that the local buckling strength is of the same order as the inelastic flexural-torsional buckling capacity. An experimental study (Kubo and Fukumoto, 1988) of simply supported steel beams with central concentrated loads showed that their strengths could be closely predicted by substituting a reduced cross-section moment capacity for the full plastic moment capacity when determining the flexural-torsional buckling capacity, the reduced section moment capacity allowing for the effects of yielding and local buckling and being based on test results.

2.1.4 Web distortion

It is usually assumed in the analysis of flexural-torsional buckling that there is no change in the shape of the cross-section during buckling. Web distortion (Fig. 3b) may significantly reduce the flexural-torsional buckling resistance. Flexural-torsional buckling with web distortion is often described as distortional buckling.

Web distortion allows larger deflections of the critical flange than usual. The resistance to buckling is reduced through a corresponding reduction in the strain energy stored during buckling caused by reduced flange twists, as is especially the case for hollow flange beams (Heldt and Mahendran, 1992) whose flanges have high torsional stiffness (Fig. 4). Web distortion may occur in beams with flexible webs, due either to their thinness or to holes. It also occurs in beams whose compression flanges are unrestrained at the supports (Bradford and Trahair, 1981) as shown in Fig. 3d, near the supports of continuous composite beams (Johnson and Bradford, 1983) as shown in Fig. 3e, and in trough girders (Seah and Khong, 1990), as shown in Fig. 3f.

The distortional buckling of beams in uniform bending has been analysed (Hancock, 1978) by using the finite strip method (Cheung, 1976). A simple approximate solution has been obtained for I-section beam-columns in uniform bending by assuming that the web distorts in a cubic shape (Hancock, Bradford, and Trahair, 1980). This assumption has also been used to analyse beam-columns of general thin-walled open cross-section under general loading conditions (Bradford and Trahair, 1981, 1982). This method has been applied to a wide range of situations and extended to inelastic buckling problems. A recent study (Chin, Al-Bermani, and Kitipornchai, 1992) has used a super finite element to include the effects of local, distortional, and lateral buckling. An extensive review is given by Bradford (1992).

2.1.5 Pre-buckling deflections

The usual method of analysing flexural-torsional buckling neglects the effects of the pre-buckling deflections in the plane of bending, which increase the buckling resistance. For a simply supported beam in uniform bending, the increased elastic buckling resistance is given by

$$M = \frac{M_{yz}}{\sqrt{[(1 - EI_y / EI_x)\{1 - (GJ / 2EI_x)(1 + \pi^2 EI_w / GJL^2)\}]}}$$

in which

$$M_{yz} = \sqrt{\{(\pi^2 EI_y / L^2)(GJ + \pi^2 EI_w / L^2)\}}$$

is the classical resistance.

The pre-buckling deflections transform the beam into a 'negative' arch. The concave curvature of the deflected beam increases its buckling resistance, just as the convex curvature of an arch decreases its buckling resistance. The resistance of the beam in uniform bending increases with the ratio I_y / I_x, and theoretically becomes infinite when $I_y = I_x$. This is consistent with the common assumption that a beam with $I_y > I_x$ does not buckle, because it finds it easier to remain in the more flexible plane of bending than to buckle out-of-plane by deflecting in the stiffer plane. (Note that this common assumption does not always hold true, as for example when loads are applied well above the shear centre axis, which may lead to buckling in a dominantly torsional mode).

Early work on the effects of pre-buckling deflections were reviewed by Trahair and Woolcock (1973) who considered simply supported beams in uniform bending. This was extended to a wide range of load, support, and restraint conditions by Vacharajittiphan, Woolcock, and Trahair (1974). Finite element methods of analysis have been developed by Roberts and Azizian (1983) and Pi and Trahair (1992a, b).

2.1.6 Post-buckling behaviour

Slender statically determinate beams which remain elastic have slowly rising load-deformation paths after flexural-torsional buckling (Woolcock and Trahair, 1974), as shown in Fig. 5. This behaviour is similar to that of the behaviour of slender columns after flexural buckling. The increased load carrying capacity over that predicted by the small deflection theory of elastic buckling is associated with disturbing effects at large deflections which are not as large as those predicted by the small deflection theory.

However, slender redundant beams (Masur and Milbradt 1957, and Woolcock and Trahair 1975, 1976) may have significant increases in load capacity above those predicted by the small deformation theory of flexural-torsional buckling. The twist rotations that occur after initial buckling reduce locally the effective bending rigidity in the plane of loading, which causes a redistribution of the bending moments. In continuous beams, the moments near the untwisted regions at the supports increase, and play a greater part in resisting the applied load.

The buckling resistance is generally affected by the bending moment distribution, and in this case the redistribution is favourable, and increases the buckling resistance. The buckling resistance may asymptote towards a limiting value (Fig. 5), but then may continue to increase, but only slowly, as in the case of statically determinate beams.

Significant increases in strength after post-buckling are only realized in slender rectangular beams or in very slender I-section beams. Yielding effects usually cause practical beams to fail before any significant post-buckling reserve can be achieved (Woolcock and Trahair, 1976).

2.2 STRENGTH

While the historical development of knowledge of flexural-torsional buckling undoubtedly was initiated by the need to prevent premature failure of steel structures in this mode, this is not well documented. It seems likely, however, that early design procedures for preventing the lateral buckling of steel beams followed and were closely related to those used for preventing the flexural failure of columns.

The need to be able to design against flexural-torsional buckling was the catalyst for the development of a theory for flexural-torsional buckling which would allow the successful prediction of failure. Early theoretical research was into the elastic buckling of perfectly straight members, some of which was verified experimentally. However, the very straight and slender members used for these experiments were unrepresentative of the real steel beams used in practice, tests of which showed that their strengths were reduced below those predicted solely by elastic buckling theory.

Theoretical research therefore extended from the elastic buckling of straight members to study the influences of crookedness, yielding, and residual stresses on the strengths of real steel beams, and to determine how to incorporate these into the procedures used in design. These developments tended to follow behind the corresponding developments from the elastic flexural buckling theory to the strengths of real steel columns.

Experiments on the lateral buckling of real steel beams were reviewed by Fukumoto and Kubo (1977a, b, c) who produced a data base of the experimental studies prior to 1977.

At present, realistic models have been developed of the stress-strain behaviour, residual stresses, and initial crookednesses and twists in real members (Bild, Chen, and Trahair 1992, Pi and Trahair 1993a) and are being applied to an increasing range of problems (Pi and Trahair, 1993b, 1994) to predict theoretically the strengths of real beams. There have been reasonably extensive series of tests on the strengths of hot-rolled and welded steel I-beams.

2.3 DESIGN

Early rules for designing steel beams against lateral buckling were generally transpositions of rules for designing columns against flexural buckling, with perhaps the first proposal based on flexural-torsional buckling being made by Timoshenko (1924), and the first modern treatment by Kerensky, Flint and Brown (1956). More recently, most countries have or are transforming their design standards into the limit states format (Ravindra and Galambos, 1978).

Modern design codes such as SA (1990) give increased guidance designing against lateral buckling, including methods of assessing the effectiveness of bracing and the forces that braces transmit. Current design criteria for steel beams (AISC 1986, AISI 1991, BSI 1990, CEC 1990, CSA 1989, SA 1990) have been reviewed by Galambos (1988), Beedle (1991), Trahair and Bradford (1991), and Trahair (1993a).

3. RESEARCH NEEDS

3.1 CROSS-SECTION TYPES

While the effects of different cross-section types on elastic buckling are well understood, most studies on inelastic buckling and the effects of residual stresses and initial crookedness and twist on the strengths of real beams have been limited to steel members of equal flange I-section.

There is a need to extend this research to other cross-sections, such as hollow rectangles, hollow flange beams, channels, angles, and unequal flange I-sections (Fig. 4). These different sections may well have different residual stresses and initial crookednesses and twists, and so different strength formulations may be required, leading to the development of multiple beam curves to complement the multiple column curves already in use. Already a study (Pi and Trahair, 1994) has been made of cold-formed rectangular hollow section beams, and a different strength formulation proposed.

3.2 MATERIALS

Similarly, research on inelastic buckling and the effects of residual stresses and initial crookedness and twist on the strengths of hot-rolled steel beams needs to be extended to other materials such as *cold-formed steel, stainless steel, and aluminium. These have different stress-strain curves,* and different residual stresses and initial crookednesses and twists arising from their different methods of manufacture, and can be expected to require different strength formulations. A recent survey of research on the lateral buckling strengths of cold-formed beams (Trahair, 1994) found that design formulations are based on those of hot-rolled columns and that there are very few theoretical or experimental data that can be used to assess the adequacy of these.

3.3 STRUCTURAL FORMS

While there has been extensive research into the elastic buckling of simple, continuous, and braced beams, cantilevers, and beam-columns (Trahair, 1993a), there has been less work done on some other forms, including pitched roof portal frames, multi-storey multi-bay frames, and arches (Fig. 6).

The multiplicity of variables makes research into specific frame arrangements somewhat daunting and perhaps purposeless. More attractive is the development of computer programs capable of predicting flexural-torsional buckling in frames, but difficulties arise in determining the warping continuity conditions at joints (Fig. 7 and Vacharajittiphan and Trahair, 1974). Another possibility is to develop approximate methods of predicting frame buckling by assessing the restraining actions of adjacent members on the most critically loaded member, possibly by extending the method of Nethercot and Trahair (1976) for braced and continuous beams.

Recent research on arches (Yang and Kuo 1986, Trahair and Papangelis 1987, Papangelis and Trahair 1987a, b, c, 1988, 1991, Kuo and Yang 1991, and Yang, Kuo, and Yan 1991) has concentrated on the theoretical prediction and test verification of elastic buckling. There appears to have been very little research into the effects of residual stresses and initial crookedness and twist on the strengths of real arches, and appropriate design methods have been lacking or largely speculative (Papangelis and Trahair, 1993).

Similarly, research work on bracing has concentrated on its effects on the elastic buckling of the beam it restrains, but there is little information available on the forces developed in the bracing itself due to its connection to real beams with initial crookednesses and twists. Current research on bracing includes extensive studies of the bracing effects of roof sheeting on purlins under wind uplift conditions (Rousch et al, 1993).

3.4 COMBINED LOADING

In many cases, beams are subjected to out-of-plane loadings in combination with their in-plane loadings. Examples include the eccentric loading of channel-section beams (Fig. 8) and angles which induces primary torsion actions, and the biaxial bending of zed-section beams (Fig. 9) or angles by loading acting in the plane of the web or leg.

The biaxial bending of hot-rolled I-section beam-columns has been extensively investigated (Lindner 1974a, Chen and Atsuta 1977), but most studies have been concerned with biaxial bending produced by eccentric compression, and only in a few cases has the biaxial bending of beams caused by inclined transverse loading been considered (Lindner 1971, Pi and Trahair 1993b). Each component of an inclined load will produce a transverse deflection which combines with the other component to induce a secondary torque. First yield strength formulations based on second-order elastic analyses of residual stress free beams (Roik, Jurgen, and Linder, 1972) significantly underestimate the inelastic strength, and so theoretical analyses need to include the effects of residual stresses and yielding as well as second-order effects. There appears to be very little reliable experimental evidence with which to test theoretical predictions.

Beams with transverse loads acting eccentrically have small to moderate primary torsion actions in addition to the primary bending actions. There is surprisingly little knowledge of the torsional strengths of real steel beams, apart from the work of Farwell and Galambos (1969) who showed experimentally that very large rotations could occur before failure occurred by tensile fracture at the flange tips initiated by Wagner stresses. Recently, Pi and Trahair (1993c) extended the Merchant approximation for torsional plastic collapse (Dinno and Merchant, 1965), and proposed the use of this as a "strength limit based on the onset of significant rotations".

For a beam under eccentric loading, the torsion action will produce a twist rotation which combines with the primary bending action to produce a secondary bending action. Again, first yield formulations significantly underestimate the inelastic strength. Lindner (1971, 1974b) and Pi and Trahair (1993d) have made theoretical studies of the strengths of steel I-beams under combined bending and torsion caused by eccentric loading by making analyses which include the effects of material and geometric non-linearities. Again, there appears to be very little reliable experimental evidence with which to test theoretical predictions.

Other examples of beams subjected to combined loading include beams curved in plan (Galambos, 1988). For these beams, the curvature of the beam causes vertical loads to induce large primary torsion actions in addition to the primary bending actions. The curvature of the beam causes added complications for the elastic analyst, as there are curvature terms in the bending and torsion equations which are additional to those of the straight beam. These additional terms are not easy to formulate correctly, as recent experience (Papangelis and Trahair, 1987a) with formulations for the elastic lateral buckling of arches has shown.

It can be expected that the prediction of the elastic deformations of curved beams will require careful analysis which includes geometrical non-linearities and the effects of large twist rotations. The theoretical predictions of the inelastic strengths of curved beams will be even more difficult, while the development of design approximations is likely to be delayed until experimental confirmation can be obtained for the theoretical predictions.

3.5 ANALYSIS

While there are user-friendly computer programs available (Papangelis, Trahair, and Hancock, 1993) for analysing the elastic buckling of simple and continuous beams, cantilevers, and beam-columns, there is a need to extend the application of these. Extensions might allow their application to frames, arches (Papangelis and Trahair 1987b, 1991), and tapered members (Nethercot 1973, Kitipornchai and Trahair 1972, 1975, Bradford and Cuk 1988, and Bradford 1988a, b, 1989), in which case existing research programs (Hancock and Trahair, 1978) might be easily adapted. The most difficult problem associated with these extensions is the question of warping and distortion continuity at joints between members (Fig. 7 and Vacharajittiphan and Trahair, 1974). Some different approaches to this have been summarised by Trahair (1993a).

For research purposes, computer programs have been developed (Pi and Trahair, 1993a, b) for the inelastic strength analysis of steel beam-columns with elastic-plastic-strain hardening stress-strain curves, residual stresses and initial crookedness and twist. These programs tend to consume both storage space and time, even for simple structures, and are usually quite unfriendly. As more powerful computers become more economically viable for practising engineers, there will be a need to convert these into user-friendly formats. These programs will then be able to be used for the advanced analysis of laterally unbraced compact members in steel structures, in the same way as is now permitted for laterally braced compact members (SA, 1990). When advanced analysis is used, it is only necessary that the structure be able to maintain an equilibrium position under its design loads. This method removes all the complications of the lateral buckling rules of present design codes.

For members which are not compact, local buckling reduces the section capacity. Current research on the interaction between local and in-plane buckling (Hancock et al 1990, Rasmussen and Hancock 1991, and Key and Hancock 1993) aims to extend the method of design by advanced analysis from structures with laterally braced compact members to structures with other laterally braced members. Future research on the interaction between local and lateral buckling will aim to further extend advanced analysis from structures with laterally unbraced compact members to other laterally unbraced members (White, Liew, and Chen, 1993).

3.6 EXPERIMENT

While there have been many experiments made to verify elastic flexural-torsional buckling predictions, and reasonably extensive series of tests on the strengths of hot-rolled and welded steel I-beams, there are a number of other areas where test data is sparse. There is generally a need to carry out experiments on real beams of other materials than hot-rolled steel and of other than I-section. Experiments are also needed on tapered beams and arches, and on the biaxial bending and torsion of beams with inclined or eccentric loading and of curved beams.

Experimental reports need to document the test conditions carefully. Information is needed which defines the stress-strain curves, residual stresses, initial crookedness and twist of the test specimen, the load eccentricity and inclination, and any restraints induced by the supports and braces, as well as the geometry of the test specimen and the maximum load reached and the failure mode.

3.7 DESIGN

While methods are available for the design of a wide range of unbraced beams against lateral buckling, some of these are rather primitively based on extensions of design methods for the flexural buckling of hot-rolled steel I-section columns, and may require modification as analytical and experimental evidence becomes available. It seems likely that multiple beam design curves will need to be developed to deal with the many different situations that occur.

Notable applications for which there are virtually no satisfactory design methods include arches (Galambos 1988, and Beedle 1991) and all those with primary torsion actions such as eccentrically loaded or curved beams. In addition, methods for designing haunched and tapered members (AISC 1986, Lee, Morell, and Ketter 1972, and Morell and Lee 1974) require extension beyond their present comparatively simplistic approaches.

While computer methods are available for the implementation of code rules for designing against lateral buckling (Trahair and Papangelis, 1992), and general purpose elastic buckling programs exist (Papangelis, Trahair, and Hancock, 1993), these latter have yet to be extended to design, even though some codes (SA, 1990) have general rules which allow this through the method of design by buckling analysis.

Finally, advanced analysis methods similar to those for designing against the in-plane collapse of compact members and frames have yet to be developed for lateral buckling. When they are, design will be greatly simplified, as it will only need to be demonstrated by the analysis that the structure can reach an equilibrium position under the strength design loads.

3.8 EDUCATION

The process of educating structural engineers undergoes continual changes at undergraduate, postgraduate, and professional levels, as more and more information on beam lateral buckling becomes available and is incorporated into design codes. The supporting material also undergoes continual changes, whether it be in the form of source textbooks (Timoshenko and Gere 1961, Vlasov 1961, Bleich 1952, Galambos 1988, and Trahair 1993a), design codes and commentaries, design textbooks (Dowling, Knowles, and Owens 1988, Johnston et al 1986, Trahair and Bradford

1991, Nethercot 1991), explanatory papers (Trahair, Hogan, and Syam 1993, Trahair 1993b) or worked examples (Bradford, Bridge, and Trahair, 1992). User-friendly computer programs (Trahair and Papangelis 1992, and Papangelis, Trahair, and Hancock 1993) can also assist in the education process.

Expert system computer programs (Rychener, 1988) have considerable potential to assist in the education process, but while their application to beam lateral buckling has been considered (Lai et al, 1991), they have yet to be fully developed for this purpose.

4. CONCLUSIONS

This paper provides a survey of the present state of knowledge of the important influences on the elastic lateral buckling and strengths of laterally unsupported beams, and of methods of design. It then outlines a number of areas where knowledge is incomplete under the headings of cross-section types, materials, structural forms, combined loading, analysis, experiment, design, and education.

Extensive research on elastic lateral buckling has led to a well-developed theory, and it can be said that the influences of cross-section, length, moment distribution, load height, restraints and supports are well understood. The effects of bracing, local buckling, web distortion, pre-buckling deflections and post-buckling behaviour have also been studied, but further work needs to be done on the computer analysis of frames, and the development of user-friendly programs for frames, arches, and tapered members.

Studies of the strengths of real beams have been largely limited to hot-rolled steel I-beams, and there is a need to extend these to beams of other materials and cross-sections. It is anticipated that this will lead to the development of multiple beam design curves. There is also a need to extend these strength studies to frames, arches, and tapered members, and to develop corresponding design methods, ultimately providing the basis for the extension of design by advanced analysis from frames which fail in-plane to include out-of-plane failure.

Current research on the strengths of beams under combined actions, such as those due to eccentric or inclined loads, is incomplete, and there is much to be done before the strengths of curved beams can be predicted with confidence.

Design codes are being continually updated to incorporate more guidance to designers on lateral buckling, as are supporting design aids such as textbooks, commentaries, explanatory papers, and worked examples. The potential of expert systems for this purpose has not been fully explored.

In summary, although there have been many research studies made of the lateral buckling of unsupported beams and structures over the past 100 years, there are still many challenging extensions waiting to fascinate the researcher of the future.

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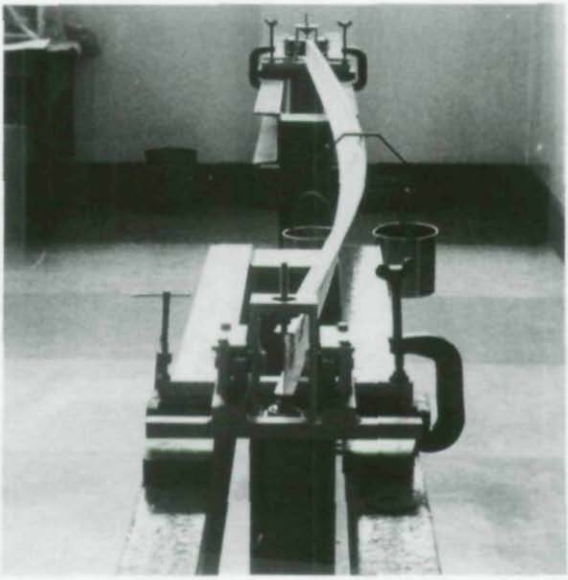
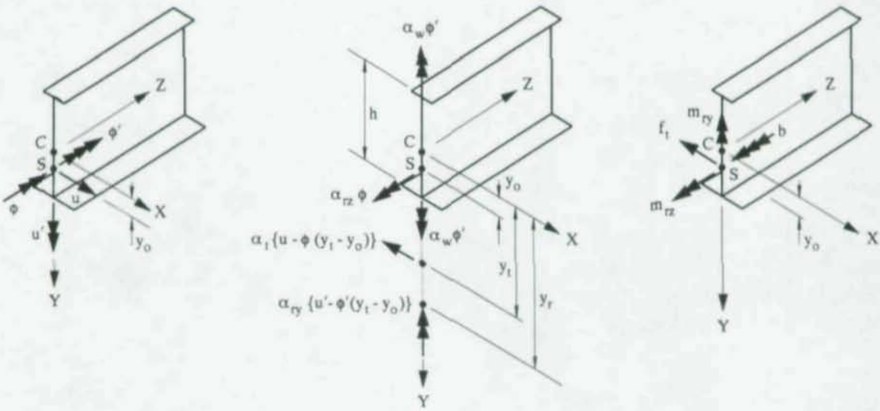
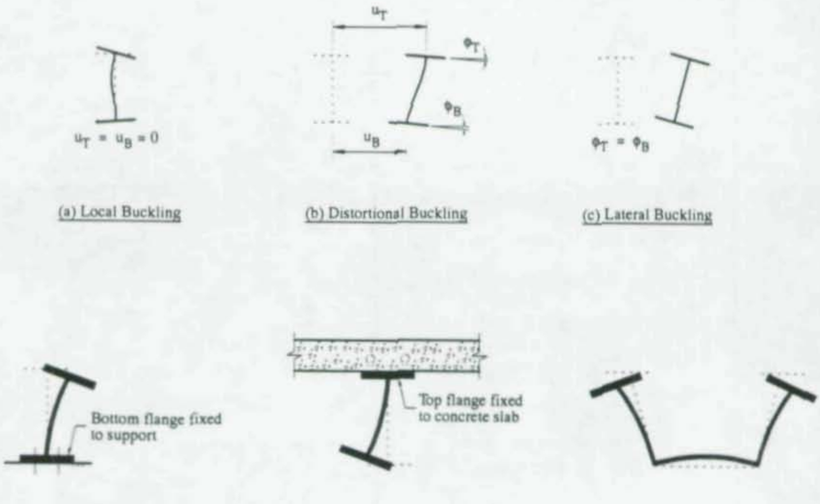


FIG. 1 LATERAL BUCKLING OF A SIMPLY SUPPORTED BEAM



(a) Buckling Deformations
(b) Restraining Actions
(c) Shear Centre Actions

FIG.2 CONTINUOUS RESTRAINT ACTIONS



(a) Local Buckling
(b) Distortional Buckling
(c) Lateral Buckling

(d) Beam at Simple Support
(e) Composite Beam in Negative Bending
(f) Trough Girder

FIG.3 EFFECT OF DISTORTION ON FLEXURAL-TORSIONAL BUCKLING

FIG.4 CROSS-SECTION TYPES

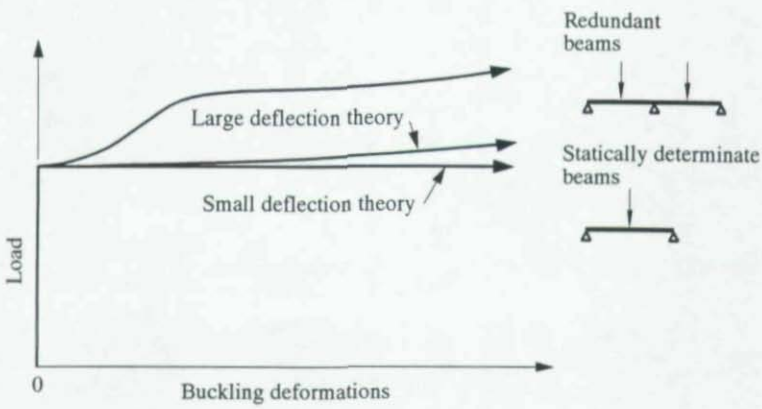


FIG.5 POST-BUCKLING BEHAVIOUR OF BEAMS

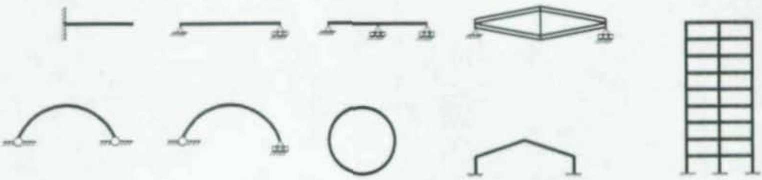
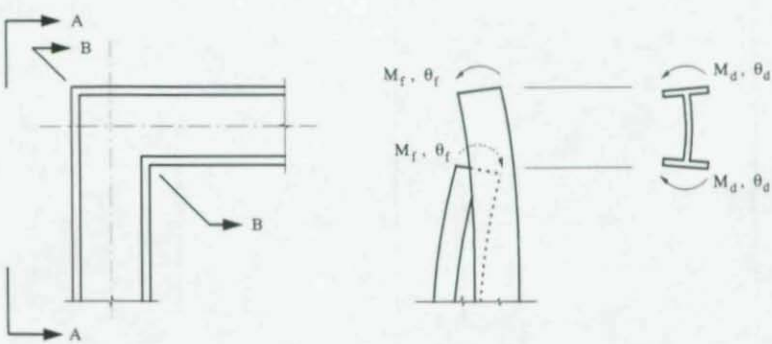


FIG.6 STRUCTURAL FORMS



(a) Unstiffened Rigid Joint

(b) Flange Warping
View A-A

(c) Section Distortion
Section B-B

FIG.7 WARPING AND DISTORTION

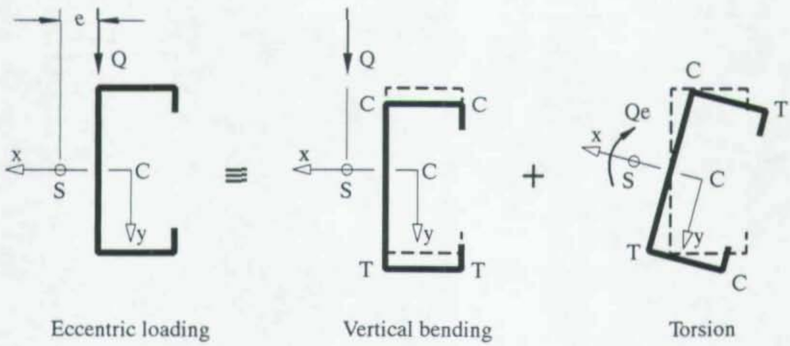


FIG.8 ECCENTRIC LOADING OF A CHANNEL BEAM

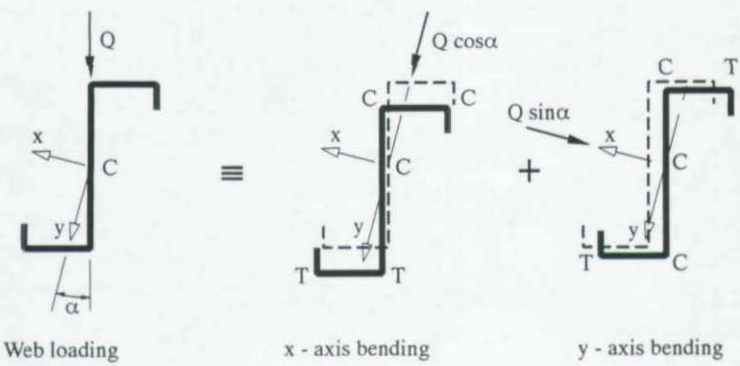


FIG.9 BIAXIAL BENDING OF A ZED-BEAM

Curved Girders Are Special

By Dann H. Hall*

I. Introduction

Horizontally curved girders may be thought of as aberrant straight girders. At times this may be the case when a straight girder is designed and it is brought into the world curved. However, it is possible also to think in the converse sense--straight girders are aberrant curved girders, that is, a curved girder with an infinite radius.

Curved girders are special in appearance, fabrication and structural behavior. The first curved girders were probably made from rolled shapes which were cold bent about their weak axis. However, when girder welding became accepted, curved girders grew in popularity.

Applications of curved girders include building structures such as balconies. However, the most widespread use of horizontally curved girders is in highway bridges where high speeds require smooth changes of direction.

The earliest laboratory tests of steel curved girders were performed in the United States during the late 1960s and early 1970s. A Federal Highway Administration project, Consortium University Research Team (CURT), financed much of this work.(1) In 1976, AASHTO adopted "Guide Specifications for Horizontally Curved Highway Bridges" (6) based on this work. The early provisions were Working Stress Design based. Later provisions based on Load Factor Design were developed.(3,11)

The Hanshin Expressway Corporation, Japan, adopted a specification for horizontally curved girder bridges in 1988.(5) These provisions are based on Working Stress Design. Other countries have not adopted special provisions for curved girders but often design horizontally curved bridges with variations of their provisions for straight girders.

In this paper, the behavior of curved girder structures will be discussed. Test results of component tests of single curved girders will be compared to several predictor equations. Finally, curved girder research underway in the United States will be discussed. Although both curved I girders (open) and box girders (closed) are utilized, this paper addresses only I girders.

II. Structural System

Figures 1 and 2 each show a two-girder, simple 160-foot span structure. Girders in Figure 1 are straight while the girders in Figure 2 are curved to a 400

**Principal, Bridge Software Development International, Ltd.*

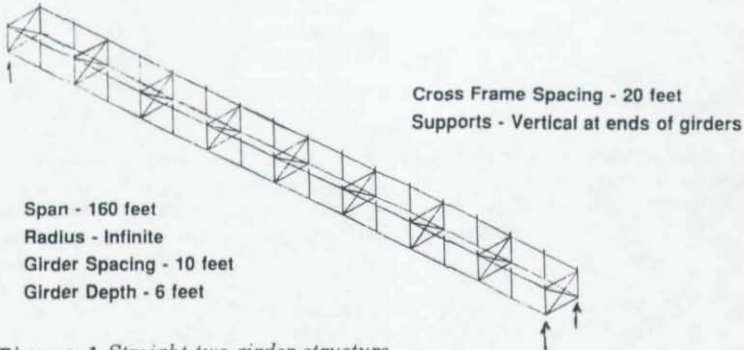


Figure 1 *Straight two-girder structure*

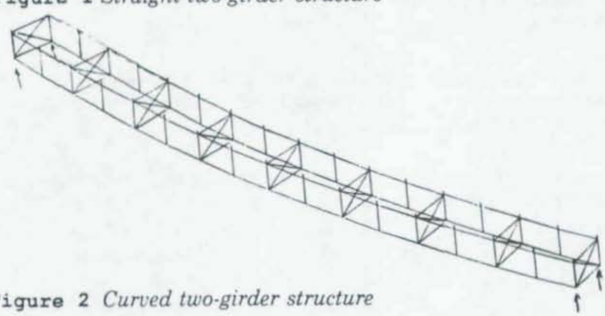


Figure 2 *Curved two-girder structure*

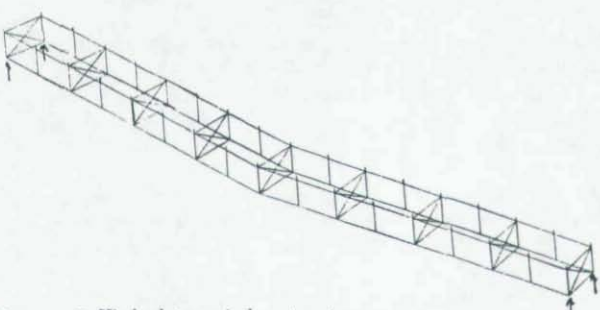


Figure 3 *Kinked two-girder structure*

foot radius. Figure 3 shows a structure similar to those in Figures 1 and 2 except that the same offset from tangent as the curved girders is accomplished with kinks. These finite element models represent noncomposite, unsymmetrical girders.

Table 1 gives selected analytical results due to a uniformly applied load of one kip/ft per girder in the downward direction. This loading represents a concrete bridge deck. Comparable reactions, mid-span deflections and vertical girder moments for the three structures are presented. Vertical moments in the two straight girders are equal; however, the outside girder in the curved and kinked cases carries the entire load. The total statical moment in the two girders is nearly equal in each case.

The interaction between the three pairs of girders differs in significant ways. The bracing members in Figure 1 only stabilize the girders, whereas they act as primary load carrying members in the curved and kinked structure. However, offset, rather than the curvature, is the critical structural effect.

Model Girder	**Vertical Reactions (Kips)**	**Mid-span Defl. (Inches)**	**Mid-span Moment (K-Ft)**
Straight			
Girder 1	80	3.18	3292
Girder 2	80	3.18	3292
Total	160		6584
Curved			
Girder 1	-8	4.71	-547
Girder 2	170	12.77	7318
Total	162		6771
Kinked			
Girder 1	14	6.10	-1114
Girder 2	148	6.63	7862
Total	162		6748

Table 1 *Reactions, Deflections and Moments*

Cross frames introduce restoring torques to the girders, resulting in lateral flange bending moments, or bimoments. The bimoments in top and bottom flanges are equal and opposite. Figure 4 gives lateral flange bending in the flange of the outside girder for the two non-straight structures. When the girder has a uniform curvature, lateral flange bending is distributed along the girder in proportion to the vertical moment. When the offset is created by a kink, the restoring forces and lateral flange bending are concentrated near the kink.

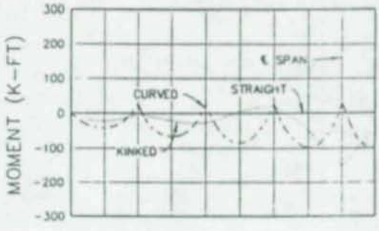


Figure 4 *Bottom flange lateral flange bending moments*

Figure 5 shows the cross frame forces for the two offset structures. Note the extremely large cross frame forces at the kink. If the cross frame was moved from the kink, the lateral flange bending moments would be extremely large at the kink while the cross frame forces would be mitigated substantially.

If deflections are small, an elastic analysis that considers the geometry, appropriate boundary conditions, bracing members, girder bending moments, shears and bimoments can predict structural behavior. Figure 6 shows the exaggerated deflected shape of the curved model. Differential top and bottom lateral deflections are often significant as well as differential vertical deflections between girders.

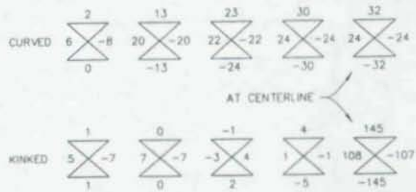


Figure 5 *Cross frame forces*

III. I Girder Free Body Diagrams

Figure 7 shows a free body diagram of a horizontally curved I girder between two torsional brace points. Vertical forces (not shown) are applied at the torsional brace points by diagonal cross frame members. Load is transferred between girders in this manner.

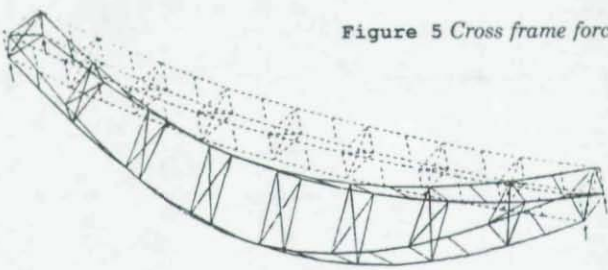


Figure 6 *Deflected shape of curved I girders*

The vertical bending moments effectively create non-collinear axial forces in the flanges.

Lateral bending moments, M_{lat}, are a function of the curvature, bracing spacing and girder depth. An approximate equation for the lateral bending moment at brace points, called the V-load Method (12), is often used.

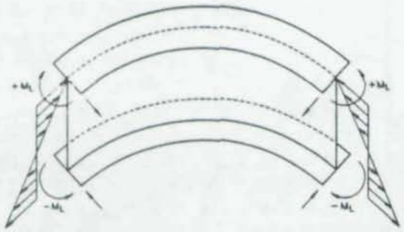


Figure 7 *Free body of curved I girder*

$$M_{lat} = \frac{M_v L^2}{10DR}$$

where:
M_v = Vertical moment
L = Unbraced length
D = Girder depth
R = Girder radius

Flanges

Figure 8 shows a compression flange free body ignoring web effects. The axial flange force tends to accentuate curvature while the lateral flange bimoments tend to reduce it. However, the net effect is always to increase curvature of the compression flange. Figure 9 shows a tension flange free body, again ignoring web effects. The axial force tends to reduce curvature and the lateral flange bending moment tends to increase it. The net effect can be to either increase or decrease the curvature of the tension flange, depending on flange stresses and stiffness. Dashed lines indicate deflected shapes.

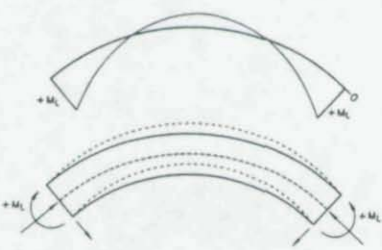


Figure 8 *Free body diagram of compression flange*

Figure 10 shows a free body diagram of the outside and inside halves of a

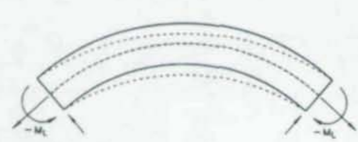


Figure 9 *Free body diagram of tension flange*

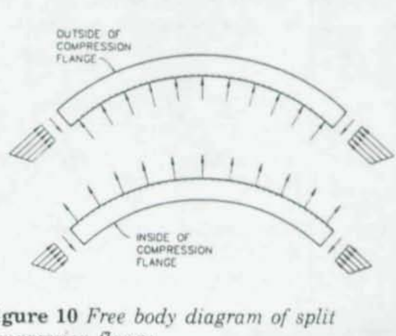


Figure 10 *Free body diagram of split compression flange*

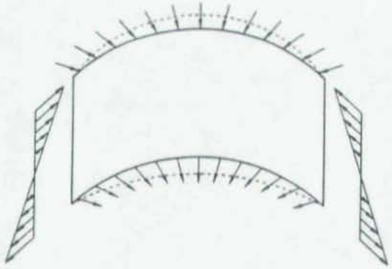


Figure 11 *Free body of web in pure bending*

compression flange with web effects. The flange stress distribution for a point near a torsional brace. The web exerts a force on the flange tending to restrain it from bowing. This force system may result in flange instability different from that of a straight girder flange. Not only does each plane warp. warping of the cross section varies with respect to location of the torsional braces. Inelastic lateral torsional buckling of curved girder flanges is not well understood at this time.

Web

Figure 11 shows a free body diagram of an horizontally curved I girder unstiffened web. The compressive edge of the web is being bowed by the compression flange and self-contained bending stresses. The tensile edge of the web may be either flattened or bowed depending on the tension flange behavior. The shear strength of the web restrains the relative position of the two flanges with respect to each other.

The addition of transverse and/or longitudinal web stiffeners tends to reduce web bowing. Attaching transverse stiffeners to the flanges has been found to increase ultimate bending strength of curved I girders.(9)

IV. Stability

Single girder

Resistance to lateral torsional buckling of a straight girder can be thought of as the ability of the girder to resist a slight lateral deflection. As long as the lateral bending moment within the compression flange due to curvature is larger than the external moment created by the slight lateral deflection, the girder has not reached its bifurcation (buckling) load.

Torsional restraints as well as lateral and vertical restraints are required for a single curved girder to be stable. For example, when a curved I girder bridge is erected, a single girder is often lifted. During this time it must be torsionally restrained either by balancing its own weight or by external means.

The need for external torsional restraint of a horizontally curved girder subjected to vertical loads has been understood and the equations expressing the stress functions have been available for 30 years.(2) However, the stability of a curved I girder which is braced only at intervals is not well understood.

In the case of a horizontally curved girder, stability is dependent on phenomena similar to a straight girder but much more complex. As compression stress increases, its curvature increases. This increased curvature is resisted by the lateral bending moments.

As curvature increases, the restraining lateral moments increase. At some point the increase in the applied moment due to increased curvature and axial force is greater than the marginal increase in the restraining bimoment and lateral deflection is unrestrained. This behavior is similar to bifurcation load except that the curvature is intentional.

Multi-girder

Figure 2 shows a simple span system consisting of two curved girders connected by cross bracing and supported by bearings at their ends. This structural system is stable only when the bracing between girders is capable of preventing free rotation of the girders.

The girders achieve stability from the interaction between girders. Since neither girder alone would be stable, failure of one of the girders would cause a significant reduction in the stability of the system.

V. Bending Strength

Most structural design specifications provide predictor equations for single members although most structures are composed of many members. Horizontally curved I girders are different from most structural members in that their behavior is dependent on the boundary conditions. Unfortunately, most tests of the curved girders have been performed on single elements. Translation of the results requires assumptions about the boundary conditions. Each of these approaches has led to its own set of problems. Nevertheless, these tests have been important in forming design provisions.

Failure of a curved I girder unsupported along its compression and tension flanges except at brace points is characterized by several unique phenomena. The compression flange commences at once to bend outward between brace points. The distance between brace points is always greater than the effective length. Typically, the unbraced length of a straight girder is equal to one ($K=1.0$). The following equation gives an estimate of the unbraced length used in the Hanshin Expressway for unbraced length.(5) The effective unbraced length equals gamma times L, the spacing between cross frames.

$$\gamma = 1.0 - 1.97\Phi^{1/3} + 4.25\Phi - 26.3\Phi^3$$

where:

$$\Phi = \frac{l}{R}$$

where:

l = Unbraced length
R = Girder radius

Web behavior

Webs of curved girders tends to bow with applied bending moment. This behavior has been observed to increase the flange bending stress. Interestingly, the Hanshin specification, like AASHTO Working Stress Design, limits transverse stiffener spacing to the girder depth. The AASHTO Load Factor Design provisions permit transverse stiffeners to be eliminated when the web meets slenderness and shear stress limits.

Nakai (9) found that attaching single-sided transverse stiffeners to the compression flange is beneficial in pure bending.

Predictor equations

Equations for compact and non-compact sections are provided in the AASHTO Guide Specifications.(6) A flange can be treated as compact without the web being compact. These predictor equations are based on modifications of the lateral torsional buckling equations used in AASHTO at the time (1976). The provisions require vertical bending moments and bimoments to be determined. *Bimoments need be determined only at brace points. The lateral flange stress is* limited to .5 times the vertical bending stress. The unbraced length divided by the radius is limited to 0.1.

$$F_{by} = F_{bs}\bar{\rho}_B\bar{\rho}_w$$

where:

$$F_{bs} = F_y(1 - 3\lambda^2)$$

where:

F_y = Yield stress flange

$$\lambda = \frac{1}{\pi}\frac{l}{b}\sqrt{\frac{F_y}{E}}$$

$$\bar{Q}_B = \frac{1}{1+\frac{l}{b}(1+\frac{l}{6b})(\frac{l}{r}-0.01)^2}$$

$$\bar{Q}_w = 0.95+18(0.1-\frac{l}{R})+\frac{\frac{f_w}{f_b}[0.3-0.1\frac{l^2}{Rb}]}{\bar{Q}_B(F_{bl}/F_Y)}$$

l - Unbraced length
b - Width of compression flange
R - Radius of curvature
f_v - Lateral flange bending stress
f_b - Vertical bending stress

Nakai (10) developed an equation that modifies the equation for lateral buckling and web bend buckling of straight girders found in the Japanese Bridge Specification. Since this specification (12) is based on working stress design, equations must be modified by a factor of safety.

$$M_u = [1.92 - 0.357\frac{L^2}{Rb_{cf}}]M_a$$

where:
M_a - Minimum of:
M_w - Moment limited by web capacity
M_f - Moment limited by flange capacity
L - Unbraced length
R - Radius of girder
b_{cf} - Width of comp. flange

Fukumoto (3) developed an equation based on a parametric study of curved girders. This equation must be solved using an iterative procedure and therefore would be difficult for designers to manipulate.

The Hanshin Expressway Corporation developed a curved girder specification that utilizes an interaction equation between vertical and lateral

$$\lambda^4\delta^4-([1+\frac{P_e(d-t_{cf})}{2M_p})(\frac{L^2}{2Rb_{cf}})]\lambda^4+1)\delta^2-(\frac{L^2}{2Rb_{cf}})\delta+1=0$$

where:

$$\lambda = \sqrt{\frac{M_p}{M_e}}$$

$$\delta = \frac{M_u}{M_p}$$

where:
P_e-*Elastic buckling load of section*
M_p-*Plastic capacity*
d-*Section depth*
t_{cf}-*Comp. flange thickness*
b_{cf}-*Width of comp. flange*
L-*Unbraced length*
R-*Girder radius*

bending stresses.(5) Since the Hanshin specifications are working stress provisions, the strength version of the equation shown here has been modified to remove safety factors. The required bimoments are computed by the V-load method.

$$\frac{f_a}{(F_{ba})_c} + \frac{f_w}{F_{bao}} \leq 1.0$$

where:
f_b-*Normal flexural stress*
$(F_{ba})_c$-*Critical normal bending stress*
f_w-*Lateral flange bending stress*
F_{bao}-*Yield stress*

The unbraced length divided by the radius is limited to 0.2. The sections must be compact.

In this section three different predictor equations: AASHTO; Hanshin and Fukumoto, are compared. The comparison is made with respect to Specimen M2 which was tested by Nakai, et al. Figure 12 shows the properties of the test specimen.

Figure 13 shows a comparison plot of the predictor equations. The abscissa is taken as the unbraced length divided by the radius (L/R). The solid line plots varies L while holding R constant at the test value. The dashed line plot varies R while holding L constant at the test value. Of course, the plots cross at the L/R value. The ultimate moment capacity is non-dimensionalized by the plastic moment capacity of a straight girder of the same scantlings and strength.

The AASHTO solid line predictor is terminated where L/R equals 0.1. The lateral flange bending stress is computed by the V-load method to be 0.56 times the vertical bending stress in the test specimen. ASSHTO limits this ratio to 0.5. The dashed line is terminated where the V-load lateral flange stress divided by the vertical bending stress is 1.0. The Hanshin predictors are terminated where L/R = 0.2. The Fukumoto predictor extends over the entire range. It is of interest to note that in no case is the strength a true function of the variable L/R. However, in all three cases the curves behave similarly. *In the lower range of L/R the* AASHTO predictor with L varying is higher than Hanshin with R varying. In other cases, each predictor remains in a relative regime with AASHTO the lowest and Fukumoto the highest. They each underestimate significantly the test specimen strength.

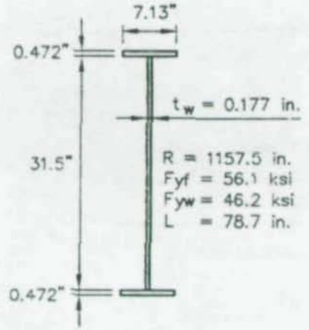


Figure 12 *Specimen M2*

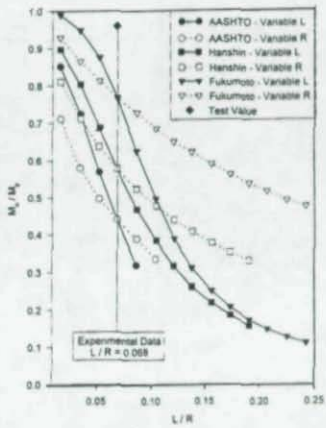


Figure 13 *Strength predictions of Specimen M2*

Component Bending Tests

Figure 14 shows a typical test arrangement used by Mozer, *et. al.*, (7,8) to test curved girders in pure bending. Figure 15 shows a typical arrangement used by Nakai, *et. al.*, to perform similar tests. Mozer's tests are in positive bending while Nakai's are in negative bending. The bending span is longer than the torsional unbraced length in Mozer's tests. The test fixture restrains the girder torsionally and vertically in Nakai's tests.

Table 2 gives a comparison of the test results for pure bending compared to the predictor equations available. The test values for the ultimate strength are compared to the predictor equations. The AASHTO provisions are limited to L/R less than 0.1 and Hanshin provisions are limited to 0.2. It is possible that the Japanese tests have excessive torsional restraint that may cause the apparent high values compared to the predictor equations.

All but AASHTO of the predictor equations apply to only compact sections. They are all predicated on failure of either the flange and/or web.

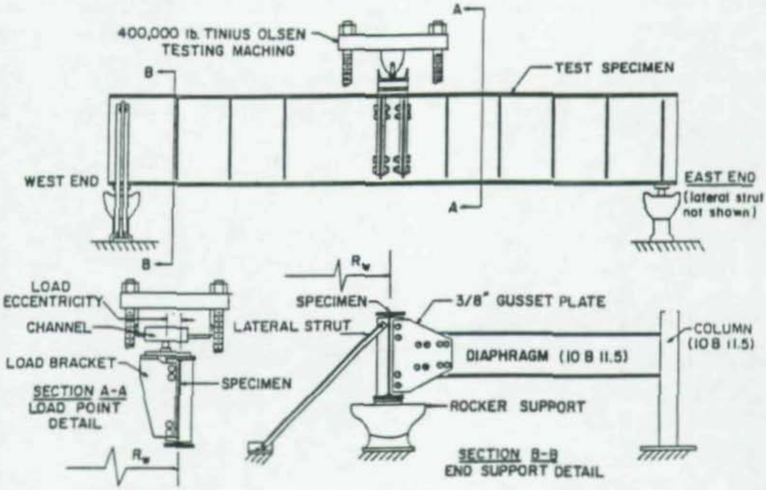
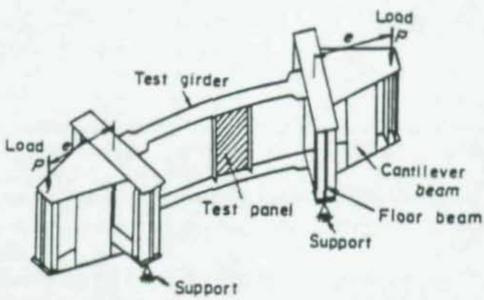


Figure 14 *Culver bending test arrangement*



Loading and supporting device for bending test.

Figure 15 *Nakai bending test arrangement*

VI. Shear

There have been no unique mathematical predictor models for shear in curved girder webs. Culver and others report that post buckling strength can develop in curved webs. However, there is some fear that deflections will be large compared to straight girders. As a result, the elastic buckling value is used in AASHTO and Hanshin design specifications.

Bending-shear interaction

Interaction between shear and bending is ignored in the AASHTO curved girder provisions because it is ignored in the straight girder provisions when elastic buckling limits are used for the web. Research (10) has shown that the typical moment-shear interaction equations seem to give rather good results for component tests of moment and shear.

	M_u Test K-ft	AASHTO		HANSHIN		NAKAI		FUKUMOTO	
		PRED	RAT	PRED	RAT	PRED	RAT	PRED	RAT
L1A	1830	1716	0.94	1107	0.61	1999	1.09	1504	0.82
L2A	1830	1749	0.96	1136	0.55	1993	1.09	1533	0.84
G15	1377	****		652	0.47	1279	0.93	989	0.72
G08	2120	1992	0.94	1164	0.55	2213	1.04	1912	0.90
M1	8098	6965	0.86	7509	0.93	5657	0.70	8007	0.99
M2	7754	3708	0.48	4713	0.61	6508	0.84	6187	0.80
M3	6131	2627	0.43	3504	0.57	5356	0.87	4803	0.78
M4	7203	****		2654	0.37	7972	1.11	4317	0.60
M5	5902	****		2298	0.39	6790	1.15	3592	0.61
M6	6287	****		2277	0.36	6649	1.06	3503	0.56
M7	6547	****		2371	0.36	7176	1.10	3818	0.58
M8	2935	****		****		1451	0.49	1054	0.36
M9	5548	****		****		6934	1.25	3073	0.55

Table 2 *Comparison of predictors to pure bending tests*

**** -- Equation not applicable (Central angle exceeds allowable.
Mx --(Ref. 8); Lx --(Ref. 6); Gx --(Ref. 7)

VII. Current Research

The National Cooperative of Highway Research Program (NCHRP) of the Transportation Research Board initiated a project (NCHRP 12-38) to update the present AASHTO curved girder provisions. The work involves the present state-of-the-art. Special emphasis is being placed on the construction of curved girder bridges. A survey of curved girder bridges fabricated across the United States is being undertaken. Results will provide insight into the range of curvature experienced as well as how the bridge have been detailed. This work is scheduled to be completed in 1995.

The Federal Highway Administration has financed a five-year basic research effort. This work is to assist in the development of an AASHTO Load and Resistance Factor Design (LRFD) design code for horizontally curved bridges. The project includes both analytical and experimental studies. Open and closed sections are included.

The experimental work involves laboratory testing of components and testing of I girder and box girder bridges. The I girder bridge is planned to be a full-scale simple span structure. Full-scale tests eliminate the need for compensatory dead loads and are able to reflect accurately fabrication influences such as initial distortions and residual stresses.

The proposed steel framing for the I girder tests is shown in Figure 16. This framing can be used to test girder components in pure bending and bending with shear. By using a bridge as a test frame for components, ideal boundary conditions can be obtained. Tests have been performed on pairs of curved girders to accomplish similar objectives. However, earlier tests always tested the entire frame to failure. In the planned tests, only the component is planned to fail. As in the two-girder examples, the statical moment across the three girders is predictable from the loads and span. Since the interior two girders are expected to remain elastic, the inelastic moment in the component can be deduced rather than measured directly.

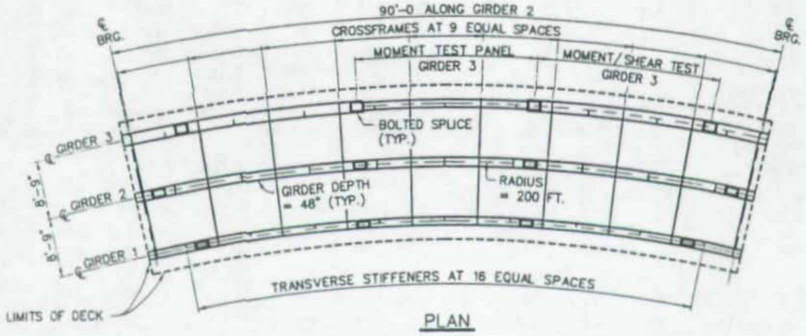


Figure 16 *Plan of I girder test frame*

Testing of a single member that derives its strength and loads in part from the entire system is a complex assignment. Analytical studies indicate that as the component commences to yield, its stiffness is reduced sufficiently to change the load distribution. The cross frames act differently under this condition. Instead of adding load to the exterior girder, the sense of force changes and load is transferred to the interior center girder.

When the component tests are completed, a concrete deck will be placed on the steel frame and the bridge will be tested to failure.

Similar tests are being planned for box girders. However, the component tests are expected to be less complex than described above for I girders.

VIII. Summary

Curved I girder research has been directed toward making adjustments to the straight girder predictor equations to predict curved girder strength. This approach has not been successful as evidenced by the wide range of predicted values. There has been no theory presented to explain the inelastic behavior of curved I girders. Most work being done at present revolves around inelastic finite element analyses.

The AASHTO and Nakai predictors involve the straight girder as a basis. The Hanshin provisions are indirectly based on straight girder provisions but deviate significantly from them via the interaction equation for vertical and lateral bending stresses.

Fukumoto's equation is most unique and seems to predict test results rather well. However, it is unwieldy from a designer's view. It may be more fruitful to develop a predictor equation for curved girder strength that can be reduced to that for a straight girder when the radius becomes infinite. Such an equation may include parameters such as Fukumoto's equation.

Such an approach will permit straight girders to be considered as special curved girders.

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STABILITY CONSIDERATIONS ON SEISMIC PERFORMANCE OF STEEL STRUCTURES

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1 Introduction

After the Kanto Earthquake which brought us tremendous disaster in the Tokyo Metropolitan area in 1923, heated debate on the seismic design philosophy of building structures came among leading researches, with many of their contentions appearing in our professional journals as well as in daily newspapers. One professor asserted that building structures should be rigid and strong to resist seismic forces, whereas his opponent argued that building structures should be designed flexibly so that they can escape from earthquake forces. The debate did not bring a conclusion, however, mainly because of the lack of knowledge at that time on the characteristics of earthquake ground motions. In 1935 and at almost the last phase of the debate, Dr. Ryo Tanabashi, a young professor of the Kyoto University, proposed a very advanced theory and contended that the earthquake resistance capacity of a structure should be measured by the amount of "potential energy" that the structure can absorb before collapse. The potential energy discussed in his theory can be interpreted as the energy dissipation capacity, the term often used nowadays[1,2]. His well-insighted theory was not realized in our design practice for a long time, but advancement of earthquake engineering has finally made it possible to apply his theory in seismic design. In fact, his theory, now recognized in the name of energy concept, has received much attention in the world.

First, this paper overviews the seismic design based upon energy concept, including the procedures for estimating the energy dissipation demand and capacity of steel building structures. Second, present seismic design procedures are outlined, with the Japan's seismic design regulation as an example, and their interaction with energy concept is discussed. Also introduced is a seismic design procedure in which energy dissipation demanded to a structure is directly compared with its energy dissipation capacity. Finally, structural testing techniques are introduced in light of their roles for further advancement of our seismic design.

2 Energy Approach in Seismic Design

2.1 Basics of Seismic Design for Steel Building Structures

In many countries with high seismic activity, a two-level seismic design procedure has been adopted[3, 4]. Level I is for minor to moderate earthquake motions, which the structure considered sustains relatively frequently in its life time, and the structure should resist these motions with minimal structural damage. Level II is for large earthquake ground motions, experienced at most for a few times by the structure during its life span. Under these ground motions, structural damage, i.e. inelastic excursions of structural members, is permitted as long as it does not cause structural collapse or loss of lives. The major source of structural resistance under Level II earthquake ground motions is energy dissipation supplied by hysteretic damping of the structure, and this damping is provided by hysteresis of structural members, which is generated under repeated deformations beyond yielding. Thus, a safety criterion for Level II design is such that the hysteretic energy dissipation demanded to the structure should not be greater than its energy dissipation capacity. In the next section, hysteretic energy dissipation demanded to steel building structures is outlined.

2.2 Energy Dissipation Demanded to SDOF Systems

For the sake of clarity in discussion, earthquake response of an SDOF system shown in Fig.1 is introduced first. If the system is taken to be linear-elastic, its equation of motion under ground motion is given as:

$$m\ddot{x} + c\dot{x} + kx = -m\ddot{x}_g \tag{1}$$

where m, c, and k are the mass, damping coefficient, and stiffness of the system, x and $\ddot{x}_g$ are the displacement relative to the ground and ground acceleration, and the dot denotes differentiation with respect to time. Multiplying $\dot{x}dt$ $(=dx)$ on both sides of Eq.1 and integrating over the entire duration of the ground motion (from $t = 0$ to $t = t_0$) lead Eq.1 to:

$$m\int_0^{t_0} \ddot{x}\dot{x}dt + \int_0^{t_0} \dot{x}^2 dt + k\int_0^{t_0} x\dot{x}dt = -m\int_0^{t_0} \ddot{x}_g \dot{x}dt \tag{2}$$

The right hand side of Eq.2 is the total energy inputted into the system by the ground motion, designated as E. The first and third terms of the left hand side of Eq.2 are reduced to the kinetic and strain energies stored at the end of the ground motion. These two terms, added together, constitute the elastic vibration energy (E_e) at the end of the ground motion. The second term of the left hand side of Eq.2 shows the energy dissipated by viscous damping, termed E_h. Thus, Eq.2 is rewritten as:

$$E_e + E_h = E \tag{3}$$

If the restoring force behavior of the system is taken to be bi-linear as shown in Fig.2, the equation of motion is:

$$m\ddot{x} + c\dot{x} + F(x) = -m\ddot{x}_g \tag{4}$$

Where $F(x)$ is the restoring force corresponding to the (relative) displacement x. The equation can be converted into an energy form as:

$$m\int_0^{t_0} \ddot{x}\dot{x}dt + \int_0^{t_0} \dot{x}^2 dt + \int_0^{t_0} F(x)\dot{x}dt = -m\int_0^{t_0} \ddot{x}_g \dot{x}dt \tag{5}$$

And the energy equation is obtained as:

$$E_e + E_h + E_p = E \tag{6}$$

in which E_p is the energy dissipated by the system's hysteresis.

Figure 3 plots the total input energy (E) in terms of the equivalent velocity ($V_E = \sqrt{2E/m}$) against the natural period of the system (T). The plotted values were computed by direct integration using the Newmark method ($\beta = 1/4$), and the 1940 El Centro earthquake record (N-S component) was employed for the ground motion. Other parameters used in the computation were: 1) the natural period of the system (T), 2) the yield force (F_y) in the bi-linear restoring force relationship, given in terms of F_y/F_{max}, with F_{max} as the maximum force to be resisted if the system would respond only elastically, 3) the amount of viscous damping in terms of the viscous damping ratio (h), and 4) the strain hardening stiffness in terms of its ratio (α) to the initial stiffness. Figure 3 indicates that the equivalent velocity corresponding to the total input energy (V_E) remains almost unaffected by the yield force, viscous damping, or strain hardening.

Figure 4 shows the energy dissipated by hysteresis (E_p) against the natural period (T), obtained from the same analysis. Here, E_p is expressed in terms of an equivalent velocity (V_p), defined as:

$$V_p = \sqrt{\frac{2E_p}{m}} \tag{7}$$

This figure also exhibits that the equivalent velocity (V_p) remains relatively unchanged regardless of the yield force and strain hardening, although, in this case, the value of V_p decreases for larger viscous damping ratios. The bold line in Fig.4 shows the (pseudo) velocity spectrum (S_v) [5] of the corresponding linear-elastic system, denoting that the equivalent velocity (V_p) is reasonably correlated with S_v.[6,7].

The foregoing observation points out that the total input energy (E) and hysteretic energy dissipation (E_p) are not sensitive to either the yield force or strain hardening but controlled primarily by the amount of mass (m) (as V_p in Eq.7 is a function of the mass) and natural period (T) of the system. This fact as well as a reasonable correlation between V_p and S_v greatly simplifies the seismic design procedure based upon energy concept. It should be remembered that the energy dissipated by hysteresis (E_p) and its equivalent velocity (V_p) discussed in this section are the ones demanded to the system. A thorough review on the energy concept is presented elsewhere [7].

2.3 Correlation Between Hysteretic Energy Dissipation and Ductility

If the system's restoring force behavior is taken to be bi-linear (Fig.2), hysteretic energy dissipation demanded to the system can readily be correlated with the demand of plastic deformation. Considering one half cycle (cycle i) between two adjacent unloaded deflections (A and B in Fig.5) in a bi-linear restoring force vs. deformation relationship. The plastic deformation (δ_p), defined as the distance between points A and B, is approximated by:

$$\delta_{pi} = \frac{E_{pi}}{F_y} \tag{8}$$

$$\text{or, } \mu_i = \frac{\delta_{pi}}{\delta_y} = \frac{E_{pi}}{F_y \delta_y} \tag{9}$$

Where E_{pi} is the hysteretic energy dissipated during this half cycle. Thus, the total plastic deformation ($\delta_p = \Sigma\delta_{pi}$), called the cumulative plastic deformation, is given as:

$$\delta_p = \Sigma\delta_{pi} = \frac{E_p}{F_y} \tag{10}$$

$$\text{or, } \mu = \Sigma\mu_i = \frac{\Sigma\delta_{pi}}{\delta_y} = \frac{E_p}{F_y \delta_y} \tag{11}$$

Here, $\mu\,(=\Sigma\mu_i)$ is called the cumulative ductility. In this manner, hysteretic energy dissipation can be converted to a well-known ductility quantity (μ), although, in this case, the ductility is given as "*cumulative*" rather than as "*maximum*".

2.4 Energy Dissipation Demanded to MDOF Systems

To generalize the discussion in the previous section, considered next is an n-DOF system as shown in Fig.6, which represents an n-story building structure. Let us consider that each story has its unique mass, stiffness, yield force, and strain hardening. Figure 7 illustrates the sum of the energies dissipated by hystereses of respective stories against the fundamental natural period of the system. Here, energy dissipated is expressed again in terms of the equivalent velocity V_p, defined as:

$$V_p = \sqrt{\frac{2\Sigma E_{pj}}{\Sigma m_j}} \tag{12}$$

Where the subscript j denotes that the quantity is associated with the j-th story. To obtain the necessary data, inelastic time-history analysis using direct integration was conducted, with the 1940 El Centro earthquake record (N-S component) as the ground motion. Also plotted in Fig.7 are the equivalent velocities of the corresponding SDOF systems. This figure demonstrates that the equivalent velocity (V_p) is nearly the same between SDOF and MDOF systems, suggesting that, for MDOF systems, the energy dissipated by hysteresis is primarily a function of the total mass (Σm_i) and fundamental natural period. Stiffness and yield force distributions over the story are other factors that may affect the equivalent velocity (V_p). Further analysis, however, has revealed that V_p remains relatively unchanged regardless of these distributions, and, by adjusting the story yield force carefully so that they start yielding approximately at the same time, and, particularly if some degree of strain hardening is present, the cumulative ductility ($\mu = \Sigma\mu_i$) can be maintained approximately the same for all stories.

3 System Ductility vs. Member Ductility

The ductility (and plastic deformation) demand discussed in the previous section is associated with the story of the building. In design process, this demand should further be decomposed into ductility demands to individual structural members such as beams and columns that constitute the building. To be emphasized here is that the value of ductility demanded to the story is not the same as the value of ductility demanded to individual members, and a simple example is given below.

Considered is a subassemblage shown in Fig.8(a), in which the column height and beam length are L and $2L$ and the flexural rigidities of the column and beam are EI and $2EI$, respectively. Further, the full-plastic moment of the beam's cross-section is set at M_p, whereas the column is assumed to remain elastic throughout. The elastic relationship between the horizontal force F and corresponding horizontal displacement Δ are given:

$$\Delta = \Delta_c + \Delta_\theta = \frac{FL^3}{3EI} + \frac{FL^3}{3EI} \tag{13}$$

Where Δ_c and Δ_θ are the displacement component given by bending of the column and the displacement component given by rotation at the beam-to-column connection (Fig.8(b)). Suppose that the subassemblage remains linear-elastic until the bending moment at the beam-to-column connection reaches M_p, and, once this moment is attained, a plastic hinge is formed at the left end of the beam. At the instant of yielding, the horizontal displacement (Δ_y) and the rotation at the beam-to-column connection (θ_y) are:

$$\Delta_y = \frac{2M_pL^2}{3EI}, \text{ and } \theta_y = \frac{M_pL}{3EI} \tag{14}$$

Now, consider that the subassemblage is further deformed to a horizontal displacement of $3\Delta_y$ (i.e., a ductility ratio of 3 for the subassemblage). Since deformation from Δ_y to $3\Delta_y$ is developed solely by the rotation of the plastic hinge(θ_p), the following equation is given:

$$2\Delta_y = \frac{4M_pL^2}{3EI} = \theta_p L \tag{15}$$

Thus,

$$\theta_p = \frac{4M_pL}{3EI} \tag{16}$$

This amount of rotation corresponds to the ductility ratio of 5 [$=(\theta_p + \theta_y)/\theta_y$)] for the beam (Fig.8(c)). As this simple example demonstrates it, ductility demand is not the same between the story and individual members and also varies from one member to another.

4 Deformation Capacity of Steel Frames and Members

4.1 Stability Effect in Seismic Performance of Steel Structures

In the previous section, it was pointed out that energy dissipation demand is a good and robust index for specifying seismic resistance required for steel structures and that this energy dissipation demand can be converted to a well-known ductility demand. Then, in seismic design based upon the energy concept, a logical safety criterion is given such that ductility demand of a structure be not greater than its ductility capacity. Since both the demand and capacity terms can be decomposed into those of individual structural members and elements that constitute the structure, to be known is ductility capacities of important structural members and elements that are assigned to dissipate input energy by their hystereses.

Structural steels are known to be very ductile in their stress vs. strain behavior, considered to possess sufficient energy dissipation capacity in themselves. Nevertheless, steel members made of such ductile steels often exhibit little ductility. The primary source for insufficient ductility capacity is "buckling and instability" inherent to steel members, such as local buckling, lateral torsional buckling, and $P\delta$ effect. Figures 9 to 11 show several examples of force vs. deformation behavior of steel members. If width-to-thickness ratios of a thin-walled cross-section are large, local buckling occurs in an early stage, which promotes the loss in resistance (Fig.9) [8]. If a beam or a beam-column is slender, lateral torsional buckling takes place before it sustains large deformation, which brings about decrease in ductility capacity (Fig.10) [9]. $P\delta$ effect is also a critical factor in controlling the ductility capacity of columns (Fig.11) [10].

A countless number of studies have been made for investigating those stability behaviors in connection with plastic design, and many of the findings have been incorporated into present seismic design specifications for steel structures. Several comments, however, should be addressed as to the difference in ductility consideration between plastic design and seismic design. First, earthquake loading, applied in the horizontal direction, causes beams and columns to sustain deformations in double curvature. This type of deformations is normally less susceptible to lateral torsional buckling than deformations with single curvature when compared for the same slenderness. Second, lateral sway in columns may cause significant $P\Delta$ effect, which is often a major source in loss of ductility. Third and probably most important, ductility demanded in seismic design is generally larger than that demanded in plastic design, although the degree of difference varies in accordance with the relative intensities of earthquake forces prescribed in seismic design. In other words, we generally have to deal with shorter members with stockier cross-sections in seismic design than in plastic design.

Stability for short members with stocky cross-sections, which involve ductile deformations after yielding, is influenced a great deal by the stress vs. strain relationship of structural steels and considered yet difficult to quantify accurately. The difficulty stems mainly from the fact that ductility capacity is very sensitive to the material's strain hardening characteristics [11].

4.2 Effect of Load Reversals on Stability of Structural Members

Earthquake loading is characterized by forced deformations involving load reversals, and these reversals should affect the ductility behavior of steel members. Many findings incorporated in current seismic design, however, are based upon investigations of experiments with one-way (monotonic) loading rather than with loading of many reversals. One of the primary reasons is that stability behavior accompanied by large ductility is already difficult enough even with one-way loading. Second reason is that forced deformations induced in structural members under earthquake motions vary from one member to another and also from one earthquake to another, and therefore, it is extremely difficult to select a particular deformation time-history that can generalize the earthquake-induced deformations. In fact, to the writers' best knowledge, we still do not have any well-accepted standard loading procedure for tests aiming at investigating the seismic resistance behavior of structural members. Considering those uncertainties still lying at the present time, we adopt a cautious approach when developing design provisions associated with member ductility. That is, basic information on ductility capacity of structural members is obtained by experiment and analysis with one-way loading. Next, ductility capacity thus obtained is correlated to "cumulative" ductility capacity in a conservative manner, and its validity is strengthened by experiment in repeated loading condition with a prescribed deformation history [12]. Figure 12 shows an example of tests conducted in repeated load condition, demonstrating that, up to a certain level of deformation, the hysteresis is stable without any reduction in resistance even after many cycles of load reversals.

5 Seismic Design Procedure Based on Static Earthquake Forces

5.1 Trade-Off Between Strength and Ductility

As indicated in Eq.11, we have flexibility for the choice of strength and ductility demands in seismic design. That is, even if the energy to be dissipated by hysteresis (E_p) is given, we can still play with the trade-off between the strength and ductility demands; i.e., a large ductility is required if a smaller strength is assigned and vice versa. Two approaches are conceived in selecting the strength and ductility requirements. One is first to assign a strength capacity for the structure, then to estimate the associated ductility demand (Eq.11), and finally to check the demand against the ductility capacity of the structure. The second approach is first to assign a ductility capacity for the structure, then to estimate the associated strength demand, and finally to check the demand against the strength capacity of the structure. Most of the seismic design specifications in the world, including the Japan's Seismic Design Regulations outlined below [13], adopt the second approach. The first and probably strongest reason is that, in the second approach, seismic design is transferred to a well-established static strength design, and conformity to design procedures enforced for other loading conditions can be achieved. The second reason, more implicit but equally crucial, is that ductility capacity of structures and structural members is yet very difficult to quantify accurately, and it is hardly possible to derive simple design equations that can specify member ductilities with reasonable accuracies. Under such circumstances, design provisions related to ductility demand are destined to be conservative and crude, and detailed checks between demand and capacity of member ductilities are no longer feasible. Difficulty in distributing the energy to be dissipated by the structure to individual stories is also a reason to support the second approach. If the structure is an SDOF system, the trade-off between strength and ductility discussed above is very straightforward. For building structures with many stories, however, distribution of required energy dissipation to respective stories depends a great deal on strength and strain hardening characteristics of the stories; i.e., energy to be dissipated tends to concentrate on a story with a smaller strength. In this condition, the profile of strength capacity along the story plays a critical role on energy dissipation and ductility demanded to individual stories. This interaction between the profile of story strength capacity and the ductility demanded to the story, together with the difficulty in accurate estimate of ductility capacities of individual structural members, makes the second approach: strength design with conservatively assigned ductility demand, more practical. In what follows, the Japan's Seismic Design Regulations are introduced for explicating the seismic design procedure practiced at the present time.

5.2 Outline of Japan's Seismic Design Regulations

As discussed earlier, the most critical factor that controls the ductility capacity of structures and structural members is "buckling and instability", characterized by local buckling, lateral torsional buckling, and $P\delta$ and $P\Delta$ effects. In the Japan's Limit State Design Specification for Steel Building Structures (in draft form) [13], ductility capacity of the structure is classified into four categories: Category S-I to S-IV, in accordance with its vulnerability against buckling and instability. To account for local buckling behavior, cross-sections are classified into four categories in accordance with their width-to-thickness ratios. Category P-I is for thick cross-sections that ensure large ductility before progress of local buckling reduces the resistance, and Category P-II is also for compact cross-sections sustaining ductile deformation, with the degree of ductility capacity not so large as that expected in cross-sections with Category P-I. Category P-III is for cross-sections that can reach full-plastic moment without local buckling but cannot undertake much ductility afterward, and Category P-IV is for cross-sections failed by elastic local buckling. Similar categories are assigned for beams to account for lateral torsional buckling effect; i.e., Categories L-I to L-IV in accordance with the slenderness ratio about the weak axis as well as the ratio of moment gradient . Finally, a structure consisting of members belonging to P-I and L-I is classified as Category S-I, one with a combination of P-II and L-II as Category S-II, one with a combination of P-III and L-III as Category S-III, and one with a combination of P-IV and L-IV as Category S-IV, respectively (Table1). Furthermore, for columns that are assigned to take inelastic behavior in Categories S-I and S-II, an additional provision is provided to account for $P\delta$ and $P\Delta$ effects, limiting the magnitude of axial force and slenderness.

A unique strength is provided for Categories S-I to S-IV, with the smallest strength for S-I (since it is most ductile) and the largest strength for S-IV (since it is least ductile). This strength ($F_{(req)}$) is a strength required for the structure and is given for each story as:

$$F_{(req)} = D_s F_{e(req)} \tag{17}$$

Where $F_{(req)}$ and $F_{e(req)}$ are the required strength for the story concerned and the strength required for that story if the structure would respond only elastically. The term, D_S, designated as the structural characteristic coefficient, is a factor to account for the ductility capacity possessed by the structure, and its values for unbraced frames are recommended as in Table 2.

5.3. Direct Ductility Check Using Energy Method

As discussed before, the energy to be dissipated by hysteresis is primarily a function of the total mass and fundamental natural period for multi-story building structures, but distribution of the energy to respective stores varies a great deal on the strength profile along the story. Then, the conventional seismic design procedure, introduced in the previous section, treats energy dissipation and ductility capacity in a rather crude manner. Here, introduced is a design procedure, which is more complex than the conventional procedure but enables us to compare the energy dissipation capacity of a structure directly with its energy dissipation demand. In this procedure, we first assign a static seismic load profile along the story of the structure. The profile should be selected so that the collapse mode under this profile of static loading be the same as the collapse mode achieved under earthquake loading. Fortunately, we can find such a profile in most cases, as demonstrated by the test described below[14].

Using the test structure shown in Fig.13, two types of test: one loaded statically and monotonically with a load profile of 1 (1st story); 2 (2nd story); 3 (3rd story), and the other pseudo-dynamically, were conducted. For the latter test, the on-line computer control method[15,16], whose detail will appear in the next section, was employed. Figure 14 is a graphical presentation of possible collapse mechanisms of the structure. Each plane of the polyhedron corresponds to a possible collapse mechanism, where F_1, F_2 and F_3 are the horizontal forces applied to the first, second, and third stories, and a set of (F_1, F_2, F_3) at a point on the plane gives a collapse load. An arrow from the origin, denoted by P, shows an increasing magnitude of the load combination employed in the monotonic loading test, where the ratio between F_1, F_2 and F_3 is kept as 1:2:3. At the instant when the arrow passes through plane b, collapse occurs in the mechanism represented by plane b (Fig.15). The on-line test was conducted for the identical test structure for examining the earthquake response behavior of the structure subject to earthquake loading, In the test, the N-S component of the ground motion recorded in El Centro earthquake(1940) was used. The inertial forces induced at respective stories, also denoted as F_1 (1st story), F_2 (2nd story), and F_3(3rd story), are the important parameters for investigating the correlation between the monotonic loading and earthquake response behaviors. Two figures in Fig.16, showing the trajectory of inertial forces in the F_1, F_2 and F_3 space, plot the projections of the trajectory with respect to the F_1-F_2 and F_2-F_3 planes, respectively, and vector P defined in Fig. 14 is also projected to these planes. It is very informative that *projections of the trajectory of inertial forces entangle around the projection of P, and* overall, they direct to the axis of vector P. This fact suggests that a dynamic collapse mode is likely to be the same as a static collapse mechanism loaded with a suitable static load profile.

If an appropriate static load profile is given, we can also estimate whether or not the structure possesses sufficient energy dissipation capacity. This can be accomplished by static analysis with a given load profile. The structure is loaded monotonically until the energy dissipated arrives at the energy dissipation demanded to the structure. Then, deformation induced in each structural member at this instant is examined against its ductility capacity. A prerequisite for conducting such analysis is to have a tool that can simulate complex behaviors of structural members involving instability effects like buckling, fracture, and others. Advanced hardware and software now available for numerical computation is considered to have made such analysis practicable.

6 Verification of Seismic Performance of Structures

For the progress of seismic design, seismic performance of structures designed should always be verified, and validity of the seismic design adopted be calibrated. Structural testing has been recognized as the ultimate tool for such verification and calibration. In what follows, the writers would like to present their perception about the objectives of structural testing, introduce the on-line computer control test, referred to as the on-line test in this paper, which is believed to be very useful in giving rich information on seismic performance of structures, and discuss its functions in light of the further advancement of seismic design.

6.1 Roles of Structural Testing

To the writers' best knowledge, the experiment has two basic functions. One major function is to prove the accuracy of our predictions regarding the structural behavior. Of course, these predictions have a variety of sources like hypotheses developed based on previous findings and experiences, theories devised based on mathematical elasticity and plasticity, simulation models based on numerical analysis, and others. The second major function of the experiment, which is contingent to the first function as the proving mechanism, is to give us new learnings about the structural behavior. Such new learnings can be obtained when the results of the test reveals discrepancy from what had been predicted prior to the test. These new learnings encourage us to open new research subjects, whose solutions, in turn, are to lead us to further advancement of structural analysis. These two functions of the experiment are also the power to the progress of structural design. Proof tests of structures, such as those designed with new concepts and those incorporated with new materials, are a tool for judging the validity of the concepts and performance of the materials. Furthermore, any unexpected performance observed in such proof tests provides us with new incentives for advancing our structural design.

6.2 Roles of On-Line Test

The on-line test is an experimental technique for simulating the earthquake response behavior of structures and structural subassemblages with respect to the time domain, but, unlike the shaking table test, the test is conducted quasi-statically. The basics of the test and previous applications are well documented elsewhere [15,16]. To facilitate the discussion on the roles of the on-line test, the on-line test is compared with the most conventional quasi-static loading test with a predetermined loading history (referred to simply as the quasi-static test hereinafter).

As far as the required test facilities and procedures are concerned, the on-line test and quasi-static test are essentially the same. In both tests, the test specimen fabricated is set on the test floor, attached by load applying devices and measuring sensors, loaded slowly with occasional pauses for data collection. The only difference between the two tests is the loading history imposed in the test. In the quasi-static test, the entire loading history is prescribed prior to the test, with its history determined by the researcher's choice, whereas, in the on-line test, the history is created in parallel to the loading rather than prescribed a priori, with the mechanism of history creation as solving the associated equations of motion. In reference to the roles of the experiment, both quasi-static and on-line tests are equally useful in providing benchmark information for calibrating the accuracy of our predictions. If one wants to acquire data on the capacity of the structure such as the maximum resistance, ductility, cumulative ductility, energy dissipation capacity, and others, the quasi-static test is no doubt the most effective experimental tool. On the other hand, if one wants to obtain data on the complex hysteretic behavior of the structure under earthquake-like loading, say, for the purpose of evaluating the accuracy of a numerical model, the on-line test is useful for generating such data. As to the function of the design-oriented proof test, the on-line test is also capable when validating the performance of the structure under earthquake loading.

Both the quasi-static and on-line tests are effective experimental tools by all means, with the quasi-static test employed more for capacity verification and the on-line test more for performance evaluation. In this regard, the quasi-static and on-line tests are considered to be complementary.

6.3 Design Implications of On-Line Test Results

Based upon the discussion on the roles of experiment, particularly the on-line test, the writers would like to view how the on-line test results can contribute to advancing our seismic design.

Proof Test: The on-line test is certainly a useful means for proving the earthquake resisting performance of structures designed, thus enabling us to judge the validity of the designs adopted. As loading is made quasi-statically and can be paused any time upon request, the on-line test makes possible close observation on the detailed behavior of the structure under earthquake loading, and capacity of this close observation is the most significant asset of the on-line test in its function as a proving tool.

Design Earthquake Force: As discussed earlier, the equivalent lateral force concept has been widely accepted in seismic design, and, using the forces, static analysis is made for proportioning and sizing structural members. Then, how effectively the equivalent lateral forces represent the forces induced during earthquake motion is the key for the success of the design. Various hypotheses have been proposed to determine the magnitude and distribution of the equivalent lateral forces, and the on-line test results can serve as realistic background information for evaluating the accuracy and limitation of these hypotheses. Some such efforts have been presented [17,18].

Calibration of Numerical Models Used in Structural Design: Recently, in the final stage of structural design, the structures designed are often checked for their earthquake resisting performance by employing numerical time-history analysis. In such analysis, numerical models that represent the restoring force characteristics of stories, elements, or materials are to be assigned, and the accuracies of the models employed are the key for obtaining reliable results from the numerical analysis. Again, in light of the functions of the experiment, the on-line test is a useful means for calibrating the effectiveness of individual numerical models, and, in fact, many previous on-line tests were carried out for this purpose. On recent effort along this line is introduced below. Developed was a database containing the results of many previous on-line tests applied to steel structures, calibrated the accuracies of several simple numerical models in terms of their ability to simulate the maximum resistance, maximum deflection, permanent (residual) deflection, cumulative ductility, energy dissipation capacity, etc., and discussed the overall capacity as well as limitation of these numerical models[19].

Failure Criteria: We use a term "failure" quite often in our structural environment, and, without much doubt, we assume this term as one of the most important indicators for structural design like: "we should design a structure so that it does not fail even under severe earthquake loading." But what is failure? Does this mean a complete loss in resistance of the structure, or a maximum damage that can be tolerated by the society? Then, what is a tolerable limit of damage? This so-called "failure criteria" is yet a very ambiguous term and has hardly been quantified. So far, our experiences and heuristics seem to be the major sources that lead us to a (subjective) consensus regarding "failure", and nothing else but the detailed information on structural damage is the resource that can make our experiences and heuristics richer. Again, the on-line test can serve as a useful mechanism for generating such detailed information. Remember that the on-line test can provide us with thorough data on the resistance, deflection, hysteresis, stress, strain, energy dissipation, damage propagation, and others of the structure tested under earthquake loading.

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Table 1 Structural classification

		Width-Thickness Ratio			
		P-I	P-II	P-III	P-IV
Slenderness Ratios	L-I	S-I*			
	L-II		S-II*		
	L-III			S-III**	
	L-IV				S-IV**

* Plastic analysis ** Elastic analysis

Table 2 Structural characteristic coefficient D_S

	S-I	S-II	S-III	S-IV
D_S	0.25	0.30	0.35	0.45

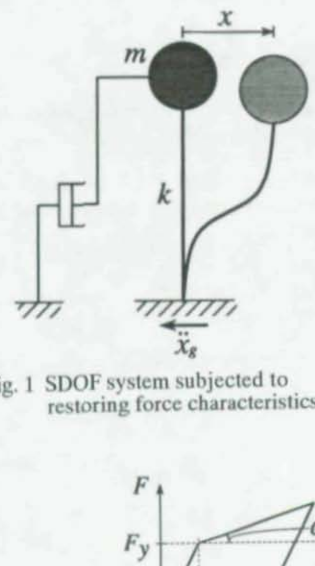

Fig. 1 SDOF system subjected to restoring force characteristics

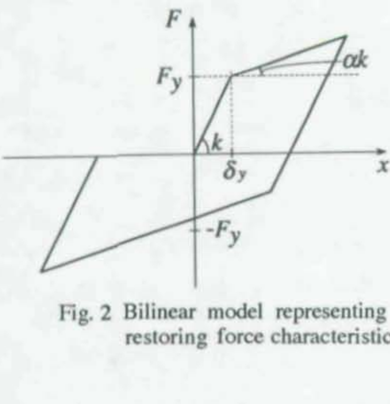

Fig. 2 Bilinear model representing restoring force characteristics

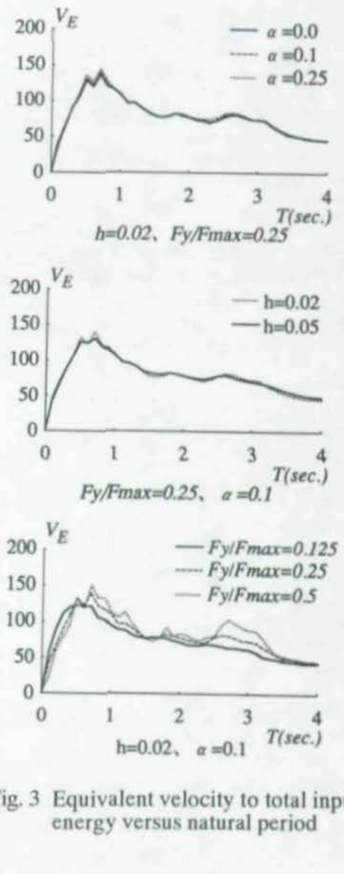

Fig. 3 Equivalent velocity to total input energy versus natural period

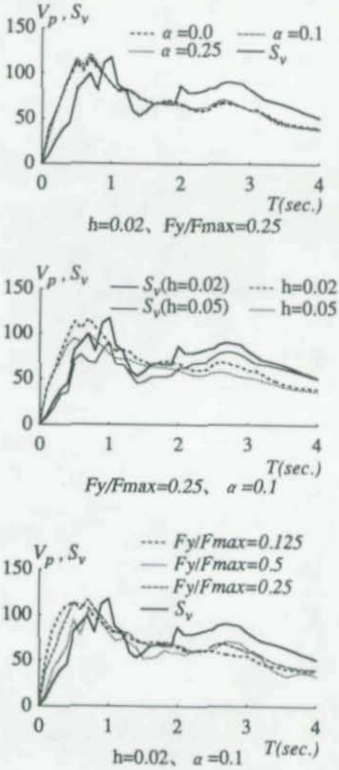

Fig. 4 Equivalent velocity to dissipated energy versus natural period

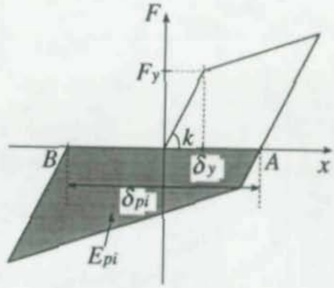

Fig. 5 Correlation between hysteretic energy dissipation and ductility

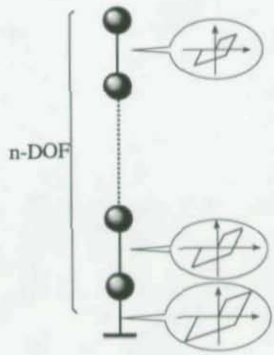

Fig. 6 MDOF system

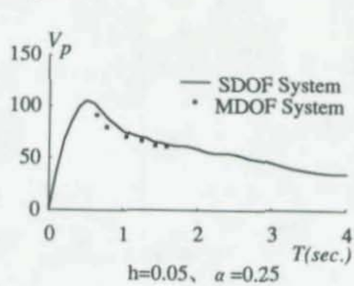

Fig. 7 Equivalent velocity to dissipated energy of SDOF and MDOF systems

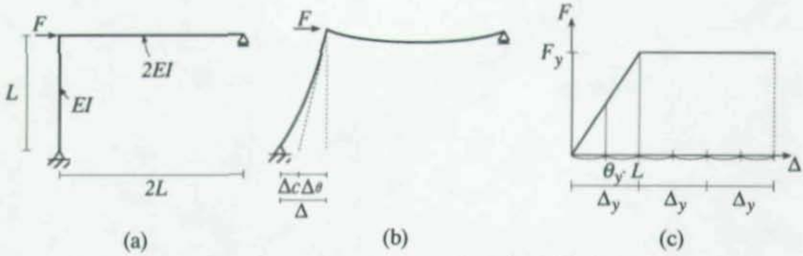

Fig. 8 Ductility demands to a story and an individual member

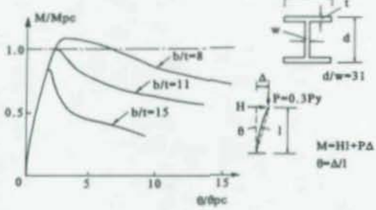

Fig. 9 Resistance loss due to local buckling

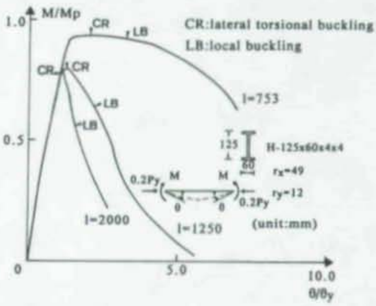

Fig. 10 Resistance loss due to lateral torsional buckling

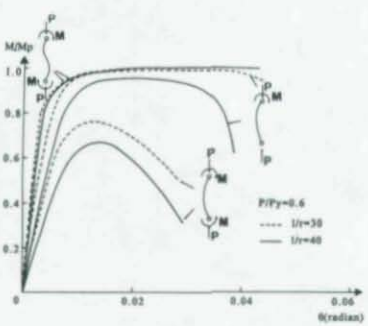

Fig. 11 Resistance loss due to Pδ effect

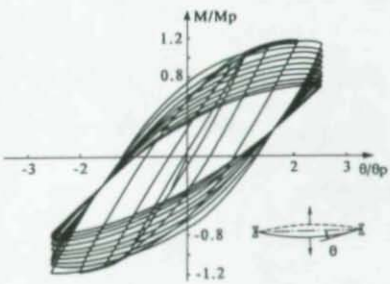

Fig. 12 Strength deterioration under reversed and repeated loading

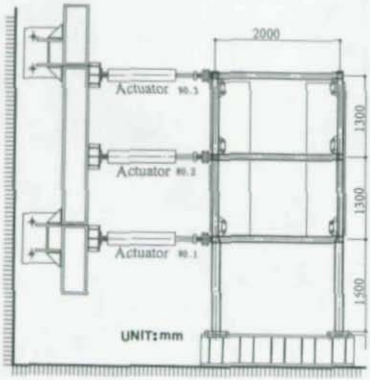

Fig. 13 Test set-up of a test structure

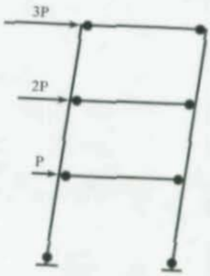

Fig. 15 Collapse mechanism under a specified load profile

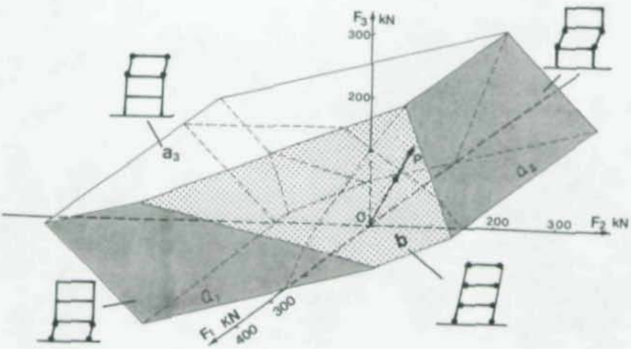

Fig. 14 A polyhedron showing collapse mechanisms

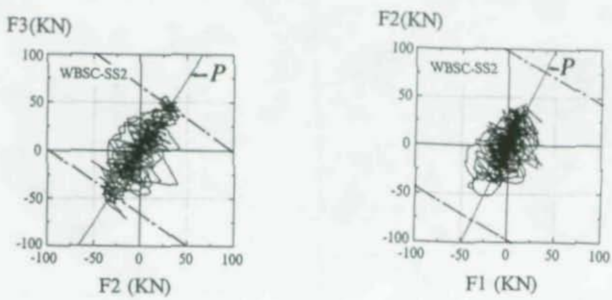

Fig. 16 Trajectory of inertial forces

SOME PROBLEMS OF STABILITY OF STEEL STRUCTURES UNDER SEISMIC LOADING

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Stability of steel members designed for seismic requirements generally requires more stringent buckling criteria than is needed in design for gravity loads. Some of the reasons for the need of a more conservative design are highlighted in this paper. Specifically, the behavior of single struts under monotonic and cyclic loadings is described, followed by a discussion of concentrically braced frames (CBFs). Some procedures of design and special devices for extending the life of CBFs under seismic conditions is included. A brief reference to eccentrically braced frames (EBFs) and their use in seismic design for avoiding buckling is indicated. This is followed by a description of seismic behavior of moment resisting frames and the measures taken to improve their behavior.

Introduction

A seismic event at a fault line can cause violent earth displacement such as shown in Fig. 1. The forces generated by a severe earthquake may be so enormous that no structure can be designed to withstand them. Therefore as far as possible important facilities such as nuclear power plants, hospitals, police stations and schools are built away from potential faults. For locations away from an active fault it is possible to design engineering structures to resist imposed accelerations by an earthquake, such as shown in Fig. 2 recorded near a fault. In such cases structures respond by vibratory motion for which measures can be taken to prevent severe damage. A representative respoonse of a building designed to resist seismic loading is shown for a simulated earthquake motion in Fig. 3. Note that because of the inertia of a structure fewer cycles per unit of time occur.

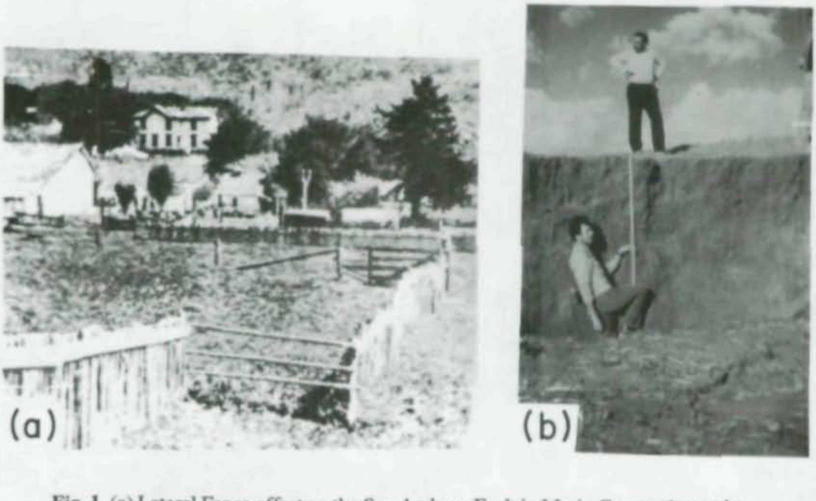

Fig. 1. (a) Lateral Fence offset on the San Andreas Fault in Marin County due to the 1906 San Francisco Earthquake (Photo by G.K. Gilbert); (b) Normal Fault Scarp near Beni Rashed during 1980 Algeria Earthquake (Courtesy of V.V. Bertero).

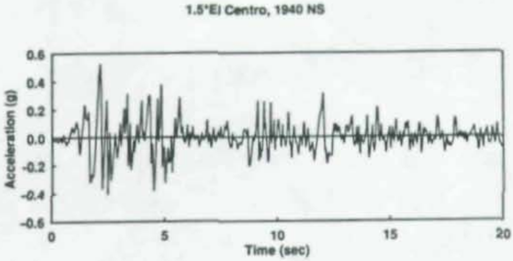

Fig. 2. Ground Acceleration vs Time during 1940 El Centro Earthquake (NS direction).

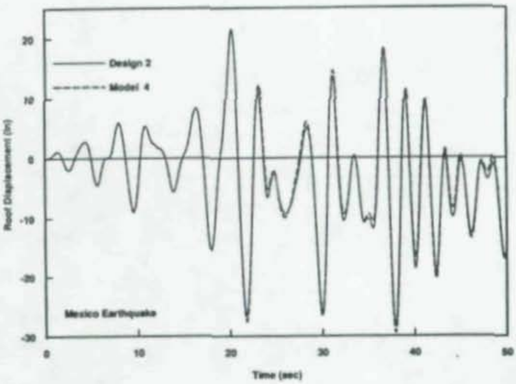

Fig. 3. Simulated Roof Displacement of a 6-story Steel Moment Resisting Building due to the 1985 Mexico Earthquake.

Behavior of Struts in Compression and Tension

The mechanical behavior of a rectilinear coupon of virgin steel under monotonically applied tension or compression may be approximated by the familiar bilinear response. The first part of this response can be assumed to be linearly elastic, followed by plateau corresponding to an ideal plastic behavior. Calculations show that a slender member of such an idealized material in compression exhibits the behavior first shown analytically by von Karman (1909), Fig. 4. This figure shows that even an ideally straight strut on reaching maximum capacity in compression rapidly deteriorates on further loading or disturbance. The other curves shown in the figure illustrate the behavior of compression members with an initial out of straightness e. In the U.S., commonly wide-flange shapes are required to have a maximum initial crookedness of less than $L/1000$ (Galambos, 1988), where L is the length of a member. This corresponds to the curve shown in Fig. 4 just below the one for an ideally straight strut. The precipitous drop in capacity after reaching the maximum must be clearly kept in mind.

The situation for steel struts subjected to repeated and reverse loading is much more deliterious, as following the first loading cycle, a dramatic decrease in capacity takes place in the subsequent loading cycles, Fig. 5. This is caused by a large decrease in stiffness in the material, measured by E, due to the Bauschinger effect, during the subsequent cycles of loading (see Fig. 6). Therefore it is to be expected that concentrically braced steel structures during cyclic dynamic loading such as occur in an earthquake would perform poorly.

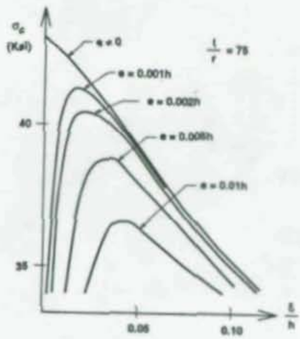

Fig. 4. Behavior of Monotonically Loaded Elastic-Plastic Columns with Different Amounts of Out of Straightness.

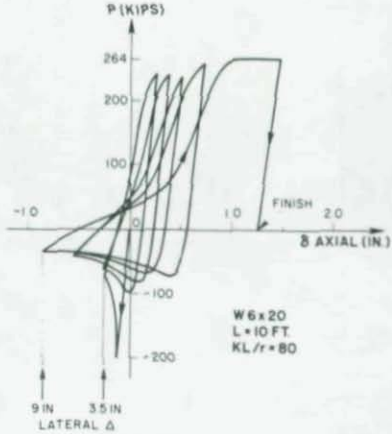

Fig. 5. Cyclic Behavior of a Column.

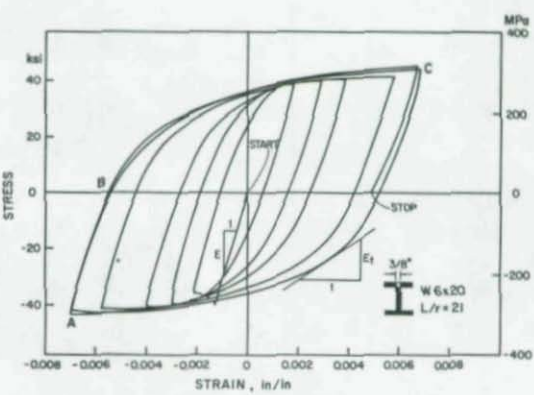

Fig. 6. Cyclic Behavior of a Short Steel Rod Demonstrating Bauschinger Effect.

It is important to note, as shown by the dashed lines in Fig. 7, that even a small excursion into tension can significantly decrease the capacity in compression. For the case illustrated, a virgin specimen, subjected to a compressive force, reached a magnitude of 201 kips (894 kN); wheras a similar specimen initially caused to yield a small amount in tension, on a return stroke, attained a maximum compressive force of only 152 kips (676 kN), i.e., only 75.6% of the first specimen.

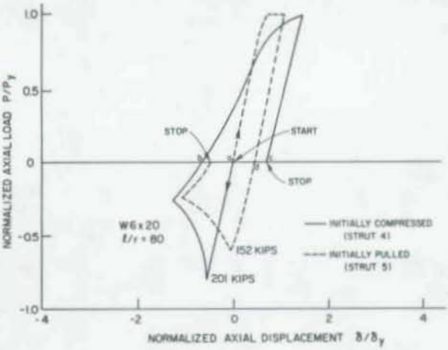

Fig. 7. A Steel Strut Initially Subjected to Yielding in Tension Decreases Its Compressive Strength Compared to Its Behavior in Virgin State.

Concentrically Braced Frames (CBFs)

Poor behavior of CBFs under extreme cyclic loading conditions, simulating what may occur during a severe earthquake is illustrated in Fig. 8 (Ballio and Plumier, 1985). For the frame braced by a single diagonal, Fig. 8(a), only the first excursion of the brace into the compression range is satisfactory. The lateral strength capacity of the frame when the diagonal is in compression during the subsequent cycles is greatly reduced. The situation for the X-braced frame, Fig. 8(b), is better. However the frame capacity progressively deteriorates. This type of behavior may occur in many CBFs.

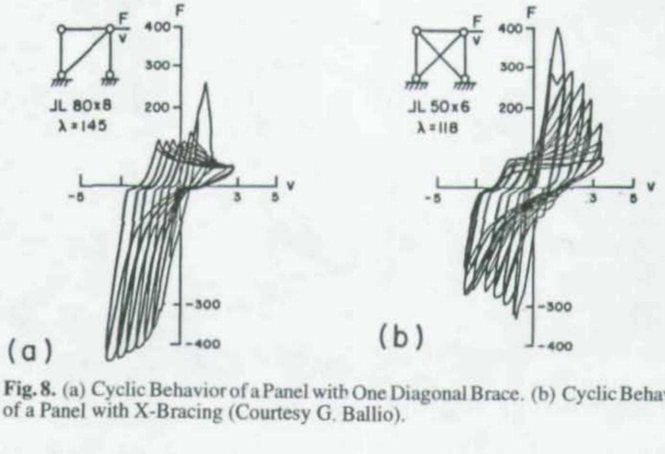

Fig. 8. (a) Cyclic Behavior of a Panel with One Diagonal Brace. (b) Cyclic Behavior of a Panel with X-Bracing (Courtesy G. Ballio).

Examples of unstisfactory behavior during cyclic loading is documented for more complex framing from laboratory experiments and from actual behavior during earthquakes. Full-scale seismic testing of a six-story steel structure in the cooperative US-Japan program, having one CBF and moment connections throughout the four 24.6 ft (7.5 m) square bays, resulted in several failures in the braces when subjected to simulated Miyagi-ken-Oki accelerations (Foutch et. al., 1987), Fig. 9. Shake table experiments of a model of the same structure at the University of California, Berkeley, by Uang and Bertero (1986) exhibited similar failure of the chevron braces.

During the January 17, 1994, Northridge earthquake in Los Angeles similar buckling or fracture failures in chevron CBFs were also observed. As an example, several braces largely in the first floor of a moderatly high office building failed in the above manner. These braces were of 12" × 12" 3/8" (305 305 9.5 *mm*) structural steel tubing. An example of fractured tubing is shown in Fig. 10; several of the braces failed in buckling.

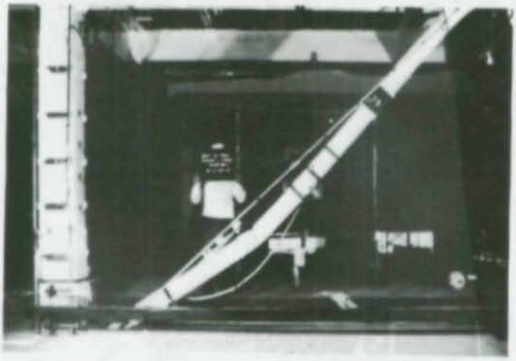

Fig. 9. Buckling of a Diagonal Brace in Full-Scale Tsukuba Test (Courtesy R.D. Hanson).

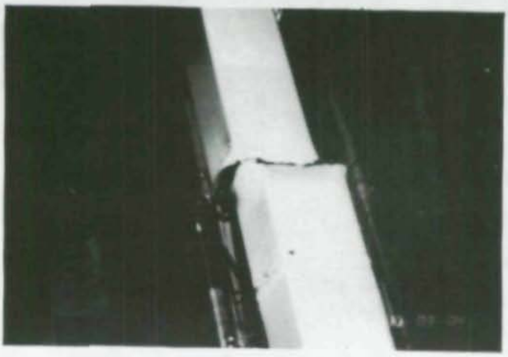

Fig. 10. Fracture of a Diagonal Brace in Full-Scale Tsukaba Test (courtesy R.D. Hanson).

The reason for poor performance of steel tubular bracing can be directly attributed to inadequate code provisions. For example in the widely used Uniform Building Code (ICBO, 1988), for rectanglar box sections of uniform thickness it is required for the width-thickness ratio not to exceed $238/\sqrt{F_y}$ in US conventional units, where F_y is the specified minimum yield stress in kips per square inch.

Even before the 1994 Northridge earthquake the Structural Engineers Association of California (SEAOC) was considering adopting a more stringent requirement for the width-thickness ratio for rectangular tubes per AISC (1992) Seismic Provisions; now even more stringent requirements may be adopted. In these provisions the width-thickness ratio for rectangular tubes states that a flat-width to wall thickness shall not exceed $110/\sqrt{F_y}$ in the US conventional units. On this basis the 12" × 12" steel tubing in the building noted above, using 46 ksi (317 MPa) steel should have been 5/8 in. (15.8 mm) thick instead of the specified thickness of 3/8 in (9.5 mm). Most likely tubes of the larger thickness would have proven to be adequate.

Awareness by engineers of the buckling problem in CBF braces, either in the Euler or local sense, can be resolved in different ways. The offshore industry solved this problem by employing X-bracing and permitting the braces to buckle **and** carry load, Fig. 11. These drilling and production towers are exceptionally well constructed. The welding process is carefully monitored and frequently **internal diaphragms** are inserted at the joints. Such an expensive method of construction is not favored in building work, and one must look at different methods to resolve this problem. Two of many possible solutions are described here.

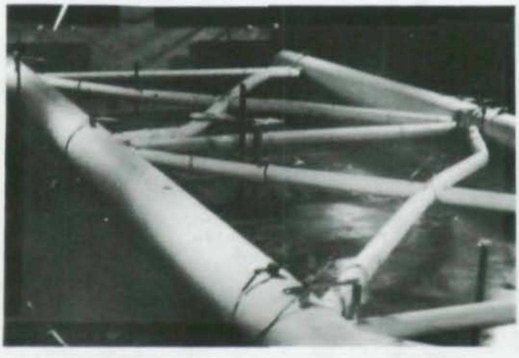

Fig. 11. Example of Completed Test on One-Sixth Scale Model of an Offshore Tubular Steel Frame (V.A. Zayas, et al.).

Recently Nippon Steel Corporation (Nippon, 1993) proposed to use conventional steel members imbedded in concrete within a tube. A bond between the force carrying steel and concrete is prevented by painting steel with a plastic. The function of concrete in a tube is to prevent steel from buckling. A steel member having a cruciform cross-section (two bars welded to a larger bar) is shown in Fig. 12. Excellent hysteretic loops have been generated with a variety of steel crossections under cyclic loads. This proedure has been used in several large buildings.

An alternative means of preventing brace buckling consists of placing calibrated slip joints at the ends of braces, Fig. 13 (Grigorian and Popov, 1993). These friction joints were demonstrated to be excellent energy dissipators. In a typical bolted connection elongated slots in the main connecting plate are parallel to the line of loading. When bolts are tightened a predetermined amount, the main plate is "sandwiched" between the brass insert plates and the outer steel plates. The holes in the steel outer plates and the brass plates are of standard size. Application of cyclic loads of magnitude equal to the slip force resulted in excellent essentially rectangular hysteresis loops.

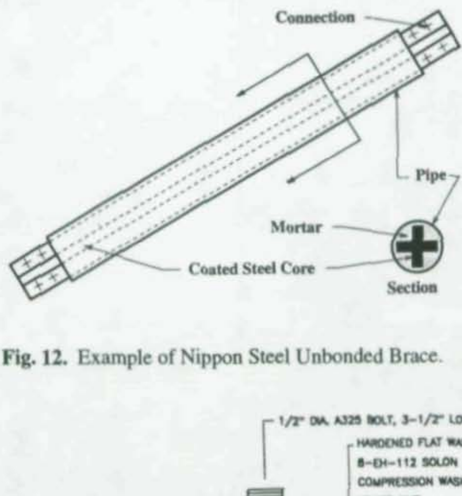

Fig. 12. Example of Nippon Steel Unbonded Brace.

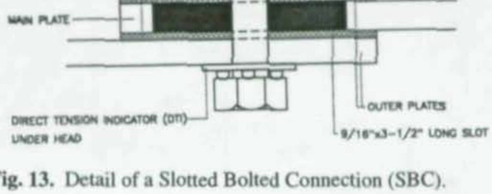

Fig. 13. Detail of a Slotted Bolted Connection (SBC).

The behavior of the slotted bolted connections (SBCs) was verified with numerous individual tests and on an experiment useing a one-bay 20-ft (6.1 m) high steel test structure loaded by approximately 30 kips (13.6 Mg) per floor, as shown in Fig.14, to over 40 tests performed by subjecting the loaded frame to simulated strong horizontal seismic excitations. A typical set of hysteretic loops from one of the twelve SBCs at the first floor due to a strong Chilean signal (Llolleo) amplified to 0.88g peak shake table acceleration is shown in Fig 15. The energy history for the frame for the same Chilean signal is shown in Fig. 16. It can be noted from this figure that the friction energy dissipators (SBCs) very effectively dissipated most of the earthquake input energy. SBCs can greatly reduce frame deflection during an earthquake, and because of their slip characteristic below yield or buckling of braces protect them from damage.

Fig. 14. 20-ft High Test Frame with 12 SBCs.

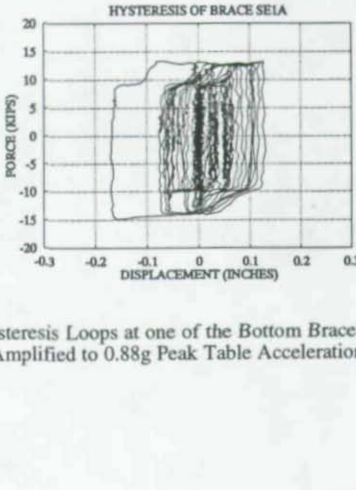

Fig. 15. Representative Cluster of Hysteresis Loops at one of the Bottom Braces Due to the 1985 Chile Llolleo Signal Amplified to 0.88g Peak Table Acceleration (PTA).

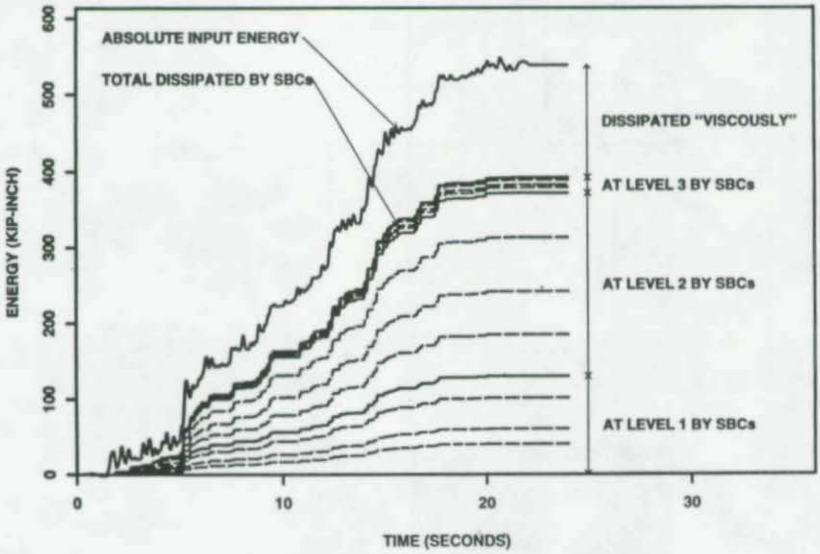

Fig. 16. Comparison of Energy Input and Dissipation Histories for 1985 Llolleo Signal with 0.88g PTA.

Eccentrically Braced Frames (EBFs)

CBFs are very effective and reliable in resisting lateral forces, provided their behavior remains in the linearly elastic range. This is not the case in seismic applications, where for reasons of economy, the braced framing systems are expected to behave inelastically, forcing the critical members to undergo cyclic plastic deformations. As discussed in the previous section, this requires special devices and/or details. An alternative approach to resolving this problem is to brace frame differently by deliberately introducing braces eccentrically at selected joints. Possible framing of this type is shown in Fig. 17. Note that at least one end of each brace terminates in a link of length e. The capacity of these links can be adjusted so as to have the link behave inelastically, before a brace exhausts it **elastic capacity.** This is the basis for design of EBFs. Such systems remain laterally stiff because the inclined braces remain elastic during extreme lateral seismic loads.

In EBF design there are several problems concerned with stability. First, the link itself, being potentially cyclically deformed into the post-elastic range, requires special consideration. Analysis and experiments have clearly shown that such links must be made short to induce shear deformation in the web throughout a link's length, although some bending of the link at the ends is unavoidable. To achieve such action, and to avoid general buckling, the web must be reinforced by vertical stiffeners; and the web itself must not buckle excessively between the stiffeners. An example of a link 36 in. (924 mm) long with two intermediate vertical stiffeners in a W18 beam segment is shown in Fig. 18. Since the web buckled, either it should have been made thicker or the stiffeners spaced closer together. Unlike the case of the link shown in the illustration, in an EBF system, generally, axial forces also must be transmitted through the link.

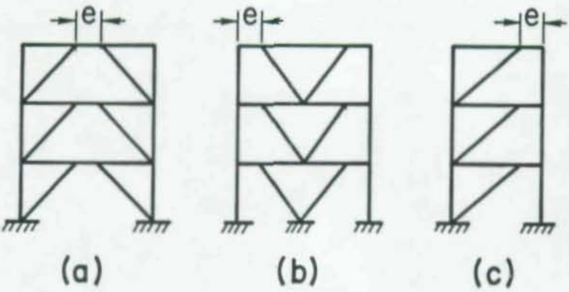

Fig. 17. Examples of Eccentrically Braced Framing.

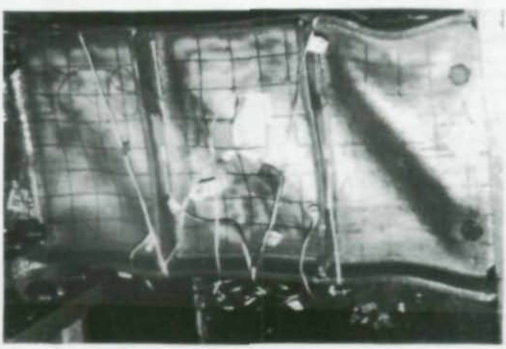

Fig. 18. An EBF Link After a Severe Cyclic Test.

The criteria for the maximum link length needed to induce shear action, and the required web thickness, as well as the required stiffener spacing to obviate the buckling problem, are given in the current codes (AISC, 1992; ICBO, 1988). Two related problems must also be considered: the attachment of the brace to the beam at the link, and the design requirements for the floor beam outside the link.

If a W-shape is selected for the brace, by aligning the brace web with that of the beam the brace flanges can be welded directly to the beam flange. Full-length vertical stiffeners should be provided directly above the flange ends of the brace.

Attachment of the widely used structural tubes requires special attention. An example of an improper detail, first used at Tsukuba on the US-Japan cooperative project, is shown in Fig. 19. Wheras this detail withstood the initially decided upon simulated seismic displacements, on increasing the intensity of the cyclic testing, the gusset buckled and the inherent frame lateral capacity was not utilized. The reason for the failure is obvious in retrospect. When a bending moment on the right is applied to this connection with a clockwise sense (negative moment), the resisting section in bending consists of the $W\ 18 \times 40$ beam and a thin vertical plate which under compressive load has a propensity to buckle. After the failure of this detail, the recomended detail for such cases in EBFs became as shown in Fig.20. A detail of this type was found to be entirely satisfactory in subsequent experiments (Engelhardt and Popov, 1989).

Two additional stability problems arise in the design of the beam next to the link. If the brace angle with the horizontal is small, the length of the panel zone between the vertical end stiffeners over the brace connection becomes large. This distance can be larger that the stiffener spacing in the link, and the top beam flange has propensity to buckle. Additional intermediate vertical stiffeners are advisable in such cases.

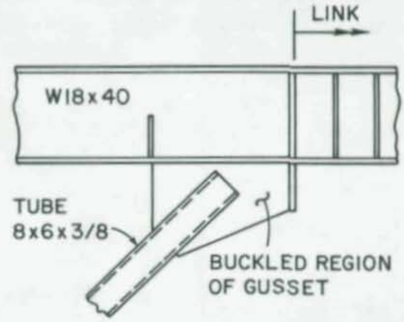

Fig. 19. An Improper Brace-to-Beam Connection.

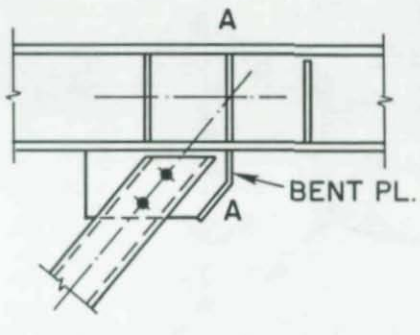

Fig. 20. An Appropriate Connection of a Tubular Brace to Beam.

Lastly, in order to prevent lateral torsional buckling of the beam, the beam must be laterally braced at both ends of the link, and along the beam outside the link and the connection panel zone. The latter requirement is important as in EBFs the beams along this length carry a large axial force, augmented by the horizontal component of the brace force and end moment from the link.

Moment Resisting Frames (MRFs)

The most widely used steel construction utilizes MRFs. This type of framing provides a maximim of unobstructed space. Again, for seismic design, several of the structural design criteria for buckling requirements are made more demanding than those for carrying gravity loads. A key element in the MRF construction is the beam-column joint; a typical joint in a deformed state is shown in Fig. 21. This type of connection is largely based on the research initiated by Huang et al. (1971) for static loads, and by Popov and Stephen (1972) for cyclic loads,

The later reference recomends, in seismic design, that beams generally have thicker webs and flanges than in the design for static loads. The current AISC (1992) Seismic Provisions require the width-thickness ratios in the US customary units not to exceed $52/\sqrt{F_y}$ for flanges. The cooresponding requirement for static loads for compact sections is $65/\sqrt{F_y}$. Similar increases for web thickness for seismic applications are given. These more stringent requirements are promulgated because in a cyclic regeme, if inelastic action takes place, the Baushinger effect, illustrated earlier in Fig. 6, becomes important and reduces the capacity of members.

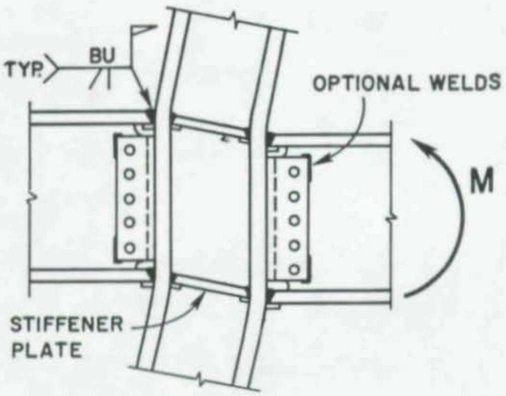

Fig. 21. Typical Beam-to-Column Flange Connection for MRFs.

It is important to recall that the 1972 tests were made for a specific project where a premium was paid for bolted rather than welded web connections. Moreover, the number of bolts in the shear tab was kept as large as possible. These connections with bolted webs and welded flanges performed well, although they were not as good as the sole all-welded connection. Photographs of two specimens with bolted webs at the end of each test are shown in Fig. 22. Note that in one of these *specimens the bolting clamping force was more effective than in the other. Unfortuanately in current* practice the number of bolts at the connection is usually determined from shear requirements.

Cyclic hysteretic loops that were obtained for the specimen with a smaller bolt clamping force, as evidenced by less whitewash that pealed-off, is shown in Fig. 23. The failure of this specimen occurred very abruptly with a "bang". Subsequent research by Tsai and Popov (1988), and Engelhardt and Hussain (1992), showed that with progressively simplified details and fabrication procedures, the reliability of such connections can be questionable.

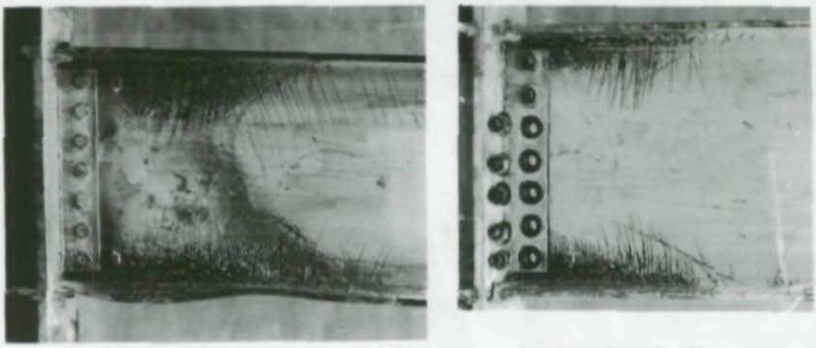

Fig. 22. Cyclic Tests of Two Beam-to-Column Joints of $W\ 24 \times 76$ Cantilevers. The 7/8 in. Left Beam Bolts Provide Better Clamping Force than the 1 in. Right Beam Bolts Showing Erratic Behavior.

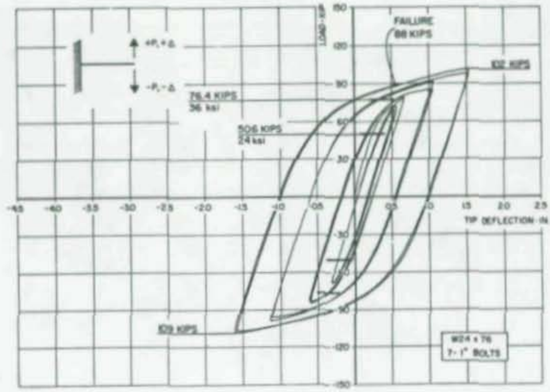

Fig. 23. Hysteretic Diagram for the Beam with Bolted Web in Fig. 22.

Another important aspect associated with these connections is the continuity plate that transfers the force developed by the beam flanges to the column web. The tendency in practice is to reduce their thickness to a minimum, and to weld them to the inside of column flanges by fillet welds. It is believed that such welds are satisfactory in welding these plates to the column web, but full penetration welds are preferable for welds to the inside column flanges. The extent of cyclic buckling activity in the continuity plate is shown in Fig. 24 by the peeled whitewash.

The emerging technology for using nonlinear methods of buckling analysis, already in use in aircraft design and offshore engineering, is beginning to be felt in the more traditional applications (Chen and Toma, 1994). This topic is beyond the scope of this paper.

Fig. 24. Continuity Plate Between Column Flanges. Shear Plate for Connecting Beam at Right Angles to the Test Specimen is in the Middle of the Panel Zone.

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Connection Restraint Characteristics and their Influence on Beam and Frame Behaviour and Design

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Abstract

It is argued that, with research into the stability of members provided with ideal conditions of support and frames that utilise perfect forms of joint having reached the stage where – at least in principle – methods for analysis and design based on a considerable volume of evidence and understanding are well established, studies of the influence of practical forms of interconnection between members on stability are likely to be a more cost-effective line of research. Progress in this area – particularly that relating to the development of an appreciation of connection restraint characteristics – is reviewed. It is concluded that for the two-dimensional response of bare steel frames understanding is now comparable to that achieved for individual members and pinned or rigidly connected frames. Progress made within this area during the past decade has had the effect of encouraging comparable studies of other types of connection restraint problem and illustrations of some of these are provided.

Introduction

Research in structural stability (1) has traditionally been concerned with the behaviour of elements – struts, beams, beam-columns, plates – or systems – frames, plate assemblies, shells. For the former, support conditions are normally assumed to be "ideal" i.e. each particular degree of freedom at a restrained cross-section – deflection or rotation – is assumed either to be prevented or to be free. For systems, inter-connection between different elements is similarly assumed to be "perfect", providing either full continuity or restraint against deflections only with rotation of one element relative to another being free to take place.

By employing these assumptions, considerable advances in the understanding of the buckling of members due to compression, bending and combined loading as well as of the complex interactive and distortional buckling of structures such as sway frames and stiffened box girders has been achieved. Indeed, when asked to prepare a state-of-the-art review on the subject of the lateral-torsional buckling of beams for the East European part of the 2nd regional colloquium on "Stability of Steel Structures" in 1986, the author concluded (2):

"Future research should be increasingly directed towards the consideration of beams as parts of structures, taking due account of the varied and complex loading and restraint conditions encountered in practice".

If this view is accepted, then it is hardly surprising that the past 25 years have seen such a growth of interest in gaining a better understanding of the actual restraining actions provided by the different practical forms of inter-connection between members and thence in making more realistic allowances for these in stability analyses. Although the topic of major interest has been beam-to-column connections in multistorey frames, several other stability related topics may be identified as having received similar treatment. This review concentrates on the beam-to-column problem as its development illustrates all the key aspects of the subject area. Some mention is made, however, of related topics as a way of indicating the potential for similar advances in other subject area.

Basic Problem

The importance of connection restraint may be illustrated using the most basic structural stability problem: the axially loaded initially perfectly straight, elastic strut of Figure 1. When end translation is prevented, then increasing the rotational stiffness of the end connections from zero (pinned) to infinity (fixed) increases the critical load by a factor of 4. In terms of the commonly used design concept of effective length, this is reduced from L to 0.5L. Thus the potential for variations in load carrying capacity due to changes in restraint conditions is enormous; to effect similar changes by altering the cross-section would require either large increases in area or a significant re-arrangement of material into a more efficient shape.

More realistic ultimate strength analyses for end restrained struts were produced by a number of authors in the 1980's (3). Several of these linked column buckling to the characteristics of the beam to column connections by relating the stiffness of the end restraint to the shape of the connection's moment-rotation (M-ϕ) curve. This led to detailed examinations of the results of previously conducted connection M-ϕ tests (4-7), the development of methods to mathematically represent M-ϕ curves (8,9), the establishment of databanks of connection test results (4,6,7) and various attempts at analytical, numerical or semi-empirical predictions of moment-rotation behaviour (10,11). The subject was thus shown to appeal to a spectrum of research interests – from fundamental stability studies, through laboratory testing to the inclusion of semi-rigid joint action in design rules.

M-ϕ Characteristics

The single most important representation of connection restraint characteristics has been the M–ϕ curve. This relates the moment transmitted by the joint M to the rotation of the beam relative to the column ϕ. Five representative M-ϕ curves (including the two axes, which correspond to the ideal cases of fixed and pinned) are provided in Figure 2. The three intermediate curves correspond to a high moment capacity and high rotational stiffness connection, a partial strength and semi-rigid connection and a low strength and low stiffness (pinned) connection as illustrated in Figure 3 Typical examples might well be an extended endplate with suitable column stiffeners, a moderately thick flush endplate without stiffening and a pair of web cleats, although since it is the relative stiffness that is important, the connection must be considered in the context of the beam and column sizes as well.

This concept has been formalized in what is arguably our most up to date structural steel design code – Eurocode 3 – through the notion of classification of connections as shown in Figure 4. In principle, this is similar to the separation of members into different classes based on the susceptibility to local buckling of their plate elements (flanges and webs) for the purposes of determining their cross-sectional flexural and compressive capabilities and deciding whether they may participate in plastic hinge action. The "classes" of connection are rather different as shown in Table 1; their role is, however, effectively the same in that they control the approach that should be taken to the determination of the distribution of internal forces in the structure. Thus this represents a formalisation in a quantitative fashion of the processes implicitly used by designers e.g. in the UK so-called "simple construction" in which the joints are assumed to behave as pins has long been an established and popular method of frame design (12), with the important requirement that when connections fall into the semi-rigid and/or partial strength category then the frame should be designed recognising its semi-continuous (the equivalent term in the AISC LRFD specification is "partially restrained") nature.

Several reviews of beam to column connection restraint characteristics – effectively M–ϕ behaviour – are available (4-11). From these 4 approaches to the determination of such information and its presentation in a form that is suitable for subsequent use in the analysis of stability problems may be identified:

- laboratory testing of joints – direct determination of M–ϕ characteristic(s) for particular joint(s) tested.

- numerical simulation e.g. through the use of the finite element method – replacement for physical testing.

- curve fitting sets of test results – permits interpolation over a wider range of parameters, coverage limited by availability of test data.

- behavioural models employing simplified analytical representation of key facets of structural behaviour plus inclusion of some coefficients whose values are obtained from comparisons with test data – can provide design equations for those joint types for which simple models of the behaviour of the principal components may be constructed.

Combinations of the first and third of these are quite advanced, with a wide range of empirically based formulae capable of representing M–ϕ curves with varying degrees of accuracy now being available. Within Europe the cooperative COST-C1 project (13) is seeking through the SERICOM computerised databank (14) to improve access to the many hundreds of test results that are available for use with curve fitting approaches. Starting from the original work of Frye and Morris in 1976 (5), considerable advances have been made in improving the accuracy and reliability of the prediction formulae as well as giving them a greater physical basis.

This leads naturally to the so-called behavioural models, an early example of which was the work of Lee and Melchers in 1986 (15) on bolted endplate connections, in which an exponential equation was proposed. This employed the three physically based quantities:

- plastic moment capacity of the connection
- initial elastic connection stiffness
- strain hardening connection stiffness

each of which may be determined from simplified analysis, plus a single coefficient obtained from test data. Several improvements and extensions to this approach have been made, notably by Chen and his associates (16-21), see Figure 5.

An alternative, rather ingenious approach has been used by Richard and his associates (22) for fin plate and web angle cleat connections. In this, the detailed examination and assessment of the behaviour of the key component parts of the connections was undertaken through a combination of finite element analysis and laboratory testing of these component parts. This led to the establishment of a "conceptual model" of the connection as a set of springs and bars of the type illustrated in Figure 6 to model the main load transfer functions. Similar "conceptual models" have been used in Europe by Tschmernegg (23) and by Jaspart (24). In a recent development (25) a connection element, capable of representing all six degrees of freedom (three deflections and three rotations) providing suitable connection data are available, that may simply be added to standard frame analysis packages, has been proposed.

Use of M-ϕ Data in Frame Analysis

Initially frame analyses that included the effects of connection restraint were limited to the studies of the enhanced strength of axially loaded columns referred to earlier. Moving through beam-columns, which allowed for studies of alternate load paths some of which required larger connection rotations and thus involved the use of lower stiffnesses from the higher regions of the M-ϕ curves, on to subassemblages, which permitted the transfer of moments from beams through connections into columns, and then to both non-sway and sway frames, the subject has now matured with various levels of sophistication being used for both the connection models and the frame analysis itself. Connection models covering behaviour under cyclic loading have permitted the examination of frame response under complex load histories such as reversals of wind loads or seismic spectra. The level of maturity may best be appreciated by considering something of the content of two recent (1993) research papers:

"Stability Behaviour of Semi-Rigid Sway Frames" (26)

This study, which is at the more "academic" end of the range, considers the elastic response under static loading of the set of 1 and 2-bay, fixed base portal frames shown in Figure 7 allowing for several combinations of:

- various forms of connection M-ϕ representation
- column loading only or distributed beam loads
- geometrical sway imperfections
- cyclic wind load
- different levels of rigour in the frame analysis

Much of the emphasis of the analysis is on bifurcation with a typical result being given in Figure 8. Results are compared and discussed and the main conclusion is that the use of an effective length form of design approach for sway frames containing semi-rigid joints is unreliable.

An interesting complement to this is the series of studies completed recently to provide justification for the use of the popular yet "not quite respectable" wind connection method (27), that have attempted to specify those combinations of frame proportions, joint types, serviceability considerations and column design methods for which the approach may safely be employed.

"Seismic Performance of Moment Resisting Frames with Flexible Joints" (28)

This study investigated the response to seismic loading of a representative 8 storey x 3 bay x 4 bay frame designed to "normal" Canadian code provisions, allowing for various forms of connection behaviour. The DRAIN-2D package was used with joint models devised from the authors' own experimental work on extended endplates, particular attention being paid to the separate and combined efforts of joint flexibility within the beam to column connection and the column panel zone as illustrated in Figure 9. Five measures of frame response were compared:

- lateral deflections
- storey shear
- storey drift
- plastic rotations in the joints
- panel zone ductility

As a result recommendations for the most efficient design approach, incorporating joint flexibility as an extra parameter under the control of the designer, for sway frames subject to seismic loading were made.

Many similar investigations could have been cited and the reader is referred to the recently published book by Chen and Liu on frames (29) for a review that is up to date as of 1991. The conclusion to be drawn from this is that work on the two-dimensional behaviour of bare steel construction has now reached a level of maturity that permits considerable choice in the form of connection modelling, has contributed significantly to our understanding of the interactions between joint characteristics and either member or frame response and has become sufficiently well accepted for it to feature in codified design rules.

Extensions

Going beyond the basic use of M-ϕ data in 2-D frame analysis, the following additional effects have been studied:

- design oriented procedures that incorporate connection restraint as an additional variable to be controlled by the designer (30,31)

- separate consideration of the different joint flexibilities in connection restraint modelling (22,23)

- inclusion of shear and axial flexibility of connections in frame analysis (25)

- consideration of semi-rigid joint action in frames constructed from light gauge members (32)

- inclusion of joint flexibility effects in welded tubular construction for which joint flexibility is due to local deformations of the members themselves (33)

- full 3-D frame analysis with simplifying assumptions on the diaphragm effect of the floors (34)

- measurement of out-of-plane connection restraint characteristics (35)

- analysis of the full 3-D response of columns in subassemblages containing semi-rigid joints (36,37)

- allowance when determining connection restraint characteristics for composite action with floor slabs (38, 39)

Conclusions

Progress in the area of research into connection restraint characteristics and their influence on the stability of members and frames has been reviewed. For two-dimensional bare steel construction it is concluded that all aspects of the subject:

assessment of connection restraint, modelling and representation, development of methods of stability analysis that allow for connection effects, production of illustrative results and the preparation of design approaches, have been addressed – often in considerable detail. The success of this work in encouraging the application of similar methodologies to a wide range of other stability problems in which connection restraint has a potentially significant role has been noted.

Acknowlegements

During the past decade the author has been fortunate to have worked with a large number of dedicated and capable colleagues on various projects relating to connection restraint. Without their efforts much of the progress referred to in this paper would not have been possible.

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Type of framing	Method of global analysis	Types of connections
Simple	Pin joints	Nominally pinned
Continuous	Elastic	Rigid Nominally pinned
	Rigid-Plastic	Full-strength *Nominally pinned*
	Elastic-Plastic	Full-strength - Rigid Nominally pinned
Semi-continuous	Elastic	Semi-rigid Rigid Nominally pinned
	Rigid-Plastic	Partial-strength Full-strength Nominally pinned
	Elastic-Plastic	Partial-strength-Semi-rigid Partial-strength-Rigid Full-strength-Semi-rigid Full-strength-Rigid Nominally pinned

Table 1 EC3 Linkage of Connection Class and Frame Analysis

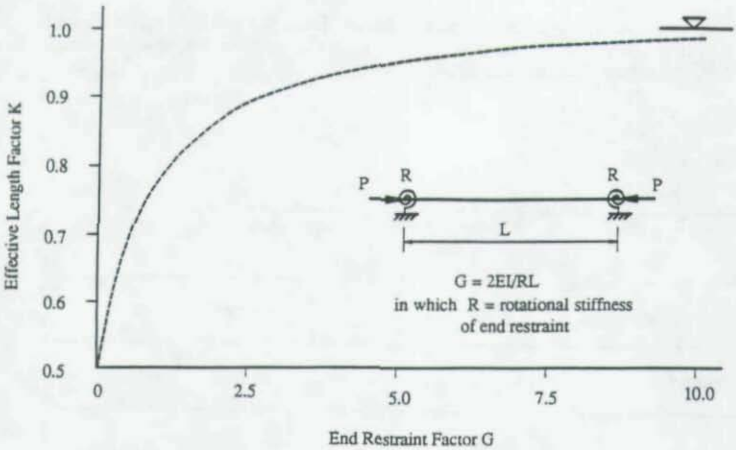


Figure 1 Elastic Buckling of End Restrained Column – Expressed in Terms of Effective Length

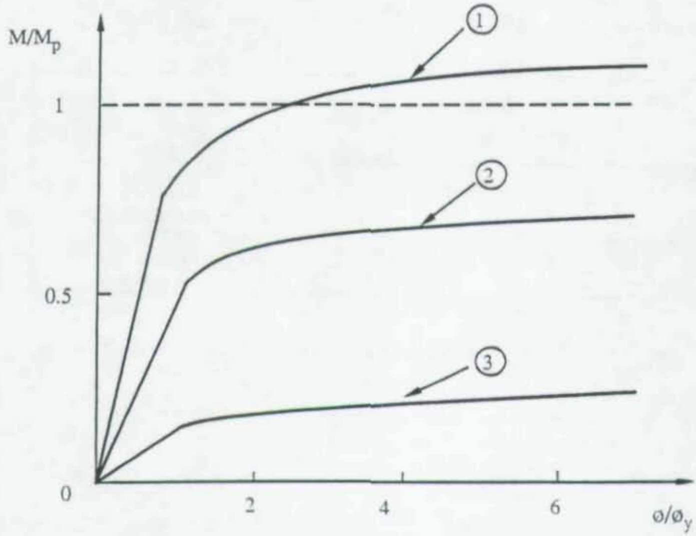


Figure 2 Typical M–ϕ curves

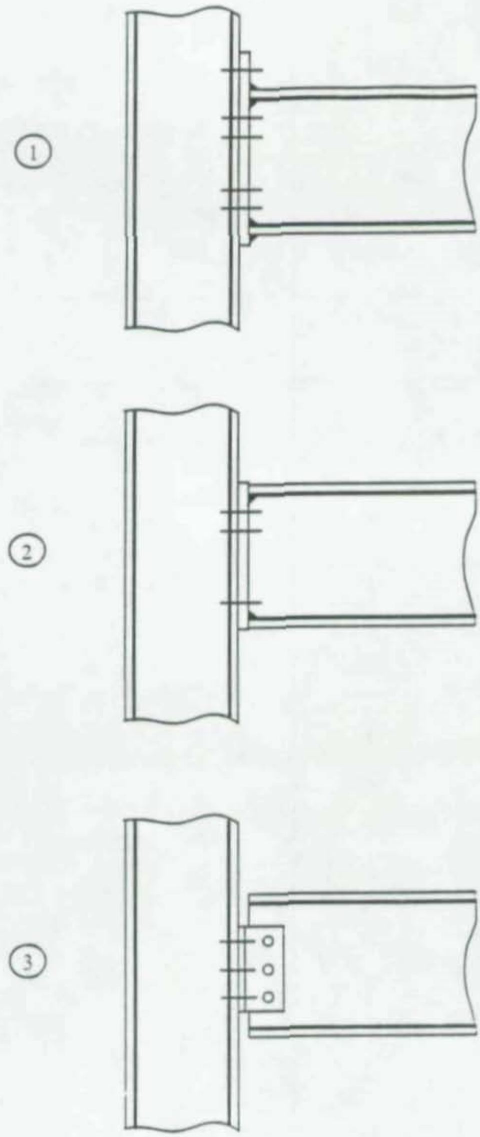


Figure 3 Example Types of Connection

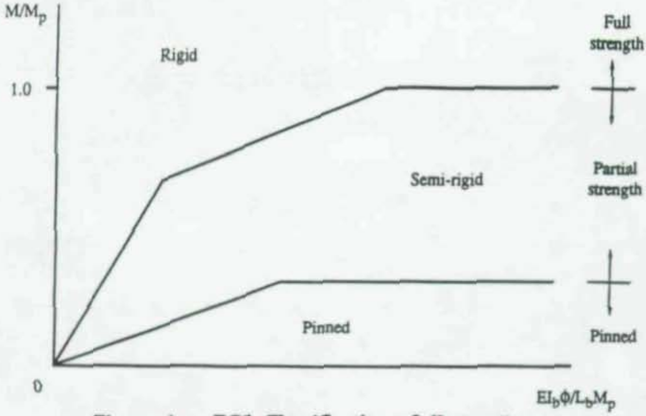


Figure 4 EC3 Classification of Connections (Non-sway frames)

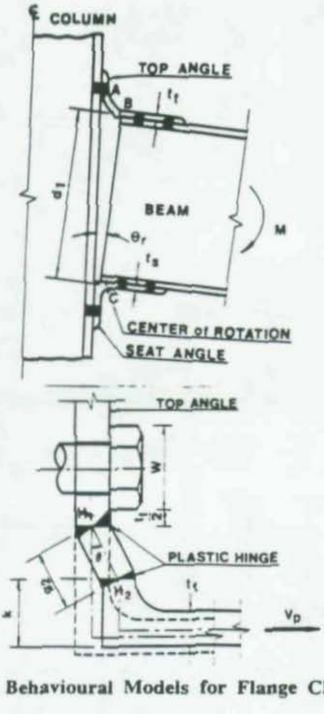


Figure 5 Behavioural Models for Flange Cleat Connection

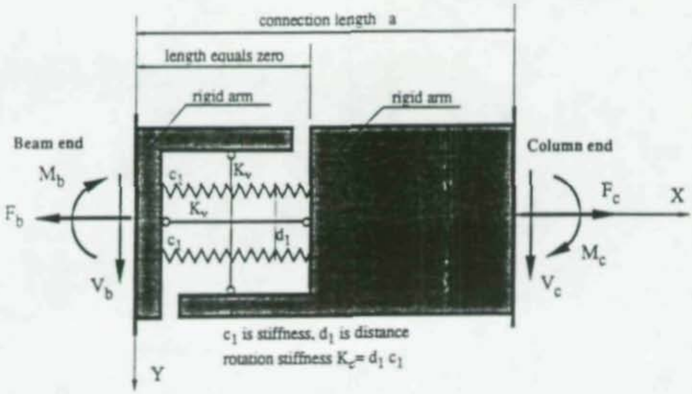


Figure 6 Conceptual Model of Connection

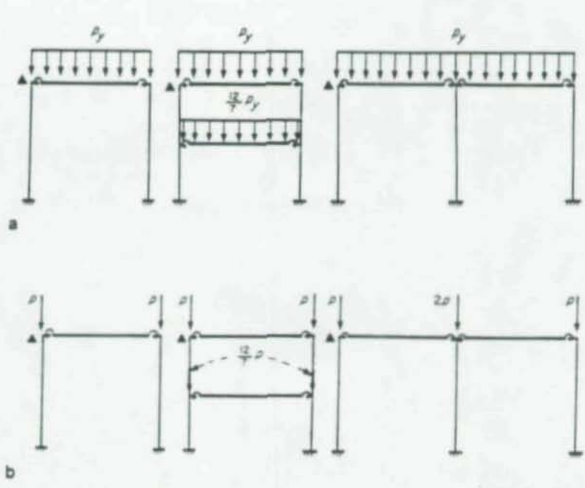


Figure 7 Portal Frames Studies by Goto et al

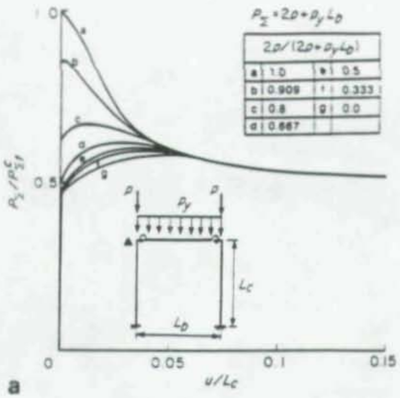
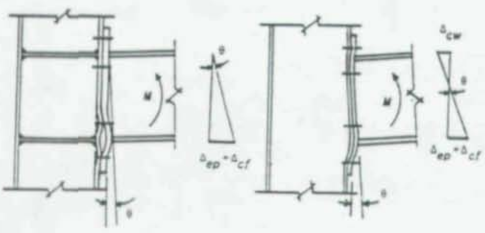
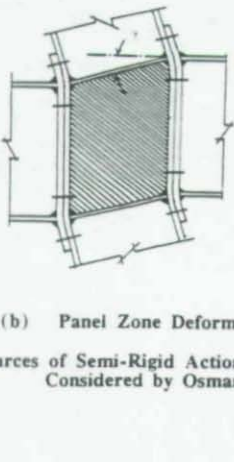


Figure 8   Influence of Loading Conditions on Bifurcation Behaviour



(a)   Connection Flexibility



(b)   Panel Zone Deformation

Figure 9   Sources of Semi-Rigid Action Within Connection Considered by Osman et al

STABILITY OF ANGLE STRUTS

by

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INTRODUCTION

Angles appear to be one of the simplest of structural elements. Connections, bolted or welded, are easy to make. In spite of this, angle compression members are among the most complex structural members to analyze and to design. This is due to the end eccentricities of the end connections and because the principal axes of a single angle do not coincide with the axis of the frame or truss of which the angle is a part.

The theme for the SSRC 50th Anniversary Technical Session is "SSRC - Link Between Research and Practice". Thus the author will: (1) review some of the research conducted at the University of Windsor and some other research to show how it relates to practice; and (2) some research needs will be discussed.

DOUBLE ANGLES

General

Double angles possess at least one axis of symmetry. This greatly simplifies the analysis and design of double-angle compression members.

There are three possible arrangements of double angles. The first is the back-to-back arrangement, as illustrated in Fig. 1(a). This still seems to be the most common arrangement for double angles in the United States. This is probably due to the fact that load tables for these angles are included in the AISC "Manual of Steel Construction, Load and Resistance Factor Design" (1986b).

In 1980 when the author became interested in the interconnection of double-angle compression members back-to-back double angles were widely used in Canada. It now seems that the starred angle arrangement (Fig.1(b)) is the one that is preferred. The CISC "Handbook of Steel Construction" (1991) includes load tables for starred angle compression members. These starred angles also have several significant advantages over the back-to-back arrangement. The starred angle leaves all surfaces exposed for maintenance. This is particularly important when you have a corrosive atmosphere, such as in the chemical industry, or where building hygiene is important, such as in the food or pharmaceutical industry. The minimum moment of inertia of the starred angle cross section made with equal leg angles is 60% greater than when the same angles are used in the back-to-back arrangement. The starred angle arrangement is one that should be considered when double angles are used.

The boxed angle arrangement (Fig. 1(c)), although aesthetically pleasing, is not widely used. The inner surfaces are not accessible for maintenance and the connections must be welded.

Interconnection of Starred Angles

The first research project with regard to angles that the author supervised involved the interconnection of starred angles (Temple et al. 1986). The steel standards in effect in 1980 contained vastly different requirements for the interconnection of starred angles. The Canadian Standard, CAN/CSA-S16.1-M84 "Steel structures for buildings (Limit States Design)"(CSA 1984) and the American Specification, AISC "Specification for the design, fabrication and erection of structural steel for buildings" (1978) required the use of only one interconnector at mid-height of the built-up member. This was based on the requirement that the slenderness ratio of the individual main member between points of interconnection should not exceed the slenderness ratio of the built up member. The German Standard, DIN 4114 "German Buckling Specification" (Deutscher Normenausschuss 1952) required four points of interconnection. The British Standard, BS 449 "Specification for the use of structural steel in buildings" (1970) required the use of five interconnectors and each connector had to be arranged so as to form a cruciform. This resulted from a requirement that the slenderness ratio between points of interconnection could not exceed 40 nor 60% of the slenderness ratio of the built-up member.

The obvious question was "which standard is correct"? The requirements varied from one piece of metal at mid-height to satisfy the North American requirements to ten pieces of metal to satisfy the British Standard. The North American requirement seemed logical and British standard very conservative. The answer was that none were correct and as shown in Fig. 2 the number of interconnectors should be two. With zero and one interconnector the starred angle buckled about the major principal axis but obviously not as an integral unit. With two or more interconnectors the starred angle buckled about the minor principal axis of the cross section. This requirement is now included in the Canadian Standard S16.1 (1989a).

In order to make the individual angles of the starred angle compression member act as an integral unit, the individual angles must be connected together intermittently. The interconnectors carry the shear and moment that may develop as a result of the deformation of the built-up member. In addition, the interconnectors reduce the effective length of the individual angles between points of connection, make the angles deflect the same amount, and also provide some rotational restraint to the individual main members.

When small plates are used as interconnectors in starred angles, three different arrangements of these interconnectors are possible. These may be referred to as aligned, alternating or cruciform and are illustrated in Fig. 3. In the aligned arrangement the interconnectors lie in the same plane. In the alternating arrangement, the connectors alternate from one plane to another at right angles as the interconnectors are spaced along the member. In the last arrangement, the interconnectors are placed in pairs, in contact, at each point of interconnection to form a cruciform.

A literature survey failed to reveal any research on the effect of the arrangement of interconnectors on the load carrying capacity of starred angle compression members. Several steel standards and specifications were examined to determine whether or not they contained any requirements with regard to the arrangement of the interconnectors. The North American standard and specification do not provide any guidance as to the preferred arrangement for the interconnectors. The British Standard, BS 449-1970 (British Standards Institution 1970) specified that interconnector must be in pairs at points of interconnection so that they form a cruciform. This standard has now been replaced by BS 5950-1985 (British Standards Institution 1985) which now requires that the alternating arrangement be used.

An experimental program was undertaken which involved the testing of nine starred angle compression members. Three specimens were fabricated with each of the three arrangements of interconnectors.

All the specimens were fabricated with 64 x 64 x 6.4 mm (2½ x 2½ x ¼ in.) angles. The length of the specimen, knife edge to knife edge, was 3050 mm. Because of the previous research (Temple et al. 1986), two points of interconnection, one at each of the third points were used. The interconnectors were 51 x 51 x 9.5 mm plates welded to the angles.

The experimental failure loads are shown in Table 1. Except for Specimen No. A3, all specimens failed by buckling which predominantly was one of buckling about the weak axis of cross section. From the failure loads obtained from the experimental research it can be concluded that the arrangement of the interconnectors does not have a significant effect on the load carrying capacity of the starred angle compression member. It seems that as long as the individual angles are firmly connected at the point of interconnection, the type and arrangement of the interconnectors does not have a significant affect on the load carrying capacity of the member. A finite element analysis of the same specimens confirmed this conclusion. Details may be found in a paper by Temple et al. (1994).

Design of Double Angles

Most back-to-back double angles will fail by buckling about the x axis. The separation of the angles about the y axis results in a large moment of inertia which is mainly due to the product of an area times a distance squared.

The load tables in the CISC Handbook assume that a sufficient number of interconnectors are used to make the built-up cross section act as an integral unit. The AISC Handbook indicates that when buckling about the x axis is considered, the number of interconnectors should be determined from making the slenderness ratio of the individual main member between points of connection equal to or less than the slenderness ratio of the built-up member. Since buckling about the x axis involves movement in the plane of symmetry, this is simply a case of flexural buckling.

The general equation for torsional-flexural buckling indicates that when you have one axis of symmetry the section is subject to flexural buckling about the x axis or torsional-flexural buckling about the y axis. It has been shown by Nuttall and Adams (1970) that for back-to-back double angles the effect of torsional-flexural buckling is to reduce the load carrying of the double angles by five percent or less from the load carrying capacity of the same member predicted by considering only flexural buckling about the y axis. Thus it is common practice to neglect torsional-flexural buckling when the design of back-to-back double angles is concerned.

When flexural buckling about the y axis is considered, the member behaves much like a battened column. Interconnectors must be used for the reasons previously outlined. The Canadian practice in the past was to simply make the slenderness ratio of the main member between points of connection equal to or less than the slenderness ratio of the built-up member. The latest edition of the Canadian Standard S16.1 now contains an equivalent length formula which is simply the square root of the square of the slenderness ratio of the built-up member plus the square of the slenderness ratio of one of the main members between points of connection. The CISC Handbook provides no guidance as to how to make back-to-back angles act as integral unit.

Consider two 90 x 75 x 13 mm angles, long legs back-to-back with a 10 mm gusset plate between the angles. Take an effective length of 4000 mm and consider y axis buckling. Compressive resistances were calculated in accordance with the Canadian Standard S16.1. If the built-up section acts as an integral unit a compressive resistance of 404.5 kN is calculated. If the slenderness ratio of the main member between points of interconnection is made equal to or less than the slenderness ratio of the built-up member, two interconnectors are required. If it is assumed that this is sufficient to make the angles act as an integral unit a failure load of 404.5 kN would, of course, be obtained. If the equivalent slenderness ratio formula is used a compressive resistance of 294.9 kN is calculated. If three interconnectors and the equivalent slenderness ratio is used a compressive resistance of 333.9 kN is determined. If three interconnectors and the AISC modified slenderness ratio is used a compressive resistance of 400.3 kN is calculated.

The author is of the opinion that the connection of built-up members is not well understood. Experimental research results for built-up members is not abundant. As a result a research project at the University of Windsor on the connection requirements for built-up members has been initiated. Current research involves built-up members made from double channels. Early results indicate that for y axis buckling the use of the equivalent length formula previously mentioned accurately predicts the load carrying capacity of these built-up members. Thus it would seem that this would also be applicable to back-to-back double angles. Further research on y axis buckling is required. It is difficult, however, to get back-to-back angles to buckle about the y axis unless bracing is provided to prevent buckling about the x axis. This invariably results in the question concerning the effect of the bracing on the y axis load carrying capacity.

SINGLE ANGLES

General

Most single angle compression members are attached to another member by one leg only. The eccentricity of the load results in a very complex analysis and design problem. This results in biaxial bending about both of the principal axes. The problem is further complicated by the fact that the principal axes of the angle compression member do not coincide with the plane of the structure, often a truss.

Design Philosophies

In North America there are two approaches to the design of single angle compression members attached by one leg.

The CISC Handbook provide no guidance as to a preferred design philosophy for these members. Past practice is that the eccentricity of the load about the principal axes is neglected. The angle is designed as if it is a concentrically loaded member which buckles about the z axis, the minor principal axis of the cross section (Fig. 4(a)). The effective length factor is usually taken as 1.0 but some engineers use an effective length factor as low as 0.9.

The approach in the United States is to account for the eccentricity about the principal axis (Fig.4(b)). The single angle is then treated as a beam-column. The AISC Handbook provides as numerical example illustrating this approach.

The Canadian approach is obviously incorrect since an angle is attached by one leg only cannot bend about the z axis. If the angle is welded to a structural tube, for example, bending just above the weld must take place about an axis parallel to that leg as shown in Fig.5(a). Even at mid-height of the specimen the axis of bending does not coincide with the minor principal axis. The deflections at mid-height of the specimen in the x direction, perpendicular to the attached leg (Fig.5(b)), is usually about 2.5 to 5 times the deflection in the y direction. The axis of bending can be determined from these deflections. If the axis of bending for an angle welded to a tee or structural tube is determined this axis, at a load approaching the failure load of angle, is something like 15 to 25° from the y axis as shown in Fig.5(b). It must be emphasized that the orientation of the axis of bending is not precise as this orientation is dependent upon the deflection which is a function of the load level at which this axis of bending is determined. It is, however, indicative of what is happening. This axis of bending varies along the length of the member. If the axis of bending for the angle is assumed to be the axis of bending at mid-height, the moment of inertia about that axis can be determined. Using this moment of inertia in a design procedure will result in a higher failure load.

Test Results

The two North American design philosophies for single angles have been discussed. The Canadian approach of ignoring the eccentricity of the load does not correspond to the behaviour of single angles loaded by one leg as observed in the laboratory. The American approach of considering the load eccentricity seems to reflect, more accurately, the expected behaviour of angles. It has been shown, however, that the Canadian approach often gives a better estimate of the load carrying capacity of a single angle loaded by one leg than does the American approach. The American approach is very conservative. This has been pointed out by Adluri and Madugula (1992), Elgaaly et al. (1992) and others. The author has noted that this approach sometimes gives values as low as 20% of the observed failure loads. This is particularly true when there is considerable restraint at the end of the angle. It is believed that many chords of trusses provide considerable restraint to the ends of angles.

The problem with the American approach is that the angle is assumed to be pinned about the load point. If the end restraining moments, both in-plane and out-of-plane, where converted to an axial load times an eccentricity and if that eccentricity were subtracted from the eccentricities shown in Fig. 4(b) reduced load eccentricities would result. The "apparent load point" would be closer to the centroid of the angle. This would explain why the Canadian approach often gives a better estimate of the load carrying capacity of an angle than the American approach.

This phenomenon of an "apparent load point" was pointed out by Batho as early as 1912. Batho tested, in tension, double angles riveted to end plates. The strain was measured mechanically at ten points across the angle cross-section. From these strain readings an "apparent load point" was determined. Fig. 6 is from Batho's paper. It could be assumed that the loads were applied to the angles at points K_1 and K_2. Due to end restraint the eccentricities of the load were reduced and the "apparent load points" as determined by Batho are L_1 and L_2. The same sort of thing must be occurring in single angles, attached by one leg, loaded in compression.

Galambos et al. (1969) tested a single 2 x 2 x ¼ in. angle attached to tee section. A knife edge was used parallel to the web of the tee. The maximum load determined in a test was 16.9 kips. Using a numerical procedure a maximum load carrying capacity was calculated which varied from 15.4 to 18.2 kips depend on whether residual stresses and an initial out-of-straightness was included in the calculations. It is interesting to note that a finite element analysis resultes in a maximum load of 17.4 kips. The beam-column approach results in a load of 8.0 kips.

Effective Length

In a previous section it was mentioned that the axis of bending for an angle attached by one leg just above the weld is one that is parallel to the attached leg. As various cross sections are examined between this section and the mid-height of the angle it is seen that the axis of bending rotates towards the z axis but never coincides with that axis. The axis of bending is always somewhere between an axis parallel to the attached leg and the z axis. Thus the axis of bending rotates as various sections are examined along the height of the angle.

The effective length of an angle is considered to be the unsupported length divided by a radius of gyration. The simplest of these quantities to discuss is the unsupported length of the angle and yet there seems to be some disagreement as to what length to use. In design it seems to be common practice to use the distance from workpoint to workpoint. This in a truss would be the distance between the intersection points of the centroidal axes of the chords of a truss and the centroidal axis of the angle. In some cases the unsupported length has been taken as the length of the angle while in other cases the length between points of rotation, from knife edge to knife edge, have been used. Thus the length used as the effective length can make the interpretation and comparison of results much more difficult.

Of more concern, and something that is much more difficult to determine, is the radius of gyration that should be used when determining the slenderness ratio. It has been common practice to use the radius of gyration about the minor principal axis, the z axis, when determining the slenderness ratio. It has been shown, however, that this is not correct as no section in a single angle attached by one leg bends about this axis.

In the authors opinion, however, the unsupported length, however that is defined, divided by the radius of gyration about the minor principal axis, the z axis, will continue to be used as the slenderness ratio of a single angle until research indicates that there is a better way of defining the slenderness ratio.

Balanced and Unbalanced Welds

Welds are often used to attach angles to other members. These welds are referred to as balanced or unbalanced welds. A weld is said to be balanced about the centroid of the angle if the weld is distributed in such a manner that the centre of resistance of the weld coincides with the projection of the centroid on the connected leg. This is illustrated in Fig. 7(a). A weld is balanced about the centre of the leg if the centre of resistance of the weld group coincides with the centre of the connected leg as shown in Fig. 7(b). Since this results in equal length welds on each side of the connected leg, this weld is referred to as an equal weld. The third possibility is an unbalanced weld such as the one shown in Fig. 7(c).

A balanced weld requires approximately twice as much weld along the heel of angle as along the toe. For this reason, the weld connecting the angles, the web members of a truss for example, to the chords of a truss is often a weld that is unbalanced. Engineers are often forced to use unbalanced welds since there is insufficient space to place a balanced weld. This is particularly true when the chord is a structural tube, as shown in Fig. 8, and the web members are at an angle of 45° to the chord. A structural tube frequently used as a chord in a truss has a wall height of 76.2 mm. When the corner radii are subtracted a flat wall height of 50 mm remains. If the leg of an angle has a 55 mm width a balanced weld cannot be achieved. If the angle has a 65 mm leg width, the weld can be deposited at the end and along one side of the angle only. Thus the question arose as to the effect of balanced and unbalanced welds on the load carrying capacity of single angles.

Gibson and Wake (1942) published a paper entitled "An Investigation of Welded Connections for Angle Tension Members". In this investigation fifty-four ultimate tension tests were conducted on angles welded to connector plates. Fifteen different arrangements of the weld patterns, including most of the common types of connections, were used in the tests. The research program included the testing of single as well as double angles. The main conclusion of the research is that the strength of the angles with balanced and unbalanced welds was essentially the same. In other words, the research concluded that the accepted practice at the time of balancing the centre of resistance of the weld group with the projection of the centroid on the attached leg is not required to develop the tensile resistance of the angle member.

The Canadian Standard, S16.1, states that "Except in members subject to repeated loads..., disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single angle, double angle, or similar types of axially loaded members is not required". The AISC Specification (AISC 1986a) and the British Standard have the same requirement. Although the Gibson and Wake research applies to angles subjected to tensile loads, the results of this research are now being applied to compressive loads as well.

An experimental program involved the testing of 18 specimens. Each specimen consisted of two angles welded to a piece of structural tube at each end of the specimen. The two angles were not connected to each other except at the ends. The structural tube was not free to rotate.

As mentioned previously, eighteen specimens were tested of which nine had angles which were slender and in the other nine the angles were of intermediate length. In each set of nine specimens, three were tested with each of the three weld patterns shown in Fig. 7.

The compressive resistance of each angle was calculated in accordance with the Canadian Standard S16.1. The resistance factor was taken as 1.0. The welds were designed in accordance with the Canadian Standard W59 (1989b).

The experimental failure loads for all the specimens are calculated in Table 2 for slender and intermediate length specimens. The effect of an unbalanced weld seems to be detrimental to the load carrying capacity of angles of intermediate length. This is undoubtedly due to the eccentricity of the centre of resistance of the weld group with respect to the projection of the centroidal axis on the attached leg. Changing the weld pattern from a balanced to an unbalanced weld can reduce the compressive load carrying capacity by as much as 10%. The effect of unbalanced welds seems to be beneficial to the load carrying capacity of slender angles. This unexpected result is probably due to the fact that an unbalanced weld provides more restraint to the toe of the attached leg of the angle than do the other two weld patterns used in this research. This, in effect, cuts down on the effective length of the attached leg by providing restraint at its weakest edge, that is, the toe of leg. Most of the bending, but not all of it, takes place in a plane perpendicular to the attached leg. Changing the weld pattern from a balanced to an unbalanced weld can increase the compressive load carrying capacity by as much as 33%. It must be remembered that the angles in these tests were attached to a structural tube that was essentially fixed. The results of this research are reported in a paper by Temple and Sakla (1994).

RESEARCH NEEDS

The author could point out some specific research needs but it seems to be more productive to lump the research needs under three categories.

1. In the literature the results of many isolated angle tests are reported. There is a need for more tests where angles are part of an assemblage, for example, a truss. Some truss tests have been reported by Galambos et al. (1969) and Elgaaly (1992). In Elgaaly's case the test angles were bolted in to a truss. Thus after each test the angles could be removed and a new one inserted. The problem with full-scale tests are that they are very expensive in terms of money and time. Tests where the angles are welded to the chords of trusses are needed. The problem is that these angles cannot readily be replaced.

2. There are a lot of excellent research papers on angles were the end result is in a form which is not readily useable by practicing engineers. Further research is

required to take these ideas and develop them into a format which can readily be used by designers.

3. The complex problem of a single angle loaded by one leg requires further research. Such a research project is underway at the University of Windsor. It is essential that end restraint be estimated, even if conservatively, so that a rational approach to the design of single angles attached by one leg will yield a compressive resistance more in keeping with experimental results. If a rational approach is not possible an empirical formula should be developed. This would permit the development of load tables for single angles attached by one leg and for cases not covered by the tables a formula could be used.

SUMMARY

The author has attempted to summarize some of the angle research as related to double and single angles and to point out, at least in general terms, some of the research needs.

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TABLE 1. STARRED ANGLES WITH VARIOUS INTERCONNECTOR ARRANGEMENTS

Type of Interconnection	Specimen Number	Experimental Failure Loads (kN)	Theoretical Failure Loads (kN)
Aligned	A1 A2 A3	176 195 177	181 184 187
Alternating	B1 B2 B3	165 189 187	201 185 187
Cruciform	C1 C2 C3	177 173 181	184 189 188

TABLE 2. BALANCED AND UNBALANCED WELDS

(a) Experimental results for slender angles

Compressive resistance = 30.2kN

Specimen No	Type of Weld	Failure Load (kN)	Average Failure load (kN)	Average Effective length factor	Ratio of average failure load to failure load with balanced welds
SB1 SB2 SB3	Balanced	32.2 34.5 35.0	33.9	0.94	1.00
SE1 SE2 SE3	Equal	38.9 42.2 46.7	42.6	0.83	1.25
SU1 SU2 SU3	Unbalanced	45.0 46.1 44.5	45.2	0.80	1.33

(b) Experimental results for intermediate length angles

Compressive resistance = 87.8 kN

Specimen No	Type of Weld	Failure Load (kN)	Average Failure load (kN)	Average Effective length factor	Ratio of average failure load to failure load with balanced welds
ILB1 ILB2 ILB3	Balanced	107.8 119.5 106.4	111.2	0.84	1.00
ILE1 ILE2 ILE3	Equal	98.5 106.7 110.0	105.0	0.87	0.95
ILU1 ILU2 ILU3	Unbalanced	98.9 101.4 99.5	99.9	0.91	0.90

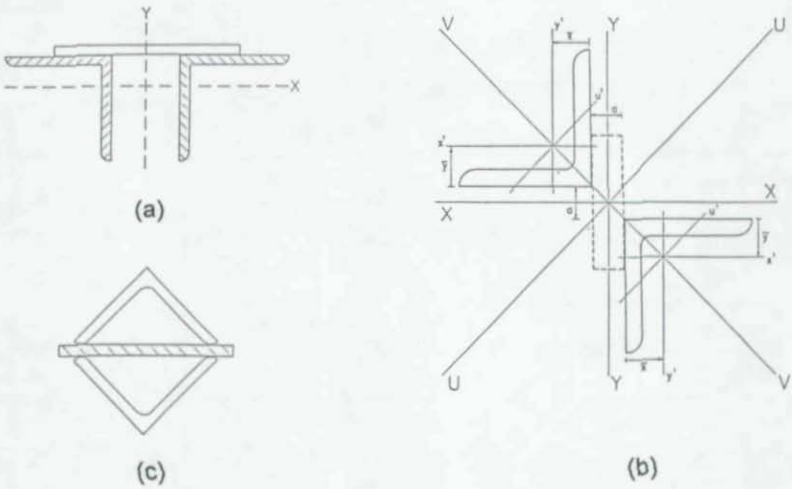


Fig. 1 Double angle configurations: (a) back-to back; (b) starred; (c) boxed

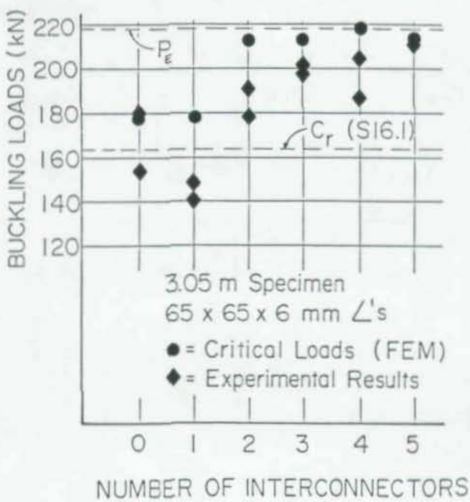
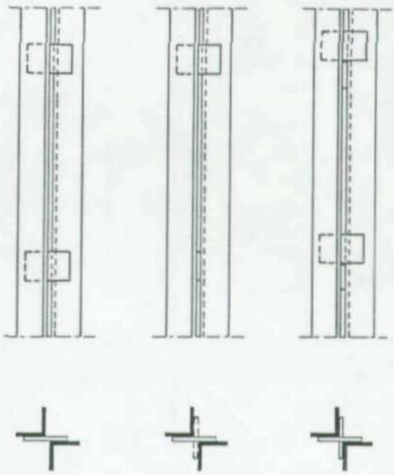
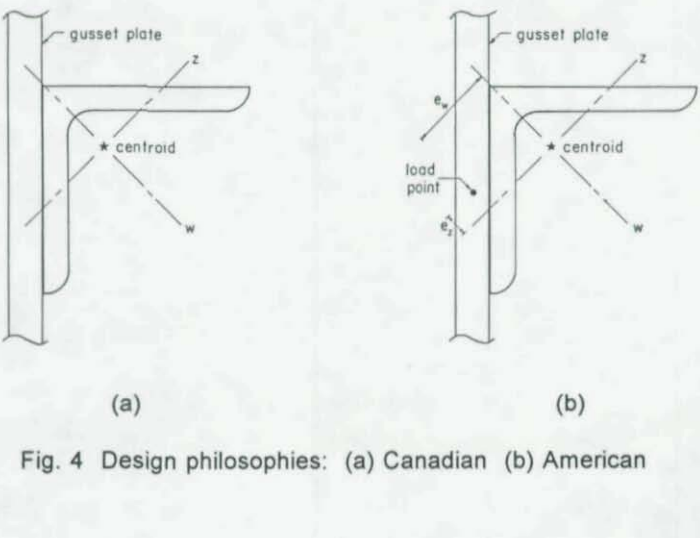


Fig. 2 Starred angle test results



(a) (b) (c)

Fig. 3 Starred angle interconnector arrangement:
(a) aligned (b) alternating (c) cruciform



(a) (b)

Fig. 4 Design philosophies: (a) Canadian (b) American

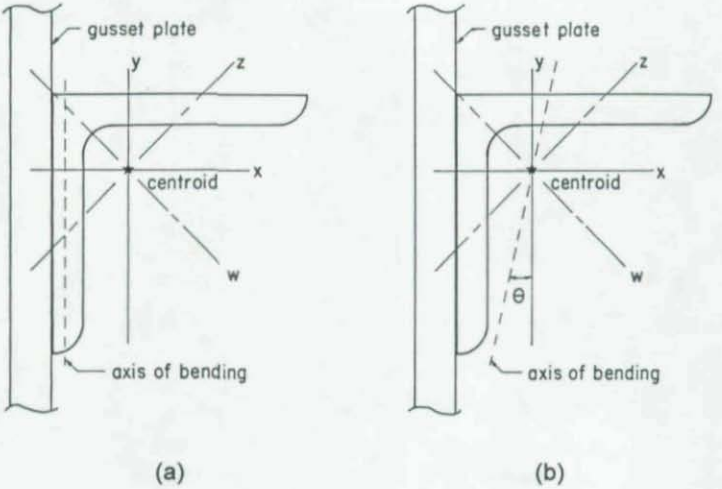


Fig. 5 Axis of bending: (a) just above weld (b) at mid-height

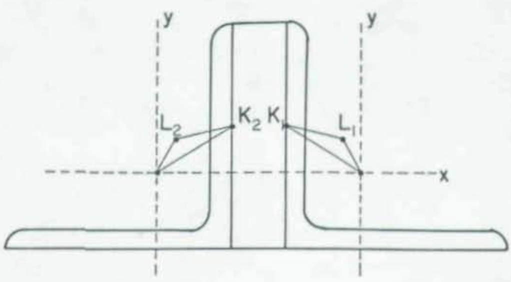


Fig. 6 Apparent load point (after Batho)

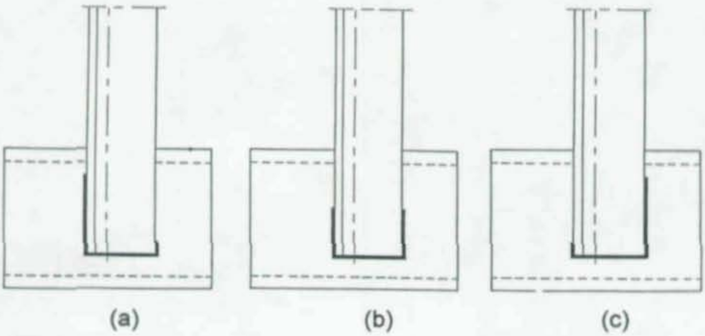


Fig. 7 Weld patterns: (a) balanced; (b) equal; (c) unbalanced

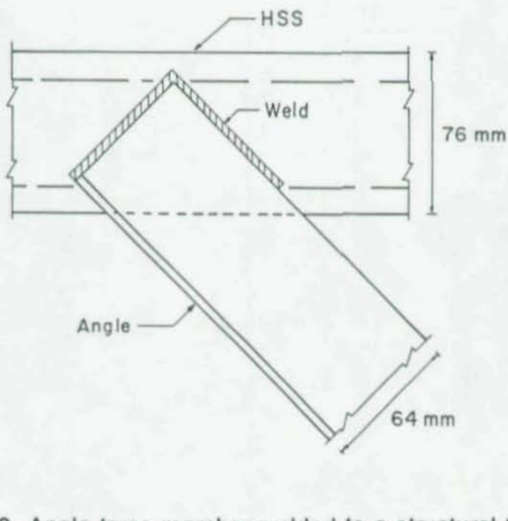


Fig. 8 Angle truss member welded to a structural tube

COMPREHENSIVE PERFORMANCE ASSESSMENT OF BUILDING STRUCTURAL SYSTEMS: RESEARCH TO PRACTICE

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1. INTRODUCTION

In a recent report submitted to the National Science Foundation, a Blue Ribbon Panel on High-Performance Computing observed that within four to five years, desktop workstations delivering up to 400 megaflops[1] costing no more than $15,000 to $20,000 should be widely available (NSF, 1993). Furthermore, the report states that personal computers, defined as machines costing $10,000 in constant 1993 dollars, will approach 200 megaflops, and that workstations in the $50,000 price range will approach one gigaflops[2] in this time frame (NSF, 1993). Some prominent computer vendors have forecast even higher estimates than these. Friedman (1993) predicts that by the turn of the century, desktop PCs will be "humming along" at between 256 to 1000 MIPS[3], the average PC will hold about 4 gigabytes of data, and networked data access will be counted in terabytes. Network data transfer bandwidths greater than 100 megabits per second can be achieved economically at the present time, and gigabit per second network backbone bandwidths appear achievable with emerging Asynchronous Transfer Mode technology (Curry and Freeland, 1994). Local access to disk storage at 20 megabytes per second is now available on low-cost workstation servers with wide SCSI-2 technology (SUN, 1994). These and many other predictions and achieved advances punctuate an expected performance per dollar increase of at least 10 fold relative to 1993 technology (e.g., single-processor SUN SPARC10 workstations and Intel 486/DX2 based personal computers) throughout the PC and workstation industries by the year 2000. Furthermore, the NSF Blue Ribbon Panel has predicted that a teraflop capability is achievable at the highest level of national supercomputing within this time frame.

What impact should these levels of computing power have on structural engineering research and practice, and more specifically, on the stability design of structures? The author would suggest that such capabilities, combined with the proper development of computational models and computer software, will permit assessment of structural system behavior and performance (e.g., ductility, damage, survivability, etc.) to an extent recently unimaginable. This paper is a call for a concerted effort by the Structural Stability Research Council to help facilitate the achievement of this technology, and to promote the proper application of such capabilities to improvement of stability design practices.

Much prior research has been devoted to nonlinear analysis of building structures subjected to service and extreme loads. Computational models have been developed which account for various aspects of the detailed behavior at structural component levels, including uncertainty in behavioral parameters. However, up to the present time, these techniques have found only

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[1] One megaflops is 1 million (10^6) floating point operations per second.

[2] One gigaflops is 10^9 floating point operations per second.

[3] One MIPS is one million instructions per second.

limited application beyond that attempted by specialized research groups involved with their development, and to the author's knowledge, there has been little integration of state-of-the-art capabilities developed by separate researchers. Little attention has been devoted to data management and computer graphics capabilities necessary to ease the creation, interpretation, and synthesis of the massive amounts of data required for and generated by sophisticated models of building structural performance. As a result, although computational models investigated in previous research have demonstrated promise for characterizing the behavior of certain types of structural components, adequate capabilities for determining the actual overstrength, ductility, energy absorption capacity, and failure modes of complete building structural systems still do not exist.

2. PROBLEM STATEMENT

During the last 20 years, design codes and specifications have come a long way in addressing the strength, stability, ductility, and serviceability of structural components. However, consideration of structural system behavior in design is largely still implicit and semi-empirical. This is evidenced in the use of *effective length factors* to account for system effects in rules for member proportioning, through devices such as the *Merchant-Rankine formula* for assessment of the interaction of plasticity and stability effects at the system level, and in the definition of *response modification* and *displacement amplification factors* which account for system inelastic actions in seismic design. All of these and other approximate tools serve an important engineering purpose. However, improvements in capabilities for direct assessment of the full-range performance of structural systems under service and extreme load conditions are needed to solve many of the questions still facing modern structural engineering practice.

The above deficiencies are not due to a lack of quantity of good research. Much research has been directed at the analysis of nonlinear behavior, and recent advances have been made in addressing aspects such as member inelasticity, second-order effects due to changes in geometry, connection and joint deformability and nonlinearity, and variability of load and resistance attributes. A number of studies have demonstrated excellent correlation between inelastic transient dynamic analyses and laboratory tests or field measurements. However, the models still require "tuning" in many cases to obtain improved accuracy of the predictions. Also, there remains a need for integration of and broader application of these sophisticated techniques. For example, there is a particular need for this type of integration in the area of partially-restrained steel construction, to take advantage of the large amount of experimental and modeling research which has already been performed in this area, and to achieve comprehensive capabilities for performance assessment.

For modern computational tools to achieve their maximum potential impact on code development and professional practice, improved capabilities are needed for synthesis and graphical presentation of the massive amounts of data that are generated from nonlinear modeling studies. Engineers need to be able to rapidly assess the overall performance of a building structure subjected to multiple loading actions, both through indicators of the net overall effect of a full set of relevant loading combinations, as well as through displays of detailed responses for each individual load case. Detailed inspection of the ductility and damage within critically loaded components should be facilitated at a local level, but also, quantitative indices of the behavior need to be provided at a global level. For large systems, present techniques found in research and commercial software for structural analysis and design of frames tend to be cumbersome for gaining an overall view of the structural performance, or for extracting detailed local component information from an overall representation of the structure. The engineer should be able to extract on demand graphical displays of various indices of local and overall performance, including indicators of the effects of variability in load and resistance quantities.

Lastly, future tools such as those described above need to be applied to facilitate the innovative and economical use of all types of building materials for design, particularly where ductility and energy absorption capacity are of key importance. There is great potential for benefits to be obtained from research on seismic structural performance. For example, the responses of building structures to potential earthquakes within the eastern and mid-western United States should be better quantified and compared to the responses to seismic events within the western regions of the country. This kind of information can only be obtained comprehensively based on refined computational studies, backed by field measurements and specifically targeted experimental tests. Both transient dynamic as well as static nonlinear analysis can provide valuable information about the seismic characteristics of structural systems.

3. PRESENT TRENDS IN PRACTICE

Of course, the full-range performance assessment of building structures requires consideration of overall component inelastic actions and second-order effects due to changes in geometry, among other important aspects of the behavior. That is, a second-order inelastic analysis is required. Although the primary short-term advantage of such techniques is in achieving and facilitating a better understanding of the response characteristics of structural systems, there are definite trends toward the direct use of second-order inelastic analysis methods in design practice.

Among the key factors driving these developments are the advent of limit states design codes, and the advancements in computing and graphics performance of workstations and personal computers. Since limit states design procedures are based on analysis at factored load levels and limits of structural resistance, it is no longer possible to disguise the fact that systems behave nonlinearly before their design conditions are reached. The increases in personal and workstation computing power have made direct analysis of significant nonlinear actions achievable.

Recent research studies have considered the implications of direct second-order inelastic analysis on design within the context of the AISC-LRFD Specification (Ziemian et al., 1992 a & b; Liew et al., 1994, 1993a-d; White, 1993; Chajes, et al. 1992; Clarke and Bridge, 1992). Along this line of development, Task Group 29 of the SSRC recently organized a workshop on the *advanced analysis* of frames (SSRC, 1993). The term advanced is intended to indicate an analysis which accounts for all the significant stability and plasticity effects for the problem at hand, to such an extent such that a member's adequacy to sustain load is completely verified by the analysis. Thus, for example, if local or lateral-torsional buckling of certain frame members significantly affects the strength and/or performance of a frame for a particular design and loading, a proper advanced analysis must capture these effects. Advanced analysis models which can represent faithfully all the possible behavioral effects that might influence frame stability are still not a practical reality. However, advanced analysis of planar frames of compact cross-section, adequately braced out of plane such that out-of-plane buckling modes do not influence the behavior, is readily available - at least within a university research context (SSRC, 1993). This covers many types of practical situations. Substantial progress is being made in on-going research toward making advanced analysis techniques even more generally applicable (McGuire, 1991; Chen and Toma, 1994).

The Australian Standard AS4100 (SAA, 1990) and the Eurocode 3 (CEN, 1993), within certain restrictions, permit the use of advanced analysis in lieu of Specification member capacity checks. At the present time, the intent of this recognition is primarily for design of special or unusual structures, for assessment of existing structures whose capacities are in doubt, and for investigation of structural failures (Clarke, et al., 1993). However, with increasing computational power of personal computers and workstations, and with improvements in computer-aided engineering procedures, these techniques have the potential for practical use whenever the engineer wishes to rigorously investigate the structural system and component behavior of a design. This can lead to greater economy and more uniform safety in design for static loadings, but the potential benefits of these analysis techniques for seismic design are even greater. Present U.S. seismic design specifications and codes (FEMA, 1988; ICBO, 1988; AISC, 1992; ASCE, 1990; SEAOC, 1990) generally recognize inelastic transient dynamic analysis as a legitimate tool "to estimate the sequence in which components become inelastic and indicate those components requiring strength adjustments so as to remain within the required ductility limits" and "to determine a more accurate distribution of forces and deformations among the various parts of a structure" (ASCE, 1990). Inelastic analysis methods have been utilized within this context for design of a number of major building structures (Amin,1991). Nevertheless, inelastic analysis methodologies still have not reached a state of development and general accuracy such that they may be utilized with complete confidence. This is reflected in the appropriate emphasis in code provisions on design and detailing of components for energy dissipation capacity, and in code restrictions on the reduction in design base shear obtained by use of dynamic inelastic analysis.

4. FOCUSING OF RESEARCH TOWARD COMPREHENSIVE SYSTEM PERFORMANCE ASSESSMENT

The present state of computational modeling techniques for seismic performance assessment is summarized well in the following statement by Galambos (1992):

> **Excellent models exist for predicting the inelastic behavior under cyclic loads of many types of structural elements, sub-assemblies and even complete steel structures, including effects of shear, P-delta forces, axial forces, and many other phenomena, resulting in stable, predictable load-deformation patterns. However, fundamental theories are needed so that fatal local and overall causes of damage or collapse can be calculated.**

This statement touches on what the author feels to be the inherent limitations of most of the present nonlinear frame analysis techniques: the inability to capture a local failure within the structural system during the overall evaluation of the system performance to a loading event, due to the lack of "complete" rigor or comprehensivity of the models. The author agrees that there is a need for more fundamental research toward improvement of models for component response, with a goal toward comprehensive prediction of structural system behavior.

Many research studies have appeared in the recent literature involving the use of inelastic transient dynamic analysis to assess the seismic performance of various types of steel building structures, for example Cheng et al. (1992), Shin (1992), Hwang et al. (1991), May and Naji (1991), Perotti and Scarlassara (1991), Schiff, et al. (1991), Goel and Chopra (1990), Wen et al. (1990), Dicorso, et al. (1989), Goel and Boutros (1989), Popov et al. (1989), Shing et al. (1989), Tang and Goel (1989), Youssef-Agah et al. (1989), and Krawinkler and Mohasseb (1987) among many others. These studies have employed a wide range in type and sophistication of structural behavior models and analysis techniques. Also, a vast amount of structural engineering research in recent years has focused on the development of more rigorous and theoretically sound structural models of nonlinear behavior. A smaller amount of research has focused on real-time graphical simulations based on these tools. Some of the notable recent software development efforts that the author is aware of include DRAIN-2DX (Allahabadi and Powell, 1988), BASYS (Srivastav, 1991), and IDARC (Kunnath et al., 1992).

A forum is needed for discussion, evaluation, and assimilation of all the above types of research efforts, and SSRC is an ideal organization for promotion of such an activity. These activities eventually need to move toward commercial software development for the maximum benefit to practice to be realized. Yet, there are many research issues still to be investigated in the understanding of fundamental behavior, in the development of appropriate computational models, and in the utilization of these sophisticated modeling capabilities. At the present time, it is difficult for researchers to compare detailed aspects of one computational model versus another, and there is a lack of integrated use of complete capabilities for addressing realistically the full-range performance of building structural systems. The commercial viability of these types of software is not expected to be high enough to attract any major commercial efforts in the immediate future. However, commercial software vendors should have an interest in these developments, and need to be active participants in this type of forum. Possibly SSRC could be effective not only in providing this forum, but also in helping to make good research software systems more accessible and more available to the structural engineering profession.

5. RESEARCH NEEDS

The following subsections outline a number of specific research needs, the net satisfaction of which can help move the comprehensive modeling of building system structural performance forward by a significant degree. These suggestions are intended just as a sampling of topics to indicate the scope and stimulate the broader discussion of issues in these areas. The discussions are targeted primarily toward structural steel behavior. However, research addressing and integrating a full range of materials and structural forms utilized in building construction should be encouraged. Such research can truly "lay out all the facts" and facilitate "optimum usage" of all materials considering strength, ductility, resistance to damage, etc.

5.1 COMPONENT MODELS

Computational models should be sought which have a strong fundamental mechanics basis and which demonstrate good correlation with experimental test results, where such results are available. The ultimate goal should be the integration of a wide range of models for realistic analysis of structural system performance. A few of the important attributes to be considered may be grouped into the following categories for purposes of discussion: general attributes, attributes pertaining to beam, beam-column, and bracing members, attributes of connection and joint elements, and walls and infilling.

5.1.1 General Attributes

It is impossible to discuss all the general attributes which can significantly influence structural system behavior within the context of this paper. A few important but often neglected attributes which may be listed are: strain-hardening and cyclic-hardening of the steel, loading rate, variations in yield strength, local buckling and low-cycle fatigue, and inelastic P-Δ effects. Numerous studies have demonstrated the importance of cyclic hardening, for example Ricles and Popov (1994), Goel and Boutros (1989), Hays (1981) and Santhanam (1979). Fundamental cyclic plasticity relationships for structural steel at the stress-strain level have been formulated by

Lee, et al. (1992), Dafalias (1992), and White (1988) among others. However, it is necessary to model the response in terms of generalized stress-resultants and generalized strains for efficient analysis of structural systems. The effects of loading rate have been studied experimentally by Chang et al. (1990) and Takanashi and Udagawa (1989), and have been found to be significant. A number of researchers have addressed loading rate in the formulation of structural finite element models, for example Hilmy (1984), but the author is not aware of any system performance predictions in which the effect of loading rate has been included. A number of recent papers have discussed the importance of variations in yield strength on structural performance. Kuwamura and Sasaki (1990) concluded that randomness in yield strength has a predominant influence on failure modes and system ductility. They point out for instance that weak-beam strong-column structures can be realized only when the randomness in yield strength is reduced by higher quality control. It is well recognized that low-cycle fatigue may often precipitate the failure of elements that undergo significant inelastic local buckling and distortion (Fukumoto and Lee, 1992), and a number of component models have been proposed which attempt to characterize these aspects, e.g. Krawinkler (1987), and Perotti and Scarlassara (1991). P-Δ effects, including inelasticity in members, connections, and/or panel zones can cause significant elongation of the fundamental period, and thus can significantly influence the time-history response of the structural system (Krawinkler and Mohasseb, 1987; Liew and Chen, 1992).

5.1.2 Beams

When composite action is achieved, there is substantial benefit from the slab in providing strength and stiffness to the beams and girders of the structural system, both for positive and negative bending. This influence should be included in the assessment of system response to lateral loads. Previous research (Zaremba, 1988) has examined the formulation of element models for composite beams. However, Zaremba's work does not consider the beneficial composite action that can occur at properly detailed beam-to-column joints. This is addressed below in the discussion of connection and joint effects. Also, plastic-zone studies of steel moment frames with non-composite "all-steel" beams indicate that beam moments greater than the plastic moment M_p can be achieved due to strain hardening and moment gradient effects (SSRC, 1993). Of course, these strengths are limited by attributes such as beam-to-column connection strength and lateral-torsional and local buckling. However, the key point of this discussion is that in some cases, the beams may be significantly stronger and stiffer than perceived based on idealized engineering models. This could have beneficial as well as negative effects on system performance. For instance, what is the effect of underestimating the actual beam strength in a "strong-column weak-beam" design?

A more detailed issue related to beams is that beam element models should accommodate the development of plastic hinges within their length. A simple element of this type is presented by Abdel-Ghaffar (1992). Without this type of capability, plastic-hinge based programs often must utilize an excessive number of elements to represent the beam behavior in regions of the structure where gravity load effects may be significant.

5.1.3 Beam-Column Members

Advanced analysis concepts such as those reported in (SSRC, 1993) are directly applicable to the two-dimensional modeling of beam-column members in steel frameworks. These concepts can be applied within the context of force-space plasticity (Powell and Chen, 1987; McGuire, 1991; Zhao, 1993) to provide an elegant and efficient basis for modeling of cyclic member actions, and to represent the interaction between axial forces and bending moments.

Also, researchers have demonstrated success with flexibility-based approaches for analysis of cyclic inelastic behavior (e.g., Ricles and Popov, 1994; Kunnath et al., 1992; Taucer, 1991). Ongoing research at Cornell University (Attalla et al., 1994) is utilizing this approach to address the spatial behavior of wide-flange beam-columns, including inelastic torsional-flexural coupling. Implications of out-of-plane lateral-torsional buckling on advanced analysis and design of planar frames have been discussed by White et al. in (SSRC, 1993). Accurate prediction of inelastic lateral-torsional buckling and post-buckling performance of members loaded primarily in plane may be quite important for proper advanced analysis of some types of frame structures. Pi and Trahair (1993a & b) have presented new finite element formulations and impressive predictions of this type of behavior for isolated members subjected to static loading. These types of predictions generally require a large number of elements per member. One possible area of further research might be the development and use of adaptive analysis techniques, where the analysis might use coarser element discretizations (e.g., one element per member) with plastic hinge based procedures where these computational models suffice, and refined elements such as that developed by Pi and Trahair (1993) where more rigorous models are required to capture the performance. Existing computational models for steel and reinforced concrete column behavior need to be extended to address the static and cyclic performance of composite columns and beam-columns, particularly concrete-filled steel tubes (CFT) and concrete encased steel shapes (i.e., steel reinforced concrete, or SRC). There appears to be a dearth of computational research targeted at assessing the performance of these types of components as isolated elements or as a part of a structural system (Goel et al., 1993).

5.1.4 Bracing Elements

Recent developments in the modeling of bracing members have been reported in Abdel-Ghaffar (1992), Perotti and Scarlassara (1991), Tang and Goel (1989), and Andreaus and Gaudenzi (1989). These formulations address the degradation in stiffness and strength, and the detailed hysteresis behavior of members loaded predominantly by axial load. Modeling of damage accumulation and low-cycle fatigue is particularly important for these types of components, and has been addressed in a number of these studies. There is potential for significant improvement in modeling of the cyclic behavior of members loaded by bending and large axial compression by merging concepts from these research investigations with the beam-column modeling techniques discussed above.

5.1.5 Connection and Joint Elements

A broad suite of connection models is required to accurately characterize the behavior of the many types of FR and PR connections utilized in practice. With respect to cyclic loading, some types of connections exhibit stable cyclic performance at extremely large plastic excursions, e.g., (Popov and Stephen, 1972), whereas other types of connection elements exhibit stiffness and strength degradation, and pinching for all practical ranges of loading (Astaneh et al., 1989). Research in connection modeling must be tied closely with the compilation and analysis of experimental data. In addition to consideration of models for "all-steel" connections, particular attention should be given to models of column base connections such as those developed by Penserini and Colson (1991 & 1993). Also, the usefulness of composite semi-rigid connections has been demonstrated by Leon and his colleagues (Leon, 1992; Shin, 1992; Leon, 1990; Ammerman and Leon, 1987). Connections such as these should be included in comprehensive system modeling efforts. Representation of joint size, and modeling of panel zone actions has been identified by a numerous researchers as having a potential major effect on analysis predictions, e.g., Liew and Chen (1992), Goel and Boutros (1989) and Krawinkler and Mohasseb

(1987). The full behavior within beam-to-column joints needs to be considered to achieve a true comprehensive modeling capability for building structures.

Goverdan (1984) collected a total of 230 available experimental moment-rotation curves (from 1942 to 1982) and digitized them to form a database of connection behavior. Chen and Kishi (1989) extended this system to 303 tests. This database has been utilized extensively by engineers interested in semi-rigid connection behavior. It should be extended further to include the composite semi-rigid connection tests, column base connection tests, and beam-to-column joint knowledge discussed above. Available cyclic test data for a wide range of connection and joint types should be gathered from researchers throughout the world and placed within the system. At present, data on cyclic behavior of semi-rigid connections appears to be limited. The SSRC should be active in identifying where additional test data is needed, and the information required from new tests. The connection database should be integrated with other software capabilities developed for performance assessment such that an engineer can use connection models calibrated with the database directly in the analysis of structural systems.

5.1.6 Walls and Infilling

Structural walls and infilling can, of course, have a significant influence on the static and seismic performance of building structures. For example, surveys of earthquake damage have certainly demonstrated cases where shear failures have occurred in reinforced concrete columns due to the length of the column being shortened by the existence of a partial wall, etc. Recent efforts in the computational modeling of these types of components are outlined in Altin and Ersoy (1992), El Haddad (1991), and May and Naji (1991). Goel et al. (1993) points out a specific need to address the "accidental" composite action in older steel frames embedded in concrete and made with semi-rigid connections. Significant advantages in economy should be gained in certain cases by increased attention to elements other than simply the bare frame in comprehensive analysis of building structural systems, even when major structural walls, etc. do not exist.

5.2 STRUCTURAL SYSTEM PERFORMANCE VISUALIZATION

Research is needed to develop data management techniques and visualization tools to facilitate the definition of parameters and interpretation of results required for full-range performance assessment. Inherent in sophisticated computational models such as those discussed in Section 5.1 is the need to define properties that characterize the component actions, and the need to manage and synthesize the vast amounts of information generated by these tools. Techniques that work well for simple models of small to moderate size structures are in certain cases inadequate for complex models of large structural systems. Two major areas of work can be identified and are discussed below.

Techniques to facilitate the understanding of and definition of structural parameters and procedures associated with system performance assessment.

The amount of information needed to fully describe a structure for a comprehensive analysis can be overwhelming. In addition to the definition of the overall geometry, member properties, and loadings, the various parameters necessary to describe detailed component responses can be large in number and difficult to understand. Efficient interaction techniques are needed for the engineer to define, manage, and relate this information to the physical structure. These techniques need to be automated as much as possible - with as much of the information as possible being obtained from typical engineering drawings - while also giving the engineer the opportunity to review the data, helping him or her understand the significance of complex parameters, and providing easy capabilities for the engineer to modify the data such that the physical structure is appropriately modeled.

Nonlinear analysis programs typically consider one load combination at a time. However, in practice, engineers usually must consider many load combinations for all the limit states under consideration. For small to moderate size structures, the engineer should be given the capability to define - with automated support from the software - all the load combinations which should be considered for a particular structure at one time, including real or synthetic earthquake acceleration histories. He or she should then be able to distribute the various analyses all at once over a network of personal computers or workstations for processing. The software should be able to balance the compute processing over a number of heterogeneous selected machines, taking advantage of unused cpu cycles which may exist on idle machines within a computer network. The software should allow the engineer to monitor the success or failure of the multiple analyses, if so desired. However, for large structural systems, it may be desirable to focus on a single load case at one time, utilizing access to specialized parallel computers, or possibly a number of high-performance workstations in parallel across a computer network, to accomplish the calculations. On-going research at Purdue University is creating a software development environment with a goal toward providing substantial support for comprehensive modeling of building structural systems (Sotelino et al., 1994). A major component of this research, in addition to providing an extensible platform for development and testing of computational models, is attempting to alleviate portability problems in moving analysis software between various parallel hardware architectures.

For optimum success of the types of tools discussed in this paper, the reliability and robustness of the analysis computations must be high, such that the engineer can concentrate primarily on the structural behavior and the adequacy of the responses for the problem at hand. Although there are certainly many pitfalls and traps in the development of nonlinear component models and solution procedures, the author believes that this degree of reliability and robustness is achieveable with the present technology.

Comprehensive response visualization tools that will help the engineer synthesize and assess the behavior at overall and local levels.

Comprehensive analyses of real building structures will generate many sets of component responses. The analysis information must be available to the engineer in a flexible way so that he or she may assess the behavior at different levels of detail. Usually, a summary of the overall structure performance to a single or to all the multiple load combinations would be needed first. Therefore, user-selectable and definable "performance indices" are needed at the global level that summarize the overall behavior of the structure in a quantitative or qualitative way. After the overall response is understood, the engineer may wish to inspect the precise response histories within various components.

Research is needed to identify and demonstrate the use of suitable performance indices, and to develop efficient interaction and visualization tools to access and understand the analysis results. Examples of performance indices which might be considered include: (a) local ductility demand in various structural components (total and/or plastic rotations, shear deformations, curvatures, and/or strains), (b) cumulative inelastic deformations, (c) highlighting of locations where deformation or energy absorption capacities have been exceeded, (d) graphical displays of beam-column plastic zones, (e) plots of maximum story drift along selected column lines and comparison to the drift of the ideal elastic system, (f) story lateral displacement and story shear envelopes, (g) relationships of member forces to perfectly plastic strengths (quantification of the amount of strain hardening), (h) post-ultimate unloading characteristics of the system, (i) system overstrength ratios, i.e., reserve capacity beyond first yield, (j) number of plastic hinges initial indeterminacy or redundancy of the system, (k) other indicators of system redundancy, including reliability-based indicators, (l) shifts in the "effective centroid" of a building system with respect to strength and stiffness, (m) degree of non-symmetry with respect to strength, stiffness, or mass, and (n) contributions to sway from panel-zones, connections, beams, beam-columns, and other structural or non-structural components. Local behavior information should include moment, shear, axial force and other member diagrams, as well as plots of the interaction between moment, shear, and axial force at selected cross-sections.

Eventually, for these types of tools to be useful in design, the engineer should be able to make rapid observations of the responses from multiple nonlinear analyses, make appropriate design changes - perhaps with suggestions from the software based on a knowledge base which captures a defined level of structural performance, and then rapidly repeat the nonlinear analysis and structural performance assessment process until a satisfactory design is achieved. Design application of these kinds of facilities are certainly an ambitious, but worthy, goal. However, long before these types of tools approach any "main-stream" design use, they should be able to provide substantial information in research toward improving more conventional design practices.

With the availability of the computing power, extensive Monte Carlo simulations might be used in reliability analyses of nonlinear structures subjected to various types of loading. Previously, the computing requirements have exceeded ordinary computing capabilities and have prohibited the use of this type of reliability analysis. Therefore, approximate methods have been employed predominantly in previous work. However, advances in the computational models and in the computing speed and resources may permit highly accurate analyses in response statistics and failure probabilities. As a result, probability-based measures of performance might be provided in addition to the deterministic indices. These measures might include: (1) component and system level reliability measures, (2) relative uncertainty in response quantities (i.e., coefficient of variation in the maximum forces and deformations), (3) identification of 95th percentile (e.g.) response quantities for all components of the system, etc.

6. CONCLUDING REMARKS

The Structural Stability Research Council has played a key role in the development of and in establishing uniformity among contemporary procedures for stability design. With the tremendous growth in computational power available to practitioners and researchers, computational models are certain to become an increasingly important tool to support and enhance stability design practices. A large number of areas have been outlined in this paper in which further research is needed for substantial progress to be made in the application of computational methods for the realistic assessment of building structural system performance. The Structural Stability Research Council, through its task group and other activities, can serve as a catalyst for an integrated effort among practitioners and researchers toward the achievement of true comprehensive modeling capabilities.

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SECOND-ORDER INELASTIC ANALYSIS FOR FRAME DESIGN

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Summary

The necessity for considering *plasticity* and *stability* in steel building design is clear. Assembly of structural members through welding and bolting of joints introduces initial stresses and imperfections to the structural system. It is inevitable that even under working loads, some local plastic flow will develop in the best of designs. Under increasing loads, many parts of the structure will deform considerably and enter appreciably into the plastic range before reaching its load-carrying capacity. An exact *second-order inelastic analysis for frame design* has been considered in the past to be a formidable task, and neglecting either *plasticity effect* in a second-order elastic analysis or *stability effect* in a first-order plastic analysis for the present LRFD design provides a practical and simple alternative. However, the advancement of computer hardware and software in recent years has made it feasible for the engineer to adopt the *advanced analysis* by combining the theory of plasticity with the theory of stability in such a way that separate specification member capacity checks can be avoided. This approach will simplify drastically the analysis/design process with the more use of computer and provide a more realistic prediction of load effects and of overall structural system performance. As a result it will yield a greater economy and a more uniform safety. This development is described in the present paper.

1. Introduction

At the present time there is a two-stage process in the current design operation: firstly, the forces acting on the structural members are determined by conducting a structural system analysis, and secondly, the sizes of various structural members are selected by checking against specification member capacity formulas. The behavior and strength of structural members have been the subject of research for many years. Accurate information regarding their behaviors throughout the entire range of loading up to ultimate strength, and simple procedures and formulas to enable designers to assess this behavior are readily available in the open literature and well-defined in various design specifications (Chen and Atsuta, 1976,1977). Despite this the structural system analysis aspect of the design operation remains almost the same, and analysis methods are, for the most part, based on the simple first-order elastic analysis. There exists therefore a fundamental *incompatibility* problem in the two-stage operation: the simple *first-order elastic analysis* method is used to determine the forces acting on each member of the system (dashed line, Fig. 1), while the advanced *second-order inelastic analysis* method is used to determine the strength and behavior of each member (solid curve, plastic zone theory, Fig. 1), treated as an isolated member.

However, the design of structural members in frames must take into consideration the effect of the *interaction* among adjacent framing members including second-order P-delta effects. In the current engineering practice to design, this interaction effect is accounted for approximately by the use of *effective length* as shown in Fig. 2 for a rectangular rigid frame. For the braced column the effective length must always be less than the actual story height (Fig. 2b) but that for the unbraced column it will always be greater (Fig. 2c). Where the relative stiffness of the beams and columns are known, the effective length can be computed directly by using *bifurcation analysis* and by obtaining the elastic critical load as shown by the upper horizontal line in Fig. 1 (Chen and Lui, 1987).

The additional moments that arise as a result of the second-order *P-delta effects* are taken into account in the current LRFD design by the use of two *moment amplification factors* B_1 and B_2. By multiplying the maximum first-order elastic moment by the B_1-factor, we can obtain the maximum moment in a member accounting for the P-δ effect. While multiplying the first-order elastic sway moment by the B_2-factor, we account for the P-Δ effect. The moment amplification factors are obtained by using the *second-order elastic analysis* that includes only the effect of change of geometry and instability (broken curve, Fig. 1) (Chen and Lui, 1987).

The piecewise linear curve without a descending branch as shown in Fig. 1 represents a *first-order-plastic hinge analysis* when the effects of the change of geometry or stability effect are ignored. Its peak value is the *plastic limit load* that can be obtained directly by a *simple plastic analysis* (horizontal dashed line marked plastic mechanism load in Fig. 1). The LRFD member capacity formulas must still be used to check the capacity of each member of a structure designed by the simple plastic analysis including the use of effective length factor K and amplification factors B_1 and B_2. The piecewise linear curve with a descending branch shows the result of a *second-order elastic-plastic hinge analysis* allowing a simple plastic hinge analysis for the change of geometry which is associated with the increase of sway deflection (or the P-Δ effect). When the spread of plastic zones, residual stresses, initial imperfections and strain hardening are all accounted for, the full smooth curve is obtained by a *second-order spread-of-plasticity analysis*. The peak load of this curve gives the *true* strength and *actual* failure mode of the plane frame.

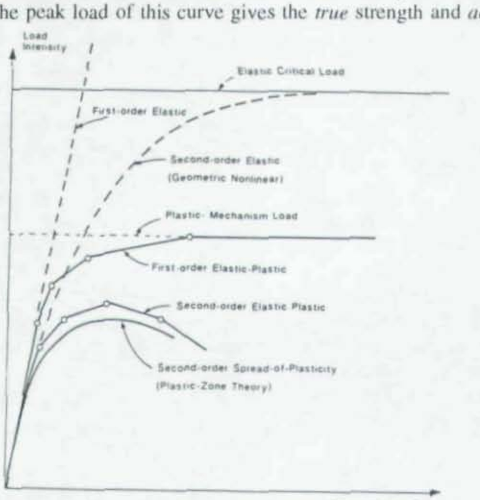
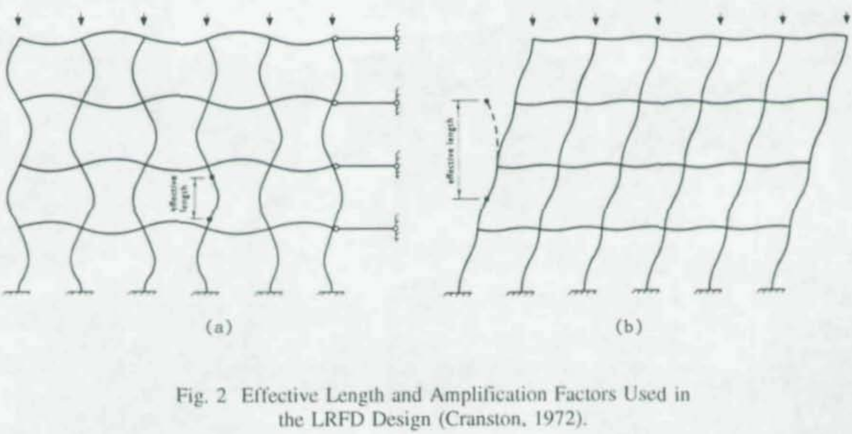


Fig. 1 Load-Deflection Behavior of Plane Frame with Different Analysis Methods (Chen and Lui, 1991)

The second-order plastic-hinge analysis or the second-order plastic zone analysis is referred to here as the *second-order inelastic analysis* methods for frame design. Any analysis methods that sufficiently represent the strength and stability behavior of a structural system such that separate specification member capacity checks are *not* required are defined in this paper as the *advanced analysis*. The applications of the second-order inelastic theory to the development of advanced analysis procedures for practical design use is the current focus of intense research worldwide including major contributions from several specification committees such as Australia (AS4100, 1990) and Europe (EC3, 1990), as well as the two task groups in the U.S.: the AISC Technical Committee 117 on *Inelastic Analysis and Design* and the SSRC Task Group 29 on *Second-Order Inelastic Analysis for Frame Design*, among others (White et al. 1991).

2. Trends Toward Advanced Analysis

With the rapid advancements in computer hardware, particularly in the computing and graphics performance of personal computers and workstations, the direct applications of advanced methods of analysis for practical design use have become more and more feasible and attractive. As a first step, for example, instead of using the B_1/B_2 amplification factors, a direct second-order elastic analysis can be used efficiently, and in certain respects more simple to perform than B_1/B_2, to determine the second-order moment for member design, especially for the analysis and design of a complex *semi-rigid structural system. Under the limit-design concepts, and coupled with* reasonable computing facility, direct second-order elastic analysis is certainly the emerging technology that will become the common practice in the not too distance future. To this end, a recent book by Chen and Toma (1993) is worth mentioning where a practical second-order elastic analysis program (Chapter 2) and a steel connection data bank program (Chapter 3) are made available in the form of a floppy disk together with a user's manual for easy use. Furthermore, benchmark problems around the world were also collected in Chapter 7 for calibration and verification of these computer programs, among others.



(a) (b)

Fig. 2 Effective Length and Amplification Factors Used in the LRFD Design (Cranston, 1972).

However, it is important to note that through the direct second-order elastic analysis, the need for the calculation of B_1/B_2 amplification factors has been eliminated, but the calculation of effective length factor is still required to determine the axial resistance for design by the current LRFD provisions. Although the second-order elastic analysis program can also be used directly to perform eigenvalue analysis (bifurcation analysis) of the structural system to compute the effective length of rigid or semi-rigid frames, the time-consuming process of checking the second-order forces and moments for each member against the specification member interaction equations is still necessary involving further estimation of effective length factors. Because of these approximations, the maximum strength of the structural system is still not determined. The *only way to assess the real performance of a structural system is through a direct second-order* inelastic analysis.

With the availability of more powerful workstations, it is now realistic to develop advanced analysis method for direct frame design. All the technologies are now in-place, the real issue at the present time is at what level of second-order inelastic analysis do we need to develop for practical use. This will be described in the following section. Once this analysis technique is in place, the need of estimating various factors for specification member capacity checks can be totally avoided. Advanced analysis method, when coupled with integrated graphic analysis and design system, allows the engineer to exercise greater freedom in structural design. Its use can assist in providing both efficient and cost-effective design solutions.

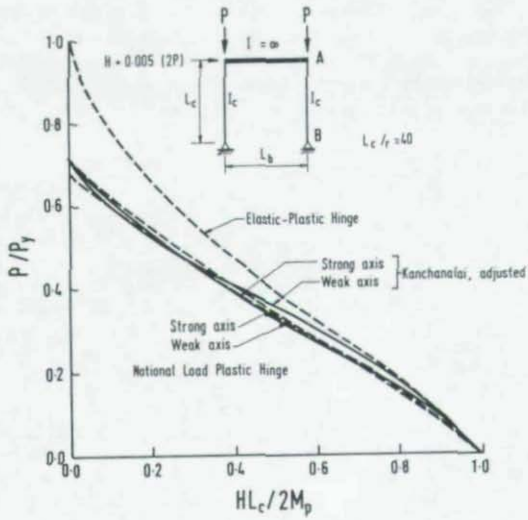


Fig. 3 Notional Load Plastic Hinge Analysis Applied to Frame Columns (Liew et al., 1994).

3. Practical Advanced Analysis Methods for Frame Design

There are generally two types of inelastic analysis for frame design:

1. The *plastic zone* or *distributed plasticity*, and

2. The *plastic hinge* or *concentrated plasticity*.

The elastic-plastic hinge analysis is the simplest approach, while the plastic-zone analysis exhibits the greatest refinement. To develop an advanced analysis method for practical use, an improvement over the elastic-plastic hinge analysis is generally made. This will result in less computational effort but better accuracy that is acceptable for actual design use. Two analysis methods have been developed and refined in recent years to achieve both simplicity in use and, as far as possible, a realistic representation of actual behavior. Extensive comparisons have also been made with the results of plastic-zone analysis and of the LRFD Specifications, providing the necessary confirmation of the validity of these analysis methods. These two methods are recommended for possible implementation for general use. A brief summary of these two advanced analysis methods is given in the forthcoming.

3.1 Notional Load Plastic-Hinge Method

To improve the second-order elastic-plastic hinge analysis for frame design, artificially large values of frame imperfections (i.e., initial out-of-plumbness) are introduced in the frame analysis. This is the general approach adopted by EC3 (1990) for frame design using second-order plastic-hinge analysis. In addition to accounting for standard erection tolerance for out-of-plumbness, these artificial large imperfections are intended also to account for the effects of such factors as residual stresses, frame imperfections, and distributed plasticity etc. that are not considered in the usual plastic-hinge-based frame analysis.

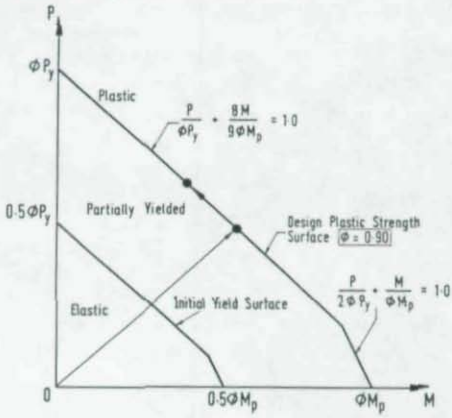


Fig. 4 Gradual Stiffness Reduction Scheme Through Two-Yield-Surface Approach for Refined-Plastic-Hinge Analysis (Liew et al. 1993).

Alternatively, Liew et al. (1992) utilizes a set of equivalent *notional lateral loads* to account approximately for the influence of the above-mentioned factors on the overall system strength. These notional lateral loads are expressed as a fraction of the story gravity loads or member axial force, and are chosen in such a way that will reflect adequately those effects that are not considered in the usual frame analysis. This concept is similar to the "enlarged" geometric imperfection approach adopted by EC3, but the actual determination of the notional lateral load value is achieved by the application of the notional load analysis to simple column case and calibrated against the current LRFD column curve. It is found that the equivalent notional load corresponding to the LRFD column curve is approximately 0.005P. Figure 3 illustrates the interaction strength curve obtained by the application of this approach for a simple frame. The results are compared with the adjusted plastic-zone solutions generated by Kanchanlai (1977) and the conventional elastic-plastic hinge solution. This frame example is one of the worst cases from the more comprehensive set of problems considered by Liew (1992).

3.2 Refined Plastic-Hinge Analysis

The basic quantity required in the inelastic analysis of frames is the value of EI, which is the tangent stiffness of the cross section, or here can be considered as the slope of the relation between moment and curvature. For a conventional elastic-plastic-hinge analysis, this presents no difficulties, but in the case of a more refined analysis there are problems, because the moment-curvature response is nonlinear reflecting a continuous smooth degradation of stiffness from the elastic stiffness at the initiation of yielding to the zero stiffness at the formation of plastic hinge. This smooth degradation of bending stiffness is due to the spread of plasticity in the cross section as the bending increases. The gradual stiffness degration as yielding progress through the volume of the member can be represented by modifying the basic elastic-plastic hinge concept from section response to member response such that the member stiffness degrades gradually from the stiffness associated with the onset of yielding, to that associated with the cross-section plastic strength.

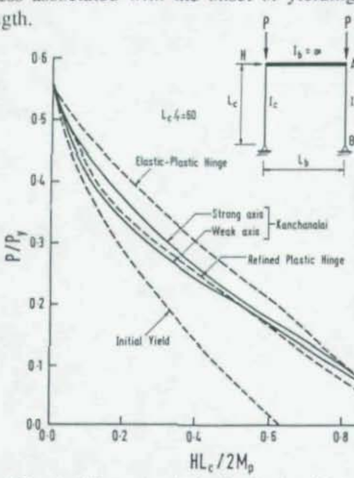


Fig. 5 Refined-Plastic-Hinge Analysis Applied to Framed Columns and Also Compared with Other Analysis Methods (Liew et al. 1993).

To this end, a gradual stiffness reduction scheme with initial yielding assumed to occur at a yield surface that has the same shape of and equal to one-half the size of the plastic strength surface is chosen by Liew et al. (1993) to represent the inelastic behavior of beam-columns subjected to a combined action of bending and axial forces. This is illustrated schematically in Fig. 4 where the LRFD interaction equations for beam-column design are used for the present refined analysis. This approach will improve significantly the accuracy of the conventional elastic-plastic hinge approach while maintaining its simplicity and we shall call this method*: the refined plastic-hinge analysis".*

Figure 5 compares the in-plane strength curves obtained by the refined-plastic hinge analysis with the results from the conventional second-order elastic-plastic hinge analysis and Kanchanlai's "exact" plastic zone analysis (Kanchanalai, 1977). The onset of yielding in the framed columns is seen to occur long before the ultimate strength of the columns is reached. This observation is true for almost all range of loading except when the frame is subject to gravity load only. The refined-plastic-hinge analysis predicts a smaller load-carrying capacity for this frame compared to the elastic-plastic hinge analysis. The results of the refined analysis are closer to the strength curves obtained by the plastic zone analysis.

4. Comparative Study of a Six-Story Two-Bay Frame

The frame as shown in the inset of Fig. 6 was proposed by Vogel (1985) as an European calibration frame for second-order inelastic analysis (Toma and Chen, 1992). Both gravity and lateral loads are applied proportionally till failure occurs. The applied load versus top- and fourth-story lateral displacement curves corresponding to three advanced analysis methods are shown in Fig. 6. An attempt is also made to evaluate the limit of resistance of this frame by the LRFD -elastic and -plastic analysis approaches. Details of these analyses can be found in the thesis by Liew (1992).

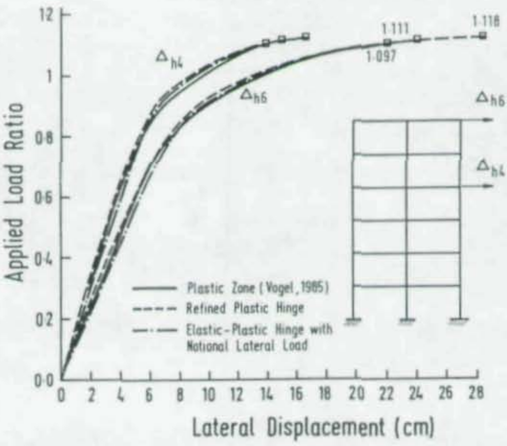


Fig. 6 Comparison of Load-Displacement Curves for the Vogel Frame (Liew et al., 1993).

4.1 LRFD - Elastic Analysis

Member forces are obtained using second-order elastic analysis and all member forces must satisfy the LRFD beam-column strength interaction equations. The maximum strength of the frame is reached when any member of the system reaches the limiting strength specified by the interaction equations. Side-sway imperfections given in this calibration frame are not used as not required in the LRFD analysis.

To evaluate the limit of resistance of each member in the system by the LRFD method, the effective length factor for each column must be determined first. The results of K-factors as determined by the LeMessurier equation (1976,1977) are shown in Fig. 7. Also included in this figure are the beam-column strengths evaluated based on the LRFD beam-column interaction equations. The most critically loaded member in the frame is found to be the roof beam shown in Fig. 7 and the corresponding load of this frame is defined as the maximum strength of the system. The results suggest that the LRFD-elastic analysis procedure is *always* conservative, since most lower-floor beams and columns in the frame still have significant capacities that have not been fully developed through the inelastic force redistribution after the strength of the most critically loaded member is reached. This inelastic force redistribution is not explicitly considered in the present LRFD-elastic analysis and design process.

4.2 LRFD-plastic analysis

The LRFD specification also permits the use of the first-order plastic-hinge analysis to evaluate member forces (AISC, 1986). The member forces computed in this way will include the inelastic redistribution of forces in the system, and if the evaluation of the strength of the calibration frame is based on this approach, then the process will not lead to unduly conservative result.

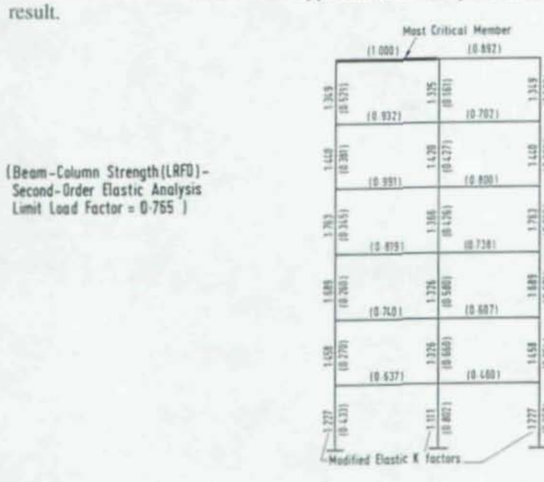


Fig. 7 K-Factors and Beam-Column Strengths from LRFD-Elastic Analysis (Liew et al., 1993).

However, in actual calculations based on the simple plastic analysis, we found that the procedures for determining the moment amplication factors B_1/B_2 and the effective length factors K for members that are connected to a hinged subassemblage are very complicated and it also requires the information on the location of the last plastic hinge formation. Although attempt was made to develop practical procedures to overcome this difficulty (Abdel-Ghaffar and Chen, 1992), but the accuracy of these factors for representing the true member and system behavior is questionable. Clearly, a direct application of the conventional first-order elastic-plastic-hinge analysis to the frame is much simpler than the use of the LRFD-plastic analysis. Of course, the more rational way of estimating the member and system strength, which also avoids the complex issues of evaluating various factors for member capacity checks, is through the direct use of advanced analysis.

4.3 Notional Load Plastic-Hinge Analysis

A notional lateral load equal to 0.5% of the total gravity loads acting on the story is applied at the top of the column at each floor level. The analysis details are the same as the elastic-plastic-hinge method except that explicit modeling of initial imperfections is not required in this approach.

As shown in Fig. 6, all the inelastic analysis methods predict essentially the same maximum strength. The maximum frame resistance is reached at a load parameter 1.097 for the present notional load plastic-hinge method, compared to 1.111 for the Vogel's plastic-zone study, 1.118 for the refined-plastic hinge analysis, and 1.124 for the elastic-plastic hinge analysis (not shown in the figure). The maximum difference between these limit loads is less than 2%, supporting the conclusion that when the overall nonlinear behavior of the frame is dominated by inelastic action in the beams, the plastic-hinge-based methods generally give sufficient representation of the overall frame behavior (White and Chen, 1993).

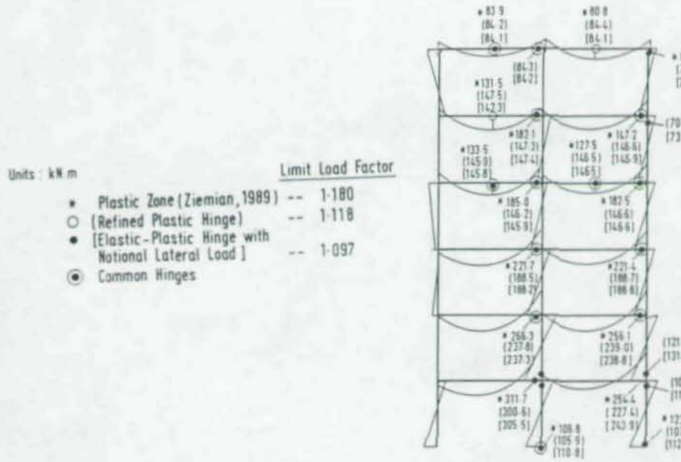


Fig. 8 Comparison of Bending Moments and Plastic Hinge Locations for the Vogel Frame at the Limit Loads (Liew et al., 1993).

4.4 Refined Plastic-Hinge Analysis

The degradation of effective member stiffness is based on the tangent modulus and parabolic function (Liew, 1992). The same stiffness reduction scheme is used for all members in the calibration frame. Initial side-sway imperfections similar to the plastic zone analysis are explicitly considered in the refined plastic-hinge analysis and the CRC tangent modulus column expression is used for the stiffness degradation.

Figure 8 shows the comparison of bending moments and locations of plastic hinges at the *limit loads computed by three inelastic analysis methods. The distribution of forces predicted* by all these methods are remarkably close. As for the plastic hinge formations, the first plastic hinge predicted by the notional load plastic hinge analysis occurs at an load level of 0.752, whereas in the present refined-plastic-hinge analysis, no plastic hinge is observed until the load level reaches a value of 0.873. The plastic-hinge analysis detects a total of 21 plastic hinges in comparison with the refined-plastic-hinge analysis which detects only 15 plastic hinges in the frame.

4.5 Plastic-Zone Analysis

The plastic-zone analysis is considered to be the "*exact*" analysis in which different residual stresses for members of different cross-section dimensions are considered, and initial side-sway imperfections as specified by EC3 (1990) can be explicitly modeled. Material straining hardening can also be included in the present analysis.

Percentage of yielded areas

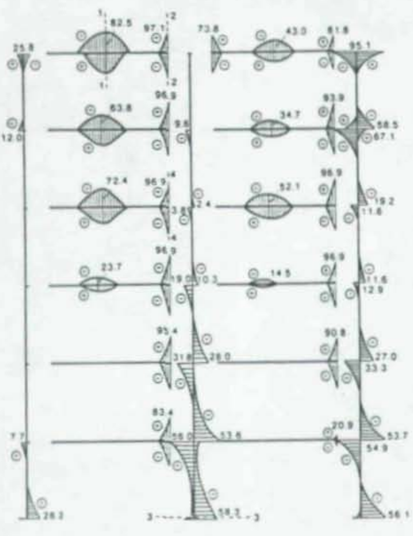


Fig. 9 Spread of Plastic-Zones for the Vogel Frame (Vogel, 1985).

In contrast to other inelastic analysis methods, the plastic-zone analysis can also give a clear picture of how the spread of yielding develops in the system. This is illustrated in Fig. 9 at the collapse state of the frame. The columns at lower levels are seen to reach collapse by an almost full plasticity, and at this state, not much strength remains to sustain further loading.

For high-rise building frames, both plastic-hinge analyses compare well with the plastic-zone method as shown previously by the closeness of the load-displacement curves in Fig. 6 and the force distributions in Fig. 8. The inability of the conventional elastic-plastic hinge method to represent member strength or simple structures such as columns or portal frames can be attributed as *"local effects"*. That is, the conventional plastic-hinge method is accurate in predicting the system response, but may not be accurate in predicting the strength and stability of isolated beam-column elements or subassemblies. This inadequacy of the elastic-plastic hinge approach can be improved by the use of the notional load method or the refined-plastic-hinge method described previously.

5. Conclusions

This state-of-the-art review of the second-order inelastic analysis method for frame design presents a few facts and techniques of immediate use to the designer of building structures. Emphasis is placed on the *system* and *plastic* behavior, and not on the *member* and *elastic* behavior, because failure of structures is generally preceded by appreciable plastic deformation leading to system instability. With today's computing technology, a direct use of the second-order inelastic analysis for frame design without specification member capacity check is far simpler than the present first-order elastic analysis for the system and the ultimate strength design for the member as used widely in the current engineering practice.

The main objective of the paper is to convey the vision of a very bright future for the structural engineering profession today. Basic theories of structural stability and material plasticity already established analytically and verified experimentally together with the rapid development of high performance computing (Chen et al. 1993) point the way to direct design of structures as a whole without the time-consuming process of specification member capacity check. Close estimate of the load-carrying capacity of building structures by the advanced analysis methods are feasible at present. If enough effort is expanded by both academic workers and consulting engineers in practice, each learning from the other, much of the present design art and guesswork can be replaced by a scientific and straightforward procedure based on today's computing technology. As a consequence, the art of structural engineering will be significantly advanced (Chen, 1993).

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NEW CONSTRUCTIONAL STEELS AND STRUCTURAL STABILITY

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Introduction

Recent advanced technology in materials science has made easy to respond to the user's needs on high performance steels in naval, offshore, civil and building structures. Under severe natural environment, thicker steel plates of 50mm to 100mm for large scale structures are required to have high strength, high fracture toughness, longer fatigue life, high corrosion resistance and better weldability. Such properties and high productivity are made partly possible by introducing a new Thermo-Mechanical Control Process(TMCP) with controlled rolling and the accelerated cooling process instead of the conventional normalizing process. TMCP may be called under the name of Thermo-Mechanical Controlled Rolling (TMCR) Process or Controlled-Rolling(CR) Process.

Fig. 1 shows the improvement of weldability of TMCP in relation to the carbon equivalent C_{eq} values of SM490 and A242 steel grades(Ohashi 1990). It is observed that approximately 40 N/mm² higher yield stress is obtained by the TMCP method than by the normalizing process. Reduction in the C_{eq} value improves the welded joint toughness and reduces preheating temperature and heat affected zone hardness. The TMCP high strength steel can be applied to increase efficiency in welding fabrication.

In buildings and bridges, the user's needs in mechanical properties of constructional steels may be summarized as:

1) High strength steel with low yield-to-tensile strength ratio(LYR) to assure the inelastic deformation capacity of structures under earthquake motion.
2) Narrow variation of yield stress which assures the design calculated sequence of plastic hinge formation in the structures. Holding a constant yield stress level from medium to thick plates.
3) Low-yield steel which exhibit yield stress less than the conventional mild steel (SM400, A36). The low-yield stress with high ductility behaves like "fuse" which can dissipate seismic energy while the rest of the structure remain elastic or undamaged.
4) High Young's modulus steel which has higher than the normal value is produced

experimentally by applying a cold-rolling process in manufactoring. High Young's modulus provides great benefits in the structural functions.

High Strength Steel with Low Yield-to-Tensile Strength Ratio (LYR HS Steel)

Stress-strain curves which are constructed from the tensile coupon tests are schematically shown in Fig. 2 from mild steel to quenched and tempered high strength steel. Steel grade of SM400A is equivalent to ASTM A36, SM490A to A441. SM570 to A572 and HT780 to A517. It is apparent for any class of steel if the strength increases, then the yield-to-tensile strength ratio(YR) also increases and the elongation decreases. YRs for the above four steel grades are 0.65, 0.74, 0.86 and 0.91, respectively from the tensile coupon tests. A stress-strain curve of low-yield stress steel is also given for comparison.

Fig. 3 shows the plotted yield stress F_y and tensile strength F_u from the results of 1612 tensile coupon tests. The regression analysis gives the least squars line, Eq.(1), with the correlation coefficient R=0.963 as shown in Fig.3(Itoh, 1984).

$$F_y = 1.205F_u - 245.6 \text{ (MPa)} \tag{1}$$

or YR is given by

$$YR = F_y / F_u = \{0.83+(203.8/F_y)\}^{-1} \tag{2}$$

Honshu-Shikoku Bridge Authority(HSBA) specifies the allowable tensile stress for high strength steels considering F_y and the values of YR. For quenched and tempered high strength steel HT780 ($F_y > 685$MPa, $F_u > 780$ MPa for t=8~75mm) steel, the allowable tensile stress Fta is given by the smaller value in Eq.(3).

$$Fta = \text{Min } (F_y / 1.7, F_u / 2.2) \tag{3}$$

where material safety factor is 1.7 for F_y and 2.2 for F_u. F_y is defined by the 0.2% offset yield stress for high strength steels.

From Eq.(3), Fta is governed by $F_y/1.7$ when the yield ratio is limited by $YR \leqq 0.77$ (=1.7/2.2), that is, $F_u \geqq F_y/0.77$. The dotted line in Fig. 3 represents Eq.(4) which connects(F_y, F_u)=(235, 400)for mild steel and (685 and 885)for new LYR HS steel. This straight line is given by Eq.(4).

$$F_y = 0.928F_u - 137.8 \text{ (MPa)} \tag{4}$$

or the value of low yield ratio as,

$$LYR = \{1.08+(148.5/F_y)\}^{-1} \tag{5}$$

Fig. 4 shows Fy-YR the curves of Eqs.(2) and (5) and the plotted points indicate the values of F_y and YR using the minimum yield and tensile strength both are specified in the indicated material standards. Target values of the LYR high strength steels in steel manufacturing are plotted for comparison with Eq.(5). Eq.(5) can serve as a target line for the future development of the new steels.

Uniform strain at the tensile strength F_u is determined by the following equation considering the strain hardening effect(Sato, 1969).

$$YR = F_y / F_u = \exp[-e_n(\ln e_n + 4.3)] \quad (6)$$

where the true strain e_n is expressed by the uniform strain ε_u as,

$$e_n = \ln(1+\varepsilon_u) \quad (7)$$

Equating either Eqs.(2) or (5) with Eq.(6), ε_u-F_y relations can be obtained for the conventional steels and for the LYR HS steels. HSBH Specification simply defines the linear relation of ε_u-YR relationship as

$$\varepsilon_u = 0.60\,(1-YR) \quad (8)$$

Fig. 5 shows the ε_u-F_y relationship for the conventional and LYR HS steels with the tensile coupon test results for various steel grades. The dotted curve shows the ε_u-F_y relationship using Eq.(8) in place of Eq.(6).

In Fig. 5, test points ○ of the conventional steels are well distributed along the ε_u-F_y curve showing the uniform strain ε_u decreases with the increase of F_y. Test points ● of low yield ratio steels which were conducted at Osaka University are small in number and more test results would be needed in the high strength F_y=650MPa class steel. The present ε_u-F_y relation will serve as a target curve in order to produce more reliable low YR high strength steels.

Narrow Yield Stress Range Steels

There exist sufficient options in chemical compositions and processing methods to allow steel producers to meet the material requirements in the specifications.

The yield stress variation depends largely on chemical compositions and on the rolling conditions, and the variation can be controlled in order to meet the required quality control of the product. In the material specifications, only the guaranteed minimum values are specified in the yield stress and their actual yield stresses are usually much higher than the specified values and thus the material ductility may be reduced. On the other hand, since the tensile strengths have their lower and upper bounds in the specifications, the variation of the tensile strength becomes less compared to the yield stress. The coefficients of variation(COV) of F_y and F_u

can be listed up from the published statistical data including the effects of plate thickness and steel grades.

	COV	
	Fy	Fu
Galambos(1978)	0.10–0.11	–
Itoh(1984)	0.110–0.116	0.060–0.063
AIJ LSD(draft)(1990)	0.090–0.103	0.037–0.041
Narrow Range Steel	0.0407, 0.0408	–

JIS(Japanese Industrial Standards) has been preparing a new material standard for building structural use to specify the maximum and minimum bounds of the yield stress variation and the recent draft specifies the following guaranteed values for two steel grades SN 400B and SN 490 B. Range between the maximum and the minimum bounds has a constant values of 120MPa from t=12 to 40mm of the two steel grades as shown below:

	t(mm)	6–12	12–40	40–100
SN 400B	Fy min/Fy max(MPa) Fu min/Fu max(MPa) yield ratio(%)	235over	235/355 400/510 80% less	215/355
SN 490B	Fy min/Fy max(MPa) Fu min/Fu max(MPa) yield ratio(%)	325over	325/445 490/610 80%less	295/415

The maximum and minimum yield stress levels shown by the solid lines are given in Fig. 6 and compared them with the test results(Ohashi 1990) of the conventional similar strength grade steel. The dotted line is for 315MPa between t=16~40mm of the coventional steel.New standard can control high yield stresses for medium plate thickness.

Narrow yield stress range steel of SN 400 and SN 490 grades for plate thickness of 16~30mm are manufactured under carefully controlled rolling conditions. Following statisfical data are obtained from the mill yield tests.

	No. of specimens	mean	standard deviation	COV
SN 400 class	310	286.6	11.67	0.0407
SN 490 class	109	371.2	17.78	0.0478

The COV values of the new narrow yield stress range steels are less than half of the conventional steels. The characteristic yield stress F_{yk} what is defined by 5% probability less than F_{yk} can be increased by 14% for the narrow range steel of COV=0.0407 compared to the conventional steel of COV=0.110.

Eurocodes No.8 Structural Design in Seismic Zones states that for dissipative parts of earthquake resistant structures not only the lower value but also the upper value of the yield strength shall be specified when the material is ordered and sufficient control must be undertaken to avoid overstrengths.

Ultimate Plate Strength of LYR HS Steel

1) Plates in Compression

In the elasto-plastic finite displacement plate analysis, the different material stress-strain curves obtained from the tensile tests and the idealized models as shown in Figs. 7 are used to compare the ultimate strength and ductility of the plates(Fukumoto and Nishimura 1992). These curves represent B-1 as a conventional HS steel, A-2 and A-3 as LYR HS steel with different yield plateau and yield ratios. After the onset of strain *hardening, the fitted formula is used for the strain-hardening range until* to reach the uniform strain ε_u.

In the analysis, the same magnitude of welded type residual stress pattern and of initial deflection of plate are adopted for simply supported square plate.

Fig. 8 shows an example of the analytical results of the plate strength for different material properties. Post-yielding strength is due to the strain hardening effects. The maximum plate slenderness for $\bar{\lambda}_{p)max}$ for the yield strength N_y are influenced by the strains at the initial strain hardening ε_{st} and by the yield ratio as listed in the next table. In the abscissa, $\bar{\lambda}_p$ is defined by,

$$\bar{\lambda}_p = \sqrt{F_y / F_{cr}} = 0.526\,(b/t)\sqrt{F_y / E} \qquad (9)$$

where b/t is width-to-thickness of plate.

	A-2	A-3	B-1
$\bar{\lambda}_{p)max}$	0.66 (130%)	0.55 (108%)	0.51 (100%)
ε_{st}(%)	0.48	1.80	1.40
YR	0.70	0.77	0.88

The maximum b/t ratio limitation can be relaxed by 30% in low YR HS steel A-2 compared with the conventional HS steel. In Fig.8, the *mean ultimate compressive* strength curve is obtained analytically using the measured initial imperfection data.

2) I-Section in Bending

Coupled strength of flange and web plates of I-section in bending is analysed by the elasto-plastic finite displacement theory(Fukumoto and Nishimura 1992). Stub beam is analysed to prevent the coupled effect with the overall instability of beam.

In order to compare the analytical results, the same magnitude of welded residual stress pattern and of initial distortional shape of the cross sections is specified for B-1, A-2 and A-3. Plate slendernesses $\overline{\lambda}_{pf}$ and $\overline{\lambda}_{pw}$ of flange are web are defined, respectingly, as,

$$\overline{\lambda}_{pf} = \sqrt{F_y / F_{cr)f}} = 1.61\ (b / t_f) \sqrt{F_y / E} \qquad (10)$$

$$\overline{\lambda}_{pw} = \sqrt{F_y / F_{cr)w}} = 0.0463\ (h_t / t_w) \sqrt{F_y / E} \qquad (11)$$

where $F_{cr)f}$ and $F_{cr)w}$ are elastic plate buckling stresses of flange in compression (the buckling coefficent k_f=0.425) and web in bending, (k_w=23.9), respectively, and b the outstand flange width. $M_u/M_y-\overline{\lambda}_{pw}$ and $M_u/M_p-\overline{\lambda}_{pw}$ relations are determined for the specified flange plate slendernesses $\overline{\lambda}_{pf}$, t_w=10mm and A_f/A_w=1.5.

Fig. 9 shows an analytical result of the ultimate bending strength for the material properties B-1 and A-2. As compared with the conventional B-1 steel, A-2 steel demonstrates high bending capacity for the specified flange-and web-$\overline{\lambda}_p$ values. The maximum width-to-thickness ratios of cross-section which deliver the plastic moment M_p are determined from the intersecting points of the strength curves and M_u/M_p=1 as shown in Fig.9. Combinations of the maximum width-to-thickness ratios of the flange and web plates for M_y and M_p are listed in the below table.

	Mp-section			My-section		
	$\overline{\lambda}_{pw}$			$\overline{\lambda}_{pw}$		
$\overline{\lambda}_{pf}$	A-2	A-3	B-1	A-2	A-3	B-1
1.0	0.441	0.318	0.315	0.682	0.701	0.720
0.8	0.547	0.386	0.395	0.850	0.871	0.914
0.6	0.665	0.540	0.540	1.175	1.133	1.138
0.4	-	0.591	0.587	-	-	-

The maximum plate slendernesses in the yield cross section for the three steels are apparently the same ones. However, for the plastic cross section, A-3 and B-1 give the similar slenderness values as compared with A-2 which has low yield-to-tensile strength ratio and short yield plateau.

3) Ductility of I-Beam in Bending

Energy dissipation index Umax can be defined by the area under a M-θ curve up to Mmax showing a shaded area in Figs. 10 where M is the equal end moments and θ is the end slope of the stub beam. Figs. 10(a), (b)and (c) show the Umax area for various combinations of the plate slenderness $\overline{\lambda}_{pf}$ and $\overline{\lambda}_{pw}$ values. Figs.10(a) and (b) show M-θ curves of A-2, A-3 and B-1 for $\overline{\lambda}_{pf}$=0.4 and $\overline{\lambda}_{pw}$=0.6 giving full plastic moment. The energy dissipation ratios of Umax for B-1, A-2 and A-3 are 1 : 2.94 : 1.03. For $\overline{\lambda}_{pf}$=0.4 and $\overline{\lambda}_{pw}$=1.0, the cross-section of B-1 and A-3 are terminated by the coupled plate instability before reaching M_p. The Umax ratios for B-1, A-2 and A-3 are 1 : 5.81 : 1.02. In both cases, M/M_p~θ/θ_p curves and thus the Umax values are similar for B-1 and A-3 sections. The A-2 plastic section possesses large energy dissipation of approx. 3 times of B-1's for $\overline{\lambda}_{pf}$=0.4 and $\overline{\lambda}_{pw}$=0.6 and of approx. 6 times for $\overline{\lambda}_{pf}$=0.4 and $\overline{\lambda}_{pw}$=1.0.

The advantage of A-2 can be obtained by the conditions that the steel has low yield ratio and short yield plateau before strain hardening set in.

4) Stiffened Plates

Nara(1981, 1994) discussed the ultimate compressive strength and ductility of longitudinally stiffened plates with different mechanical properties of steel including low YR HS steels. The numerical analyses are made for constant value of Fy= 450MPa with the yield ratios of 0.60, 0.65, 0.70, 0.75 and 0.79 and ε_u=10% and 20%. The ductility capacity and thus the energy dissipation index Umax can considerably be increased by lowering the yield-to-tensile strength ratio and the small value of $\overline{\lambda}_{pf}$=0.3 for the sub-panel plate when the stocky stiffeners are provided. The effects of the stiffener parameters in flexural, torsional and extensional rigidities are discussed on the ultimate strength and ductility of the stiffened plates.

5) Framed Structures

Kuwamura(1989, 1992) demonstrated the advantages of low YR HS steels in the application of earthquake resistant building structures. The results show that the low YR HS steel exhibits larger energy dissipative capacity in the buildings where plastic deformation is required as a source of energy dissipation. Kuwamura(1989, 1990), discussed the yield stress variation has a predominant influence on the plastic failure mechanism of the earthquake resistant multistory structures. Sequences and locations of the plastic hinge formations until to the failure may be changed from the weak-beam strong-column structures due to the much higher variation of the maximum yield stresses and thus cause unexpected performances of the structures.

Concluding Remarks

Modern innovative technologies in steelmaking has been adding high performance quantities to the conventional steels. Thermo-Mechanical Control Process Method is one of the technologies which allows to produce easy weldable steels with good fracture toughness properties and high strengths for plate thickness up to 100mm.

The behaviour of plate elements in compression and I-sections in bending is discussed to put stress on the effects of the different stress-strain curves of HS steels.

Target values of the mechanical properties of HS steel with low YR are established in the relations of yield stress-YR-uniform elongation.

Material guide to produce mechanical properties of high performance steels is proposed. HS steels with low YR can exhibit higher ultimate strength and larger ductility in the plated structures as compared with the HS steel with high YR. Further studies should be needed on the relations of plate slenderness parameters, YR and the length of yield plateau for the plated sections.

The COV values of the narrow yield stress range steel are less than half of the conventional steel. Characteristic values of the yield stress can be increased for the new steel depending on the values of the chosen probability.

Further discussions should be needed from the user's side to propose the optimum mechanical properties which are to provide the desired high performance behavior in steel structures. Advanced metallurgical technologies can respond to the requirements.

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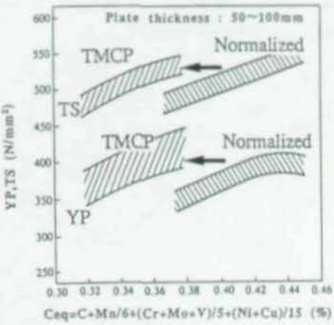


Fig.1 Strength and Ceq of TMCP and Normalized Steels of SM490 MPa Class

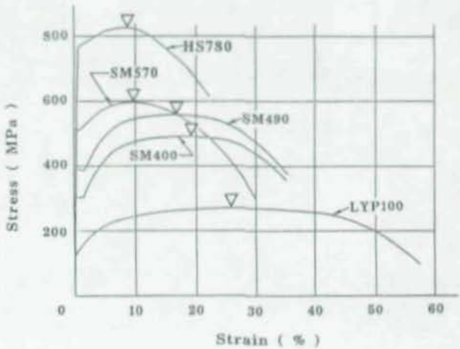


Fig.2 Comparison of Stress-Strain Curves for Various Steel Grades

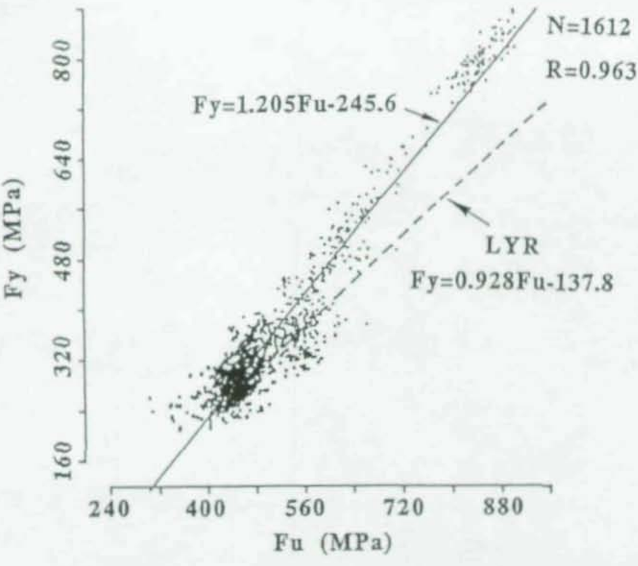


Fig.3 Yield Stress and Tensile Strength of Conventional and LYR Steels

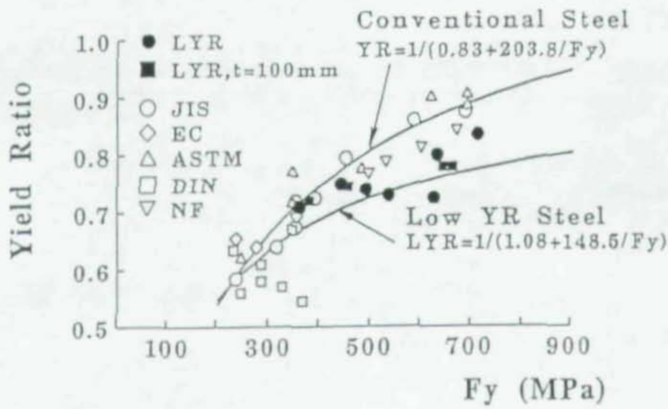


Fig.4 Fy-YR Curves and Steels in the Specifications

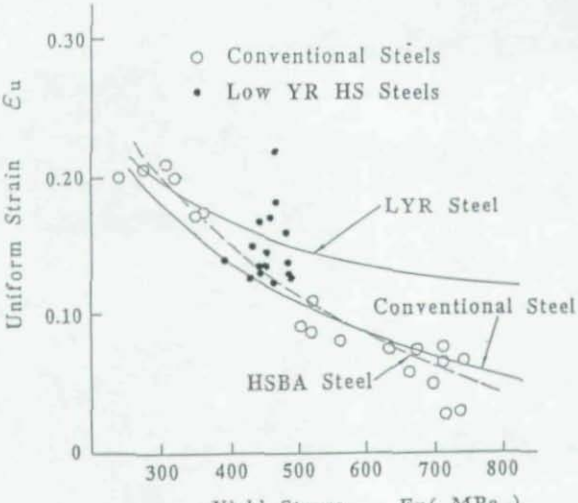


Fig.5 F_y-ε_u Relations of Conventional and Low Yield Ratio Steels

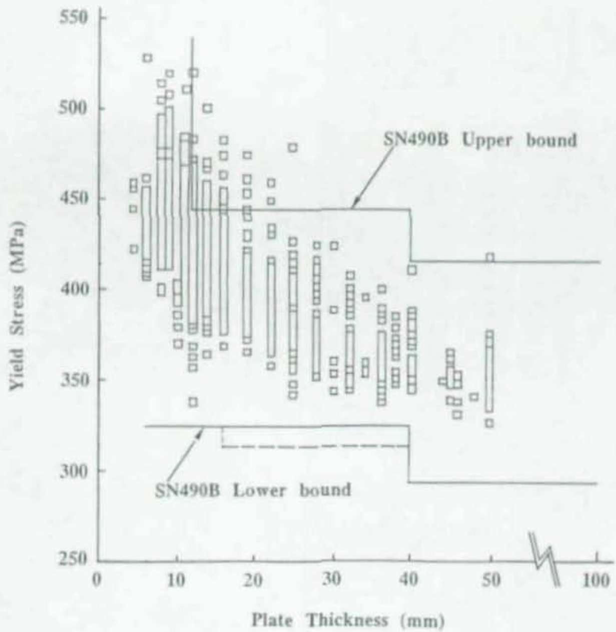
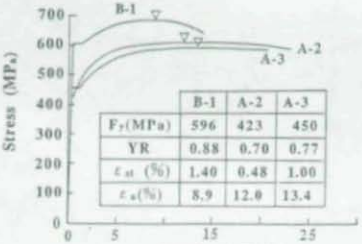
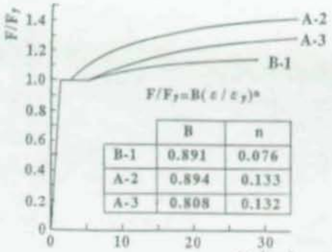


Fig.6 Yield Stress and Plate Thickness of Conventional SM490 Tests and JIS SN490B New Steel



	B-1	A-2	A-3
F_y(MPa)	596	423	450
YR	0.88	0.70	0.77
ε_{st} (%)	1.40	0.48	1.00
ε_u(%)	8.9	12.0	13.4

Fig.7(a) Stress-Strain Curves of Conventional B-1 and LYR HS Steels A-2, A-3



$F/F_y=B(\varepsilon/\varepsilon_y)^n$

	B	n
B-1	0.891	0.076
A-2	0.894	0.133
A-3	0.808	0.132

Fig.7(b) Idealized Stress-Strain Curves used in the Analysis

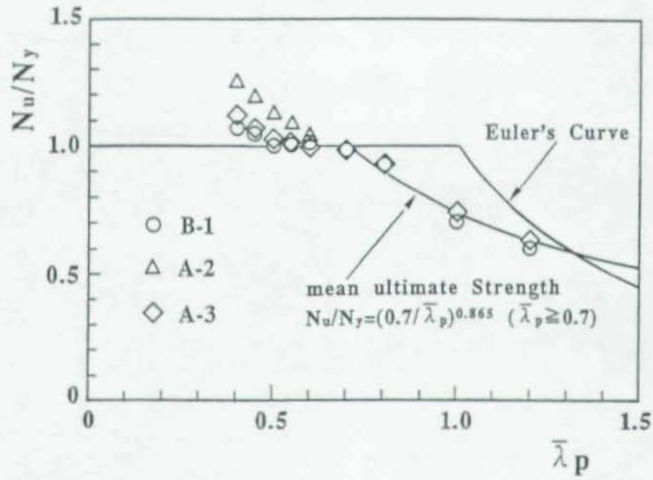


Fig.8 Ultimate Plate Strength in Compression

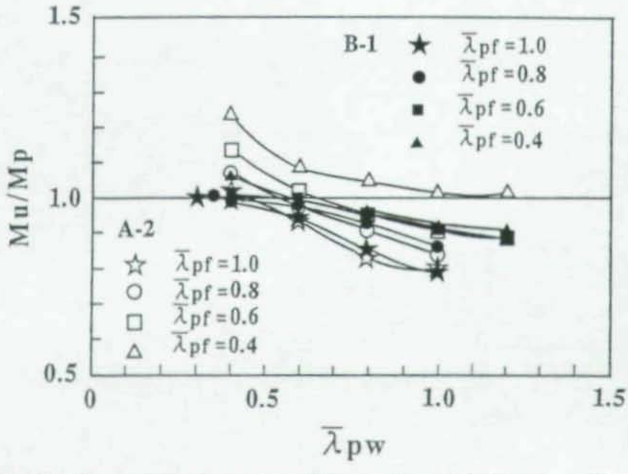
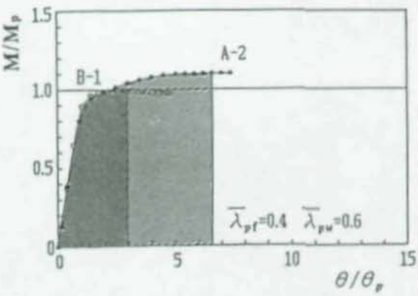
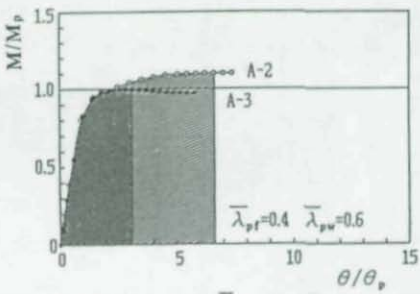


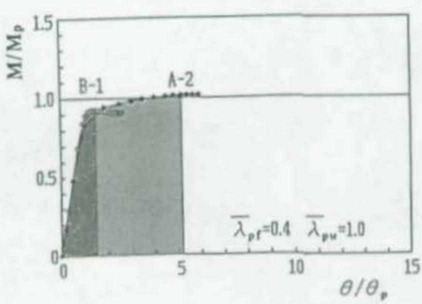
Fig.9 Ultimate Bending Strength of I-Section Beams



(a) B-1 VS. A-2, $\overline{\lambda}_{pf}=0.4$, $\overline{\lambda}_{pw}=0.6$



(b) A-2 VS. A-3 $\overline{\lambda}_{pf}=0.4$, $\overline{\lambda}_{pw}=0.6$



(c) B-1 VS. A-2, $\overline{\lambda}_{pf}=0.4$, $\overline{\lambda}_{pw}=1.0$

Fig.10 M-θ Curves for Various Flange and Web Slenderness Ratios

INVESTIGATION OF THE STABILITY OF IMPERFECT CYLINDERS USING STRUCTURAL MODELS

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INTRODUCTION

Design of shell structure subjected to loading conditions that result in buckling relies heavily upon the application of specifications and design criteria. Design rules for commercial applications are given by the American Water Work Association (AWWA) (Ref. 1), the American Petroleum Institute (API) (Ref. 2), or the American Society of Mechanical Engineers (ASME) (Ref. 3),and for aerospace structures by NASA Space Vehicle Design Criteria for buckling given in SP8007 (Ref. 4), SP8019, (Ref. 5), and SP8032 (Ref. 6). The finite-element method is also used extensively to predict shell buckling. The finite-element method has become the predominant design and analysis tool in the aerospace industry, where very large finite-element models are built with the aid of preprocessors and solved by large general-purpose finite-element codes. Visual display of prebuckle deformation and buckle mode shapes is made easy with postprocessing software.

Some complex design situations may not be adequately covered by currently available specifications and design criteria. In this case, reliance must be placed on analytical methods such as the finite-element approach. However, as the size of these analytical models grows, and the complexity of the structure increases, the possibility of generating flawed results also grows. Sole reliance on analytical predictions for complex structure may not be desirable or prudent.

An alternate approach is to supplement analytical methods and design specifications with test results from structural scale models. Data from scaled test models provide both design data and a means of verifying analytical predictions through correlation. This study illustrates the use of scale models to investigate the buckling of shells with initial imperfections. Results of experiments on cylindrical shells with initial imperfections subjected to buckling under axial compression and external hydrostatic pressure are presented and correlated to finite-element code predictions.

Imperfections are always present because of manufacturing tolerances. The presence of initial imperfections can greatly reduce the actual buckling load compared to that predicted for a shell of perfect geometry. The present effort is part of a combined analytical and experimental program to investigate the effect of manufacturing-induced imperfections on shell buckling loads. Currently, there is no method of relating manufacturing-induced imperfections to shell buckling correlation (knockdown) factors. Such a method would be useful in defining the most efficient fabrication method and least expensive tooling, and would provide acceptance criteria based on the buckling failure mode, thereby providing guidance to engineering for evaluating discrepant hardware.

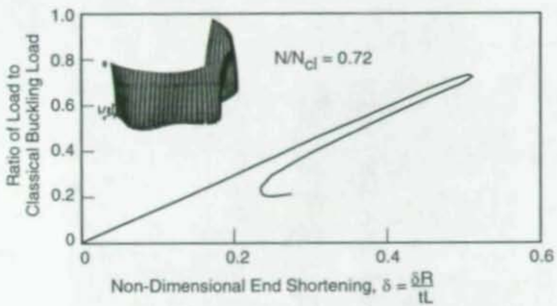
BACKGROUND

Unlike plate structures, curved shells have little postbuckling capability. Therefore, buckling of the shell results in catastrophic failure. Consequently, worst-case loads are used with design criteria that do not permit buckling. Figure 1a illustrates the axial compression buckling and postbuckling behavior for a shell with an imperfection amplitude that is 30% of the shell thickness and has the same shape as the classical buckled shape. The buckling load is predicted to be 72% of the classical theoretical value; the postbuckling load is only 22% of the classical value. As can be seen in Figure 1.b, small imperfections lead to large reductions in the buckling load for cylinders under axial compression. The results from Budiansky (Ref. 7), the BOSOR 4 code (Ref. 8) and from Koiter (Ref. 9) are for the axisymmetric imperfection; those from Yamaki (Ref. 10) are for an asymmetric imperfection. For axial compression, the axisymmetric imperfection is the worst shape. Current design practice (Figure 1.c) uses experimentally determined correlation factors. The correlation factor for axial compressive loading is related to the R/t ratio. A more useful parameter would be the imperfection amplitude-to-thickness ratio (w_o/t), as shown in Figure 1.d . Here the theoretical results from Yamaki (Ref. 10), based on an asymmetric imperfection, are used to relate correlation factors corresponding to tolerances on shell geometry permitted by the aerospace industry and those permitted by the ASME Boiler and Pressure Vessel Code. A significant difference in the correlation factors exists for these specifications. For an $R/t = 250$, the correlation factor for aerospace tolerances is approximately twice that *corresponding to the ASME boiler code. Since the required shell thickness is* approximately a function of the square root of the correlation factor, a shell designed to the ASME boiler code tolerance will be 40% thicker (and heavier) than a shell designed to the aerospace tolerance.

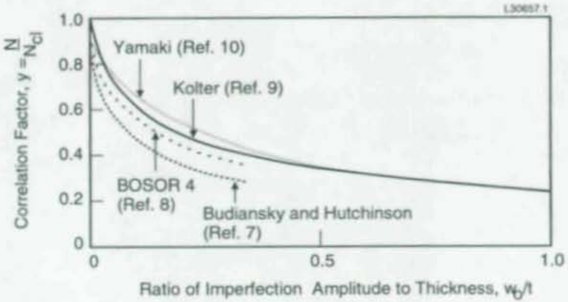
The objective of this study is to develop a methodology for predicting the buckling of shells with initial imperfections. The long-term goal is to develop design criteria through analyses and tests on scale models built with imperfections of known shape and amplitude.

PROGRAM PLAN AND TEST MODEL FABRICATION

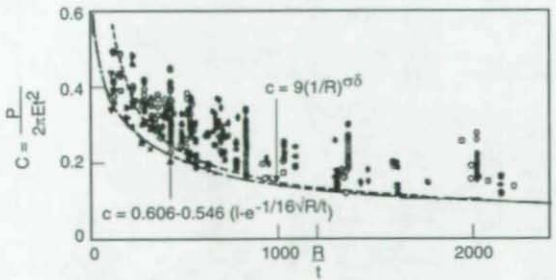
Figure 2 gives the flow diagram of the program plan for this study. Seamless cylindrical plastic models 15 inches in length and diameter with nominal wall thicknesses of 0.05 inches were fabricated from blow-molded bottle blanks. The thickness and shape of the cylinders were measured and the modulus of elasticity determined by compression test on the cylinder. The "perfect" cylinder was then tested under axial compression, external hydrostatic pressure, and finally under combined



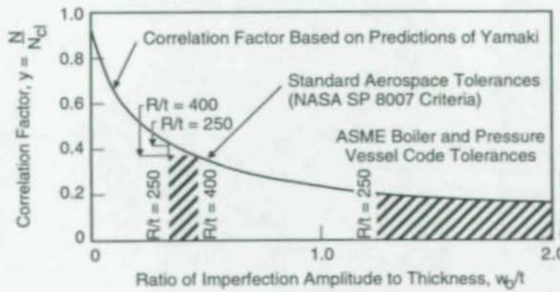
a. Curved elements have little post buckling load capability



b. Small imperfections lead to a large reduction in theoretical buckling loads



c. Current design practice uses experimentally determined correlation factors



d. Need to relate correlation factors to manufacturing practice

FIGURE 1. Background

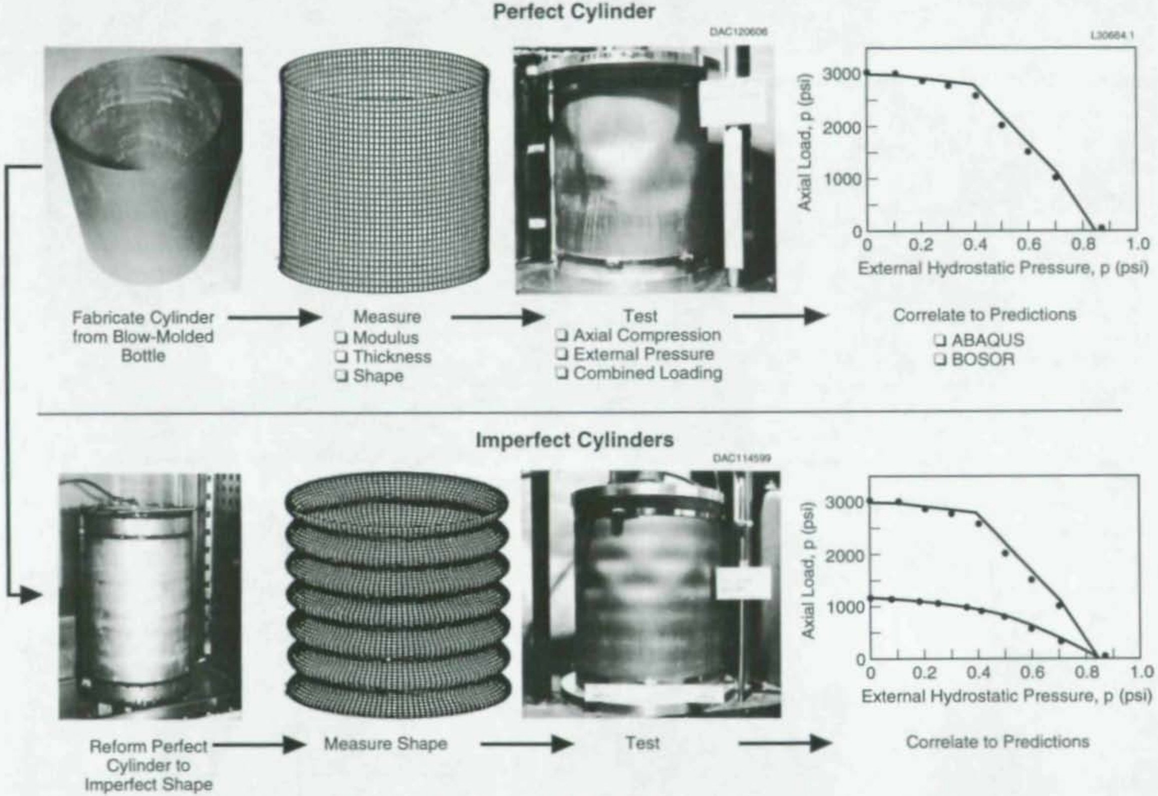


FIGURE 2. Program Plan

loading. Data for the "perfect" cylinder were correlated to predictions from the BOSOR 4 (Ref. 8) and ABAQUS (Ref. 11) computer codes.

Upon completion of this test series, the cylinder was re-formed on a mandrel with the desired imperfection shape and amplitude. Only the shape and amplitude were measured since the thickness and modulus were previously determined. The same buckling test series was repeated and the results correlated to predictions. From this combined analytical and experimental program we established (1) a comparison between experiments for perfect and imperfect cylinders, (2) a correlation between experiments and analytical predictions, and (3) a comparison between analytical predictions.

Test cylinders were constructed of Lexan. Lexan (a General Electric brand of polycarbonate plastic) was selected as the model material because prior experience with Lexan buckling models(Ref. 12) has demonstrated that repeatable buckling loads can be obtained if deformations are restricted in the postbuckled state to prevent yielding of the material. The yield stress of Lexan is high in relation to its modulus as compared with other model materials.

The cylinders were thermal vacuum formed on collapsible mandrels (see Figure 3). The mandrel surface was machined to conform to a "perfect" cylindrical shape or an imperfection shape. One mandrel was required for each shape. The "perfect" cylindrical and axisymmetric imperfection shapes were produced using a lathe operation. The asymmetric shape in the form of the buckle pattern for the "perfect" cylinder was machined into the mandrel using a six degree-of-freedom (DOF) milling machine. The eigenvector from the BOSOR 4 code was translated into the unigraphics drawing system and then to coding for the milling operation.

Blanks for the cylinders were produced using blow molding, a technique commonly used to form plastic bottles for soft drinks and water. The technique requires an aluminum female mold of the correct internal dimensions and involves injecting molten Lexan into the mold using air pressure to hold the material in place until it solidifies. The mold dimensions were oversized to allow for material shrinkage when cooled. The resulting part was a thin-walled (0.070 to 0.100 inches) bottle shape. To produce "perfect" cylinders of uniform wall thickness, the bottle blanks were first cut to remove the top and base closures. The cylindrical blanks were then mounted on a collapsible "perfect" mandrel. This mandrel had a constant diameter and was used to anneal the Lexan in a thermal vacuum process. The yield stress of the Lexan at elevated temperature was exceeded while forcing the material to conform to the mandrel with an internal vacuum pressure. After cooling, with the Lexan in intimate contact with the mandrel, the skin was lathe machined to a wall thickness of .050 ± .001 inches. A second thermal vacuum process was necessary to remove residual stresses after the machining operation. Returning to room temperature, the Lexan cylinder shrank tightly to the mandrel. A "key" block was removed from the mandrel which allowed the mandrel to collapse and release the Lexan.

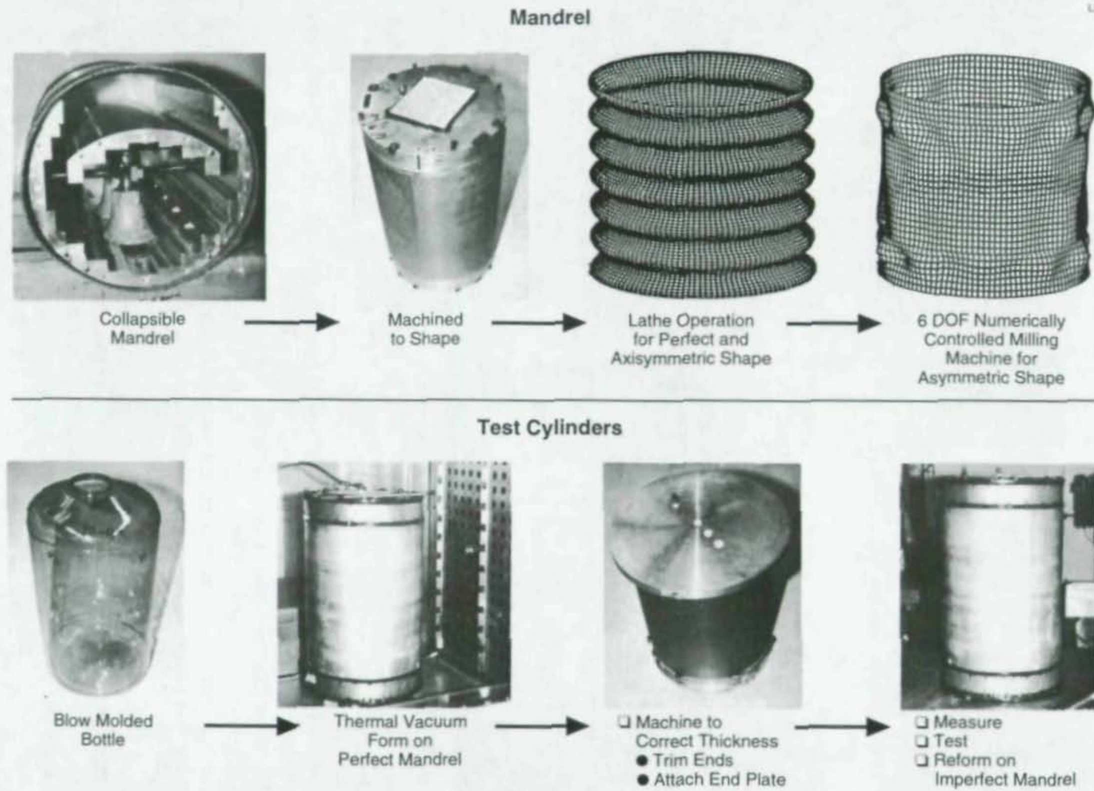


FIGURE 3. Fabrication

After testing and verifying that the cylinders were near perfect, the cylinders were re-formed on imperfect mandrels using the technique just described. The imperfection amplitude was selected as 30% ($w_o/t = 0.3$) of the cylinder thickness because this amplitude, along with worst-case imperfection spatial distributions, produces knock-down factors for axial compression, external hydrostatic pressure, and shear buckling that are near those recommended by NASA SP8007 (Ref. 4). Pretest analysis of the test cylinders with axisymmetric imperfections showed that the lowest buckling load occurred when the imperfection wave length corresponded to 13 axial half waves. At this wave length, a 30% imperfection still results in snap-through buckling, but a larger imperfection will result in axisymmetric collapse. The cylinder was predicted to buckle into 10 circumferential waves and this buckled shape was observed in the experiment.

EXPERIMENT DESCRIPTION

After fabrication of a cylinder was completed, the cylinder thickness was measured at 18 axial locations every 45 degrees for a total of 144 locations. Thickness measurements were made to determine the average thickness to be used in the analytical predictions and also to establish that the maximum-to-minimum thickness difference was less than 0.003 inches. The largest thickness difference determined was 0.0028 inches for cylinder S/N97 and the smallest was 0.0019 inches for cylinder S/N 81.

After the thickness of a cylinder was determined and found acceptable, the cylinder was then fitted with aluminum load heads at each end. A steel strap clamp band held the Lexan cylinder to each load head and restricted all three in-plane displacements as well as the meridional rotation, resulting in clamped-boundary conditions.

The modulus of elasticity was determined from load versus end-shortening data taken for each cylinder. Edge effects at the cylinder ends due to clamped boundaries resulted in a minor inaccuracy, given a 0.4% lower modulus. The modulus data was quite repeatable, having a 0.3% variation over three tests.

In addition to thickness and modulus, shape data were needed to perform analytical predictions. Shape measurements were taken to characterize the imperfections and to verify the quality of the "perfect" cylinders. Figure 4 shows two methods that were used to measure the shape of the cylinders. The Zeiss Measurement System was initially used because of its availability and because the data could be easily converted to computer code model mesh data, or displayed. However, this system is restricted to small shapes (not much larger than the test cylinders), and consequently could not be used for typical full scale hardware. Therefore, a scanning measurement system was developed that, while only used in the model laboratory, would be adaptable to full scale hardware. Both systems produce the same quality and quantity of data at about the same costs.

Cylinder coordinates (R, θ, Z) were measured at 9600 locations. Data were taken every 0.188 inches down the length of the cylinders and every 3 degrees around the circumference. The data taken by either measurement system were processed by the data

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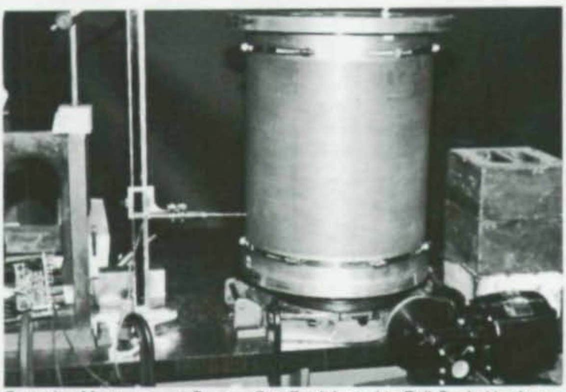
Zeiss Measurement System Is Limited to Small Shells



Scanning Measurement System Can Be Adapted to Full-Scale Hardware

FIGURE 4. Shape Measurements Taken on Forming Mandrels and Shells

reduction code. From the formatted data file, the shape can be displayed in either a cylindrical coordinate system, or as a flat-pattern development; grid point model coordinates (R, θ, Z) can be created for a finite-element mesh. Figure 5 shows the flow diagram of the process for displaying and translating measured data into the structural finite-element model. PATRAN (Ref. 13) is a pre and postprocessor code that can be used with many general purpose finite-element codes.

The shapes of the three tested cylinders are shown in Figure 6. These shapes are displayed as the deviation of the actual shape from an imaginary perfect cylinder. The radial deviation is shown as displacement from the "perfect" shape divided by thickness, w_o/t.

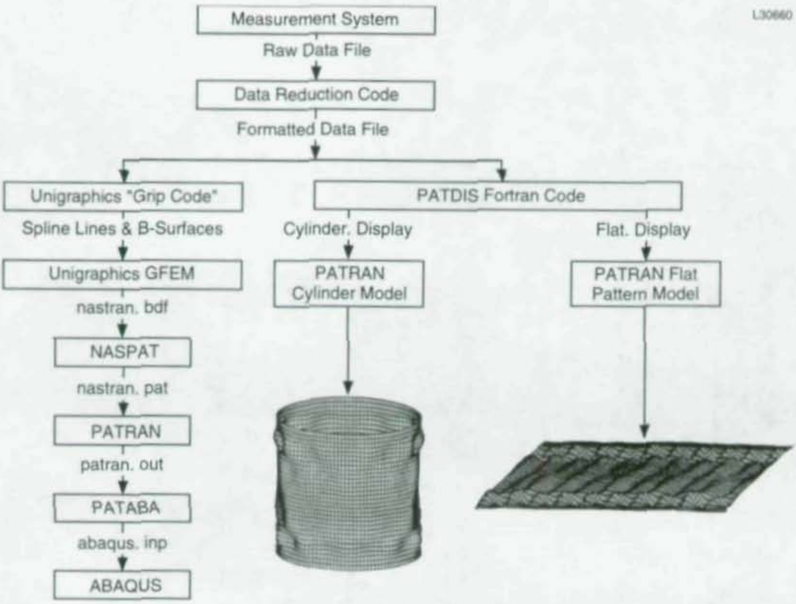


FIGURE 5. Procedure for Displaying and Translating Measurement Data into Structural Finite-Element Model

The cylinders were tested in a standard hydraulic test machine (Figure 7). A vacuum pump was used to evacuate the cylinder so as to produce external hydrostatic pressure. Load and end-shortening measurements are made; a video camera was used to record each test. Typically, several tests were conducted under the same loading conditions to verify results.

ANALYTICAL METHODS AND ANALYTICAL MODELS

The buckling load of the test models was predicted by seeking the lowest buckling eigenvalue from a nonlinear state of prebuckling stress. The presence of an initial imperfection requires the use of analysis methods that account for geometric nonlinearity. The experimental data were correlated to predictions made by the BOSOR 4 and ABAQUS computer codes.

The BOSOR 4 code performs the stress, stability, and vibration analysis of complex branched shells of revolution by the finite-difference method. The code is restricted to axisymmetric shapes and was used here to predict the buckling of "perfect" cylinders and those with axisymmetric imperfections. The buckle mode shape need not be

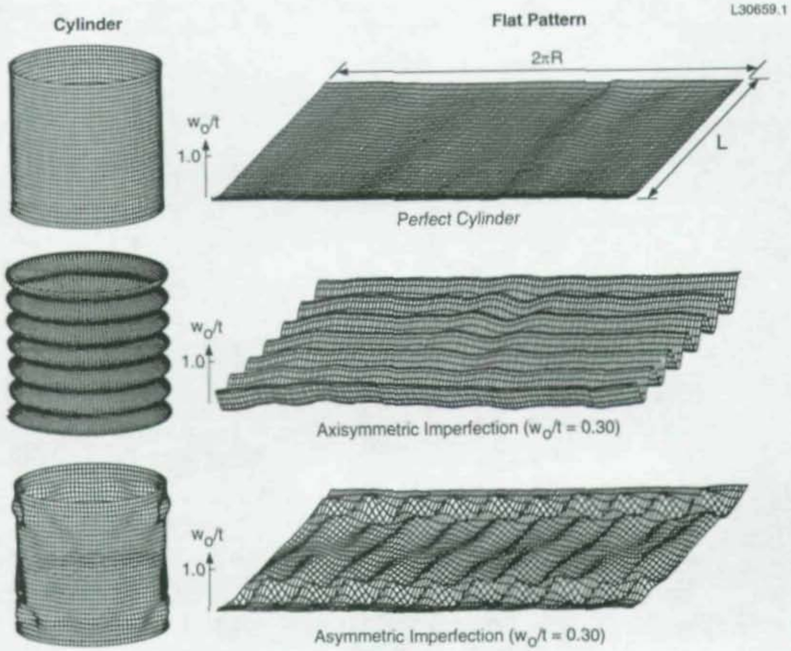


FIGURE 6. Measured Shapes

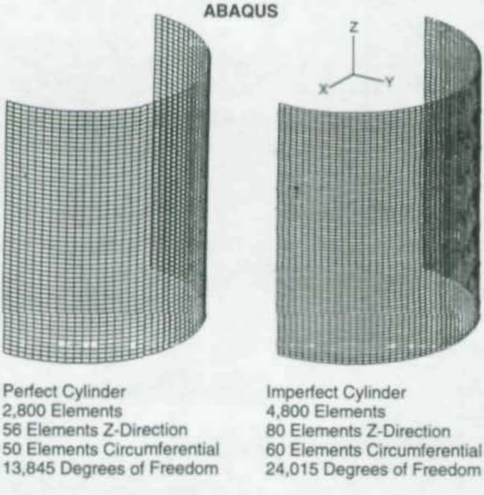
restricted to the axisymmetric mode since BOSOR 4 will search for the lowest buckling load over a range of user-supplied circumferential harmonics, or buckle-wave shapes.

ABAQUS is a large-scale, general-purpose computer code that performs the static, dynamic and heat transfer by the finite-element method. Because both the meridional and circumferential coordinates can be modeled, ABAQUS can account for both axisymmetric and asymmetric imperfections.

Figure 8 shows the analytical models used in this study and provides some model statistics. Two 180-degree symmetry models were used with the ABAQUS code; one for "perfect" cylinders and one for imperfect cylinders. Both models had symmetry conditions imposed on the unloaded edges and clamped (all DOF fixed) conditions on the loaded edges representative of the test conditions. The full length of the cylinder was modeled so as to admit both symmetric and nonsymmetric axial-mode shapes.

The mesh spacing was chosen to adequately describe the axial and circumferential buckle-wave lengths that were predicted. The mesh spacing in the axial direction can

FIGURE 7. Test Setup



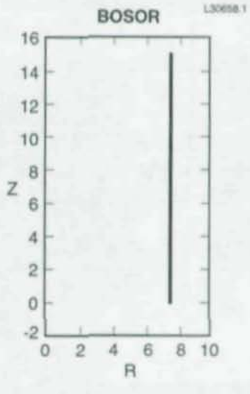


FIGURE 8. Analytical Models

be based on the half-wave length, l_x, for the axisymmetric mode determined by

$$l_x = \frac{\pi}{\sqrt{2}\sqrt[4]{3(1-\mu^2)}}\sqrt{Rt}$$

where R is the radius of curvature, t is cylinder thickness, and μ is Poisson's ratio. The circumferential mesh is determined by the requirement to describe the circumferential half-wave length. The circumferential mesh spacing should not be much larger than 5 degrees of arc so as to adequately model the curved surface with flat-plate elements. Additionally, the aspect ratio of the flat elements should not exceed 4.

A finer mesh is required for the imperfect cylinder model because, in addition to meeting the above criteria, the mesh must adequately model the imperfection shape mapped on the mesh by the coordinate measuring system. The larger imperfect shape model with 24,015 active DOF is considerably more computationally expensive than the smaller "perfect" shell model with 13,845 DOF.

The BOSOR 4 mesh used 400 finite-difference grid points to model the generator of the cylinder. Options in BOSOR 4 permit a description of the wave length and amplitude of an axisymmetric imperfection. The user supplies circumferential wave numbers so buckling loads for asymmetric modes can be computed.

CORRELATION OF PREDICTIONS AND EXPERIMENT

Excellent agreement between theory and experiment for axial compression has been demonstrated. Table 1 gives analytical and experimental results for "perfect" and imperfect cylinders under axial compression loading. Maximum deviation of theory and experiment was 4.4%; most predictions correlated to within 1% or less. The axisymmetric imperfection results in the greatest reduction in buckling load as predicted. Referring to Table 1, the buckling load resulting from an axisymmetric

TABLE 1.—Excellent Agreement Between Theory and Experiment Has Been Demonstrated For Axial Compression

Axial compression			
		Theory (lb) (% correlation)	
Test specimen description	Experimental results (lb)	BOSOR 4	ABAQUS
Perfect cylinders			
S/N 81	3059	3046 (0.4)	3085 (0.8)
S/N 97	3015	3010 (0.2)	3006 (0.3)
S/N 78	3045	2960 (2.8)	2936 (3.6)
Imperfect cylinders			
Axisymmetric imperfection (w_o/t = 0.30 S/N 97)	1183	1181 (0.2)	1235 (4.4)
Asymmetric imperfection (w_o/t = 0.30, S/N 78)	1656	Not applicable	1658 (0.1)

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imperfection is (1183/3015)(100) = 39.2% that of the perfect cylinder based on a comparison of experimental data and is predicted to be 39.2% by BOSOR 4 and 41.1% by ABAQUS. Similar results for the asymmetric imperfections are 54.4% by experiment and 54.2% by ABAQUS prediction.

Excellent agreement between theory and experiment for external hydrostatic pressure has been demonstrated (see Table 2). Maximum discrepancy between theory and experiment was 2.5%. The imperfection shapes studied so far are the same as those for the axially compressed cylinder. These imperfection shapes do not significantly reduce the buckling load under hydrostatic pressure. Experimental results for hydrostatic pressure show that the axisymmetric imperfection reduces the buckling pressure of the "perfect" cylinder by 1.5%. By comparison, BOSOR 4 predicts a 1% increase and ABAQUS a 0.2% decrease. The asymmetric imperfection results in a reduction of 6.4% in buckling pressure based on experimental data.

TABLE 2.—Excellent Agreement Between Theory and Experiment Has Been Demonstrated for External Hydrostatic Pressure

Hydrostatic pressure			
		Theory (psi) (% correlation)	
Test specimen description	Experimental results (psi)	BOSOR 4	ABAQUS
Perfect cylinders			
S/N 81	0.870	0.851 (2.2)	0.880 (1.1)
S/N 97	No experiment	0.834	0.863
Imperfect cylinders			
Axisymmetric imperfection (w_o/t = 0.30, S/N 97)	0.857	0.860 (0.40)	0.878 (2.5)
Asymmetric imperfection (w_o/t = 0.30, S/N 78)	0.816	Not applicable	Not calculated

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Correlation of combined axial compression and external hydrostatic pressure test data is shown in Figure 9. The interaction data has been correlated with BOSOR 4 predictions for both the "perfect" cylinder and the cylinder with an axisymmetric imperfection. BOSOR 4 is not applicable to the asymmetric imperfection case and ABAQUS predictions have not been completed.

The interaction curve for the "perfect" cylinder has two branches. The one for a circumferential wave number of N = 10 is dominated by axial compression loading and the corresponding compression buckle mode shape. The branch for N = 6 is dominated by external pressure loading.

The interaction curve for the axisymmetric imperfection is composed of three branches. The N = 10 and N = 9 branches correspond to compression-dominated behavior while the N = 7 branch corresponds to the external-pressure-dominated region. A comparison of the results for the "perfect" and imperfect cylinder again illustrates

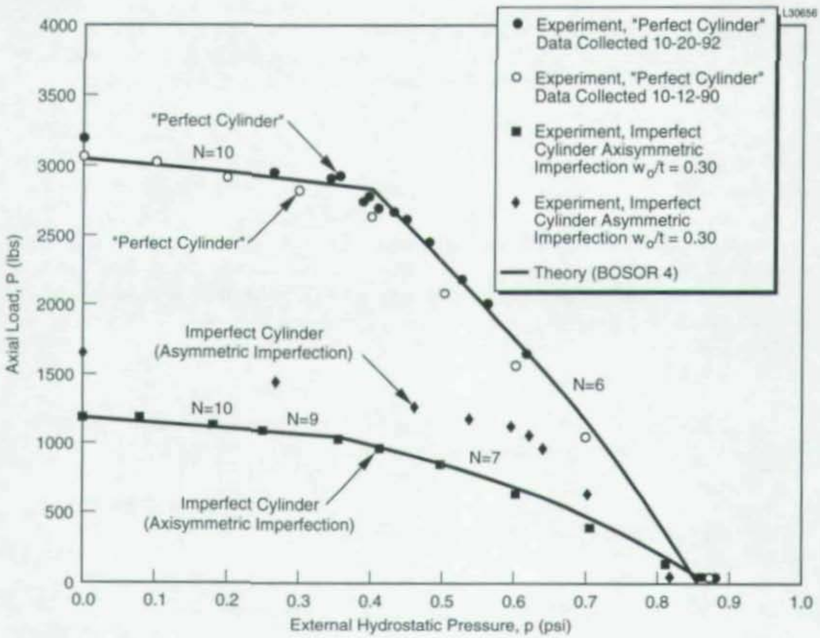

FIGURE 9. Correlation of Combined Axial Compression and External Hydrostatic Pressure Test Data

the dramatic reduction in buckling load for compression loading for an imperfection amplitude just 30% of the cylinder thickness.

SUMMARY AND CONCLUSIONS

This study on the effect of geometric imperfections on shell buckling loads illustrates how scale models can be used to provide design data and verify analyses through correlation studies. The ability of the test models to be reconfigured and subjected to multiple loading conditions provided a comparison between experiments for "perfect" and imperfect cylinders using the same model. The test data were correlated to BOSOR 4 and ABAQUS computer codes, and excellent agreement (within 4.4%) was achieved.

Small amplitude axisymmetric imperfections of 30% of the shell thickness can result in a 60% reduction in buckling load. This illustrates how important imperfection shape and amplitude are in reducing the compressive load-carrying capability of cylinders, and how important it is to minimize the effect of imperfections by controlling the fabrication and assembly process.

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DEVELOPMENT OF ALUMINUM STRUCTURAL TECHNOLOGY IN THE UNITED STATES

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Introduction

We don't need to look back too many years to chart the development of structural design technology, because aluminum has been produced commercially only a little over a hundred years, and most structural research took place after 1930(Reference1). Commercialization occurred about the same time in Europe and in the United States and structural technology developed in both areas of the world. This paper will deal with the development of technology primarily in the United States. Table 1 shows that only a few companies were producing aluminum 50 years ago. Only one company existed in the U.S. for the first 50 years after commercial production started. Now over 40 aluminum companies are in the U.S. and they account for about 20% of the total work force employed in all of the metal industry. The earliest problems that faced the industry were those to identify the applications for the new material being produced. The quest for new applications continues to this time. So far the primary uses have been in the replacement of incumbent materials in existing applications. The world's aluminum industry has had a rapid growth in production and in use and continues to grow currently as illustrated in Figure 1.

The Material

Some representative aluminum alloys normally used for general structures and their properties are given in Tables 2 and 3. Their strengths are generally somewhat lower than those for common steels. Alloys used in aerospace applications have tensile and yield strengths over twice those shown in Table 2. Aluminum alloys have about 1/3 of the density and the modulus of steel. They have excellent corrosion resistance, and thermal and electrical conductivity. One or more of these attributes has been important in the markets developed for aluminum. There were few structural uses for aluminum immediately after commercial production started. The Wright brothers used a few aluminum parts in their first plane in 1903. The earliest significant structural use was for framing and skins for a dirigible called the *Shenendoah* completed in 1923 for the Navy of an alloy 17S. This alloy was developed in the US with much help from technology obtained by analyzing German alloys, advertised as being as strong as steel. Although there were other structural applications, there were no structural guidelines established until about 1930. The first design rules were developed by scientists in a new department established at the Aluminum Laboratories in 1929, the Engineering Design Division. Ernest C. Hartman was hired to head up the group.

Corrosion of some of the early alloys was a problem, and considerable effort by the scientists was devoted to developing methods for measuring the relative performance of the alloys. The work on corrosion actually preceded that for structural design. Some of the earliest structural applications made use of those alloys with poor corrosion resistance. The battle deck floor and floor beams, shown in Figure 2 were made of an alloy 27S, specifically developed for this bridge application, but poor in corrosion resistance. The parts were installed on the Smithfield Street

Bridge, Pittsburgh, Pa. in 1933. The members were joined with steel rivets, the structure was painted but the faying surfaces were left bare, because of a requirement that the structure conduct electricity for the street cars, a feature that was not used. The deck system was replaced with an unpainted aluminum orthotropic deck in 1967, and the floor beams and deck will be replaced in 1994. The floor beams have been repaired several times over the years because of degradation of the joints due to corrosion. The current deck needs a new wear surface but otherwise is in excellent condition.

The railroad bridge over the Grass River in Massena, N.Y. had aluminum riveted girders in one span(see Figure 3). The alloy selected was 2014-T6, an aircraft type alloy but somewhat better in corrosion than 27S. Aluminum rivets, protected faying surfaces and painting have prevented deterioration due to corrosion of these girders since their erection in 1944. There were more corrosion resistant alloys available(6061 was developed in 1934) which could have been used without painting, but the designers apparently wanted the higher strength of the 2014. Several other aluminum bridges have been built using modern alloys and riveting or welding.

By 1952 technology on alloys, finishing and structures was sufficiently developed so that aluminum panels were installed as the exterior face of the Alcoa Building in Pittsburgh. Figure 4 is a photo of the installation of one of the panels. The panels have functioned well to the present without the need for maintenance or cleaning(2).

Product Forms

The first product forms made were castings. The cap installed on the Washington Monument in Washington D. C. in 1884 was a cast aluminum pyramid. Flat rolled products also developed early, and now are available for thin household foil to thick plate(say 6 to 8in.). Most of the earliest shapes for aluminum were either formed from sheet and plate, or rolled, which made use of the processes available to make steel shapes at that time. The extrusion process was developed during the 1920's and allowed the engineer to design unique, intricate shapes that saved both material and fabrication costs. Current technology for extrusions accommodates sizes up to about 30 in., although the more practical size is that which will fit in a 15-in. circle size. Forgings, now used for aircraft parts and truck wheels complete the list of product forms developed over the years and are now available to the designer.

The People

Some mention should be made to the people that were leaders in developing structural technology useful for general aluminum structures. Figure 5 shows some of them. Ernest C. Hartman had the initial responsibility for developing structural technology at the newly created Aluminum Research Laboratories in New Kensington, Pa. He also was the author of the first Alcoa Structural Handbook in 1930, the author of numerous technical papers, and contributor to most of the early structural specifications. Harry N. Hill was assistant chief of the Engineering Design Division and performed experimental and theoretical investigations of lateral buckling of various types of beams, beam columns and welded construction. Raymond N. Moore contributed in several areas including torsional behavior of stiffened and unstiffened cylinders and fasteners. Marshall Holt was most active in pressure vessel design, and also contributed to column buckling research. These first four people were part of the initial contingent of the Engineering Design Division and were the leaders of the early research activity. John W. Clark joined the division about 17 years later with the responsibility for general structural research and structural design code development. He set the direction for the format of the current aluminum specifications.

Design Science for Aluminum Structures

Table 4 presents some pertinent information. Some of the earliest work affecting structural technology came from Europe(alloy development), governmental agencies(use in dirigibles/aircraft) and the steel industry(general structural behavior). Some influence of these sources continues to this time. Because most of the researchers were "trained" in steel technology, much of the aluminum technology development was similar to that needed for the traditional steel products.

Industry Research-The research conducted soon after the establishment of the Engineering Design Division in 1929, was focused to support the development of the aerospace industry. This industry needed the low density, high strength material. Many of the earliest reports of research by the aluminum industry were issued as governmental agency reports. Structural research by the aluminum industry on aerospace type problems, notably stability and fatigue, continued to the early 1960's, and then was discontinued, as the aerospace industry matured in aluminum technology. Subsequent research was more related to buildings, infrastructure and transportation. Examples of some of the programs that have contributed to current design guidelines are provided below.

Many of the early research programs were related to structural stability. As shown in Figure 6 aluminum has a stress-strain curve that gradually bends over and thus does not have the yield plateau, characteristic of mild steel. In addition, different alloys had different shapes of the stress-strain curves. A yield strength defined to be equal to a 0.2% offset strain was adopted. As a result of numerous, carefully made tests of several shapes of columns, a straight line was defined for the inelastic portion of the column buckling curve(see Figure 7)(3,4). The straight line closely approximates the tangent modulus curve. The slope of the line depends on the shape of the stress-strain curve. The straight-line representation for the inelastic buckling of columns was found subsequently to be useful for most all buckling cases; beams, plates and cylinders(5,6).

Figure 8 shows failures of various lengths of tubes tested under torsion, an investigation representative of those early research programs(7). The early guidelines for design had a great deal of experimental data to substantiate the proposed theoretical approach. Other areas that received much attention were lateral buckling of beams and the behavior of beam columns. The results were generally published(8,9).

The initial design guidelines for aluminum structures did not take into account the increased strength of buckled plate elements above the buckling value. The aircraft design rules did include ultimate strength provisions for thin members. Research in the 1960's on building sheathing products provided for postbuckling behavior of thin aluminum elements(10). This work was incorporated into a unified treatment for the strength of aluminum elements that included "thick" construction corresponding to that for steel covered by the AISC Specifications to that for "thin" construction covered by the AISI Specifications. The format developed is illustrated in Figure 9: there is a cut-off based on yielding at slenderness ratios less than S1, a straight line inelastic buckling between S1and S2, and buckling or post buckling strength at slenderness ratios greater than S2. The format is used for both member and element strength.

Compression tests of sections with stiffening lips as shown in Figure 10 and analysis provided the basis for stiffener design(11) based on buckling strength. Several investigations were directed toward concentrated loads on thin webs(12,13,14). Large deformation, plastic deformation finite element analysis was utilized in recent papers for parameter studies to extend the information available from tests. The model for one case is given in Figure 11. An interaction equation was defined for combined concentrated loads on thin sections and member bending, with the end points being pure bending(no concentrated loads) and concentrated loads with no bending(see Figure 11).

All of the general purpose alloys can be joined by welding, mechanical fastening and adhesive bonding. Structural design of welded construction, however, is somewhat different for aluminum than steel because most of the alloys and tempers employed have lower strength at and near the weld compared to base metal properties. This required the development of design procedures based on the welded strength(15).

University Research-Iowa State University was active in summarizing and interpreting the behavior of aluminum weldments in fatigue(16,17). A data base was developed from information obtained over the world, and now is shared in Europe and the U.S(18). The data base includes much of the information on fatigue generated by the aluminum industry. This work provided the basis for the first specifications for fatigue design of aluminum structures published in 1986(19).

Cornell University has completed two programs that extended structural design information on two buckling problems(20,21). The first program consisted of compression tests and analysis of columns that failed in flexural-torsional buckling. Four open shapes, three alloys and several lengths of columns were investigated. The second program was a comprehensive analysis and test program to refine and improve the design guidelines for thin sections primarily under bending, including effects of intermediate and flange stiffeners. Two alloys, 5052 and 6061, were used in the various specimens employed in the investigation. The results of these investigations are incorporated in the new Aluminum Association Specifications to be issued in 1994.

"Fatigue of Welded Structures" was the topic of a program conducted at Lehigh University(22). Large welded girders of alloy 5456, with stiffeners and cover plates were tested. These tests and other beam tests in Europe provide the basis for improved fatigue design provisions, scheduled for the 1994 specifications.

Research at Washington University provided a detailed review of the design specifications for aluminum structures(23) and tentative provisions for LRFD for building structures. This work is the basis for the LRFD specifications scheduled to be published in 1994.

Structural Design Technology

In order to implement the results of the various research programs, the information has to be incorporated into handbooks and specifications that designers use. Table 5 provides a summary of this implementation for aluminum structural design. The first structural design handbook by industry was issued in 1930 by Alcoa. Many editions followed as new information became available. Other aluminum producers developed similar handbooks as they began to market their products. These company produced books were the primary sources of design information to the mid 1960's. At that time the individual companies discontinued the publishing of these handbooks and the Aluminum Association assumed responsibility for the dissemination of design information.

There were no text books on aluminum structural design written until the mid 1980's. Two were published within two years of each other(24,25), one published in 1993(2) and one is scheduled to be published in 1995(26).

Specifications are important for any material, particularly those used in buildings and infrastructure. *Specifications for Aluminum Alloy Structures* was first published by the industry in 1932. Leon Moisseiff, with help from other bridge design experts developed an aluminum specification for bridges and structures in 1940(27), also published by the industry. Other specifications followed, written by the members of the Task Committee on Lightweight Alloys, Structural Division, American Society of Civil Engineers, and published in the Journal of the Structural Division. The Aluminum Association assumed responsibility for structural specifications and published its first *Specifications for Aluminum Structures* in 1967. Several editions followed with some minor revisions(with the exception of fatigue provisions in 1986). A major upgrading of the specification is currently being finished, and the new specification is scheduled to be published in 1994.

Two other specifications are available for the design of aluminum bridges. A guide specification was issued by AASHTO in 1990, and LRFD provisions are to be available in 1994(28,29).

Future Structural Technology

The needs will depend on the structures that will utilize aluminum in the future. Two areas of major growth look to be possible.

Increased use of aluminum in infrastructure looks possible. The deck system shown in Figure 12 is light weight and doesn't require painting. The concept has performed well on the Smithfield Street Bridge in Pittsburgh for over 25 years and will be used on a bridge in Virginia this year. The new look at aluminum for infrastructure is of interest because of the recent decreased cost of aluminum ingot which makes aluminum structures more competitive in life cycle costs compared to painted steel structures.

The use of aluminum in transportation and specifically the automobile has been increasing. A new concept is an aluminum space frame for an automobile. Extrusions and ductile castings are welded together in this concept as shown in Figure 13. A plant in Germany has been constructed to produce parts for a vehicle to be built by Audi. Other automobile manufacturers are exploring this and other concepts using aluminum. The incentives are ultra-lightweight safe vehicles that have excellent fuel economy and possible savings on manufacturing costs because of the use of the different product forms in the frame.

The research that is ongoing on these applications will, in the long term need to be incorporated into new design guides. The form of these guides will need to be changed to accommodate the new analytical techniques available. For example, structural simulations by computer have evolved over the past 5 to 10 years so that they are commonly used for design. It is now possible to do the large deformation, plastic deformation analyses needed to simulate complex behavior on

PC's. These new capabilities somehow need to be integrated into future structural guidelines and specifications.

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Table 1
INITIAL PRIMARY PRODUCERS IN THE US

Company	Year of First Primary Production
Aluminum Company of America	1888
Reynolds Metal Company	1941
Kaiser Aluminum and Chemical Corporation	1946

Table 2
REPRESENTATIVE ALLOYS AND PROPERTIES

		Minimum Values		
Alloy	**Common Product Form**	**Tensile,ksi**	**Yield,ksi**	**Elongation,%**
6061-T6	Sheet, Plate, Extrusions Forgings	38(extr.) 42(other)	35	10
6063-T5	Extrusions	22	16	8
3003-H16	Sheet	24	21	4
5052-H32	Sheet and Plate	31	23	9
5083-H116	Sheet and Plate	44	31	12
A356-T6	Casting	33	22	3

Table 3
REPRESENTATIVE PROPERTIES OF GENERAL PURPOSE ALLOYS

Property	Representative Value
Density	0.1 lb/in^2
Modulus of Elasticity	$10.1x10^3$ksi
Coefficient of Thermal Expansion	$13x10^6$in/in/°F
Thermal Conductivity	800-1500Btu in/ft^2.h.°F
Electrical Conductivity (%of Cu for equal weight)	90 to 200

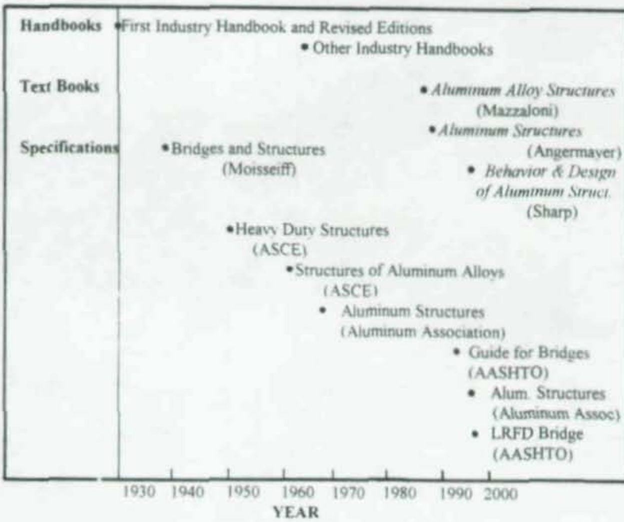
Table 4
SOURCES OF STRUCTURAL RESEARCH AND DEVELOPMENT

- Europe
- Military and Government-Aerospace
- Steel
- Aerospace Industry(aircraft design: buckling and fatigue)
- Aluminum Industry(columns, beams, tubes, thin element behavior, fatigue, welded constr., stiffeners)
- Iowa State(Fatigue)
- Washington Univ.(LRFD)
- Cornell Univ.(Flex.-tors. buckling) (Thin elem. behav.)
- Lehigh Univ.(Fatigue)

1900 1940 1980 2000

YEAR

Table 5
DEVELOPMENT OF HANDBOOKS AND SPECIFICATIONS



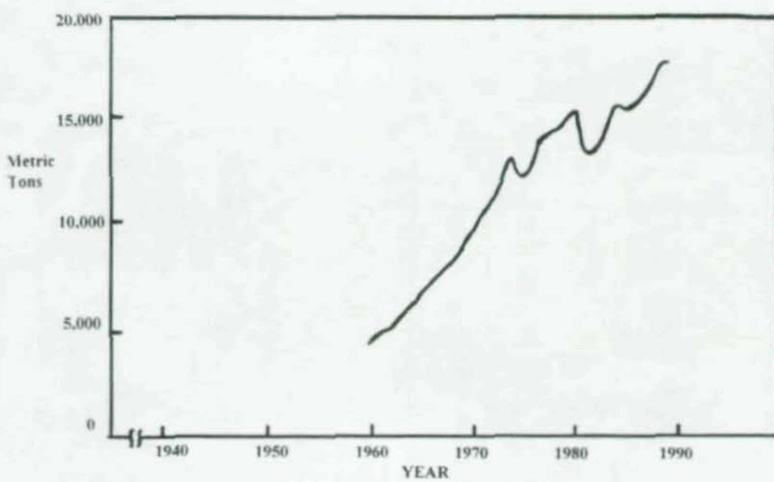


Figure 1 World Primary Aluminum Production

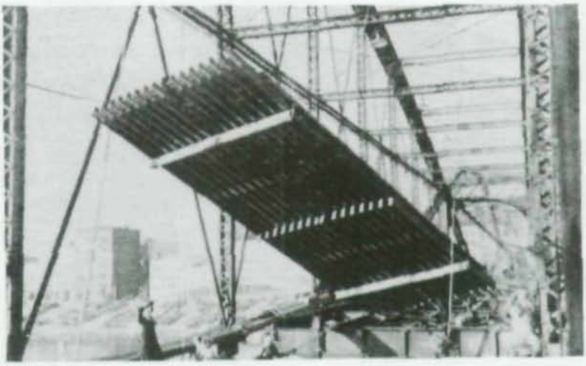


Figure 2 Framing for Smithfield Street Bridge Deck - 1933

Figure 3 Grass River Bridge Girders

Figure 4 Curtain Walls on the Alcoa Building

Figure 5 Aluminum Structural Technology Leaders

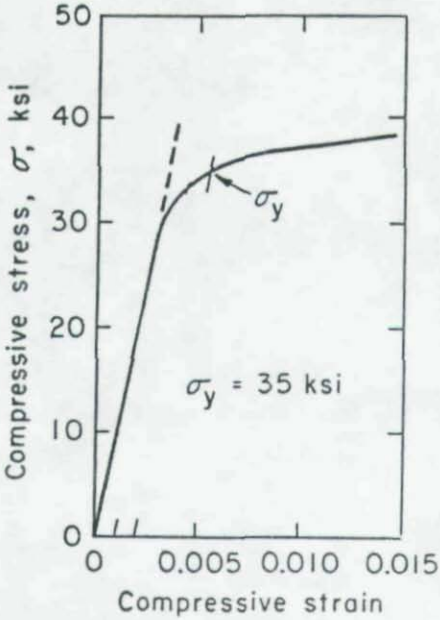


Figure 6 Stress-Strain Curve for 6061-T6

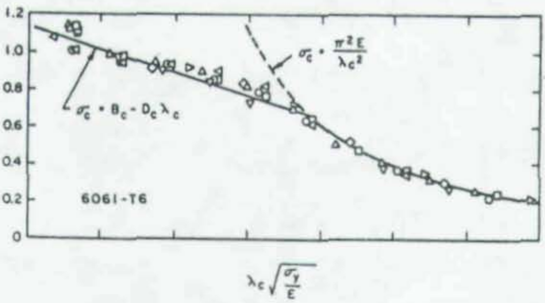


Figure 7 Straight Line Column Formula

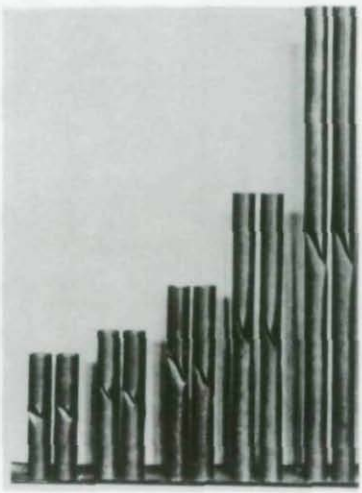


Figure 8 Tubes Tested in Torsion

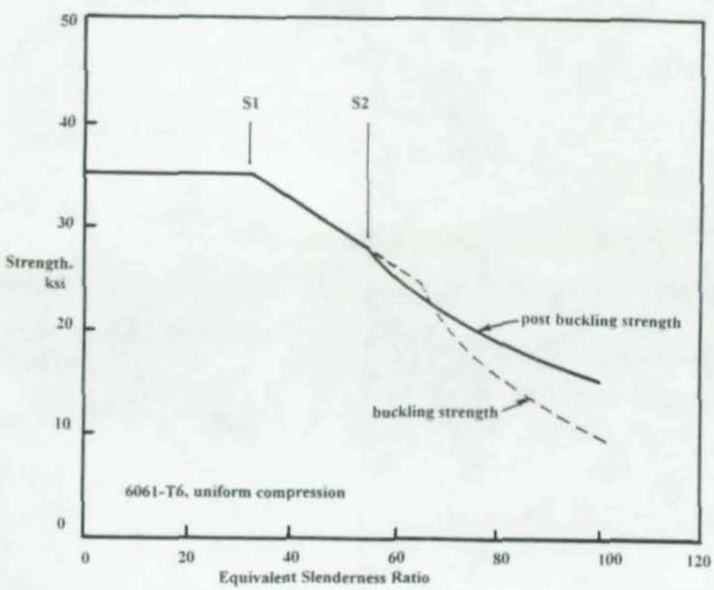


Figure 9 Equations for Design of Plates

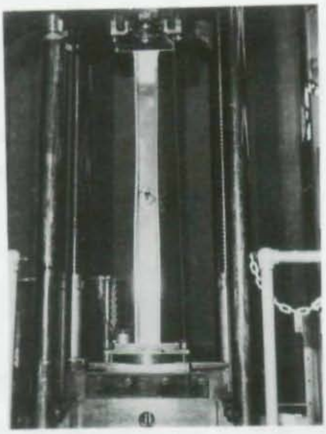


Figure 10 Buckling of Lipped Flanges

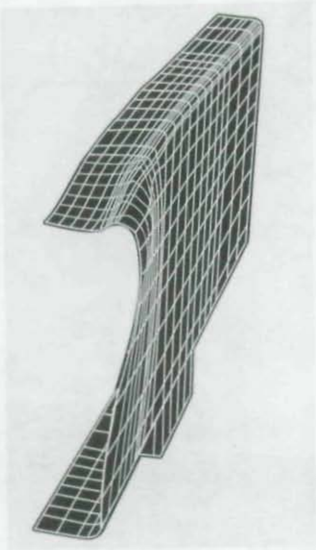


Figure 11 Web Crippling of Section

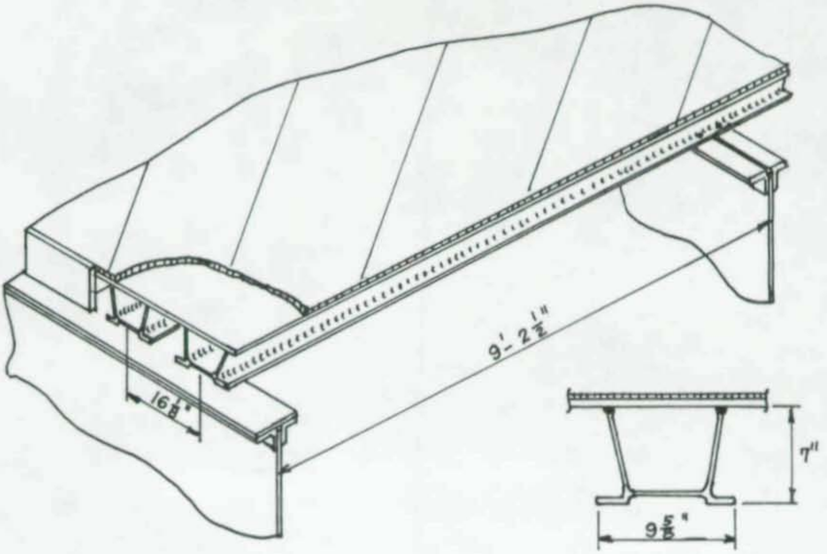


Figure 12 Bridge Deck

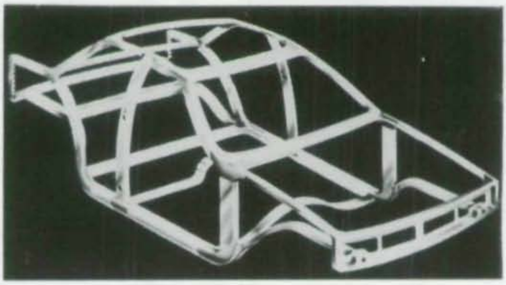


Figure 13 Automotive Space Frame

PLATE AND BOX GIRDERS FROM RESEARCH TO PRACTICE

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1. INTRODUCTION:

Plate and Box girders are mostly used in bridges and industrial buildings where large loads and/or long spans are frequently encountered. Recently, thin steel plate shear walls have been effectively used in buildings. Such walls behave as vertical plate girders; the boundary members in this case consist of the building columns acting as flanges and the floor beams as intermediate stiffeners. This paper addresses the behavior and strength of plate and box girders (including steel plate shear walls), from research to practice. The paper does not address girders curved in plan.

Buckling of plates which are adequately supported along its boundaries is not synonymous with failure; plates exhibit post buckling capacity which can be several times its buckling strength, depending on the plate slenderness. Nowadays, most design codes take post buckling strength into consideration. In this paper aspects of stability and post buckling strength of plate and box girders and their components will be addressed.

The so-called diagonal tension field models as well as other approaches to determine the web ultimate capacity in shear will be reviewed. In some cases the load deflection characteristics up to the ultimate capacity need to be determined; this issue as well as the post buckling behavior under cyclic loading will be addressed. The stability of the compression flange in plate girders will be discussed. Additional problems in box girders related to the compression flange, such as stability under transverse and in-plane loads, and intermediate as well as load bearing diaphragms will be considered.

The web of a girder or in load bearing diaphragms, the plate can be subjected to in-plane compressive patch loading. The ultimate capacity under this loading condition is controlled by web crippling which can occur prior to or after local yielding. The issue of web crippling will be discussed and the design of stiffeners if required will be addressed. The presence of openings in plates subjected to in-plane loads is unavoidable in some cases, research work related to the stability and ultimate strength of plates with openings will be considered.

In order to reduce the weight of plate and box girders and to avoid the use of stiffeners for the sake of economy and improved fatigue life, the use of corrugated webs had been considered. Girders with corrugated webs have been used in Europe; in Germany and Sweden in buildings and in France in bridges. Results from research work on the behavior of girders with corrugated webs will be discussed and design rules established based on this research will be presented.

In general, the paper will address the aforementioned issues highlighting the link between research and practice. Unresolved problems which require research work and results of research work which did not make its way to practice will be identified.

2. ULTIMATE SHEAR CAPACITY OF WEB PLATES:

As stated earlier, in most design codes buckling is not used as a basis for design. Minimum slenderness ratios, however, are specified to control out-of-plane deflection of the web. These ratios are derived to give a small factor of safety against buckling, which is conservative and in some cases extravagant.

Before the web reaches its theoretical buckling load the shear is taken by beam action and the shear stress can be resolved into diagonal tension and compression. After buckling the diagonal compression ceases to increase and any additional loads will be carried by the diagonal tension. In very thin webs with stiff boundaries, the plate buckling load is very small and can be ignored and the shear is carried by a complete diagonal tension field action, as proposed by Wagner (1931). In welded plate and box girders the web is not very slender and the flanges are not very stiff; in such a case the shear is carried by beam action as well as incomplete tension field action.

Basler (1963), based on the results of tests he conducted, developed the analytical model shown in figure (1) to calculate the ultimate shear capacity of the web of a welded plate girder. The flanges were assumed to be too flexible to support the vertical component from the tension field. The inclination and width of the tension field were defined by the angle ø, which is chosen to maximize the shear strength; and a formula was derived to calculate the ultimate shear capacity of the web. It was shown later by Gaylord (1963) that Basler's formula gives the shear strength for a complete tension field instead of the limited band of figure (1). Therefore Basler's formula is not for a girder with flanges incapable of supporting the vertical component from the tension field. The results obtained from Basler's formula, however, were in good agreement with the test results, and the formula was adopted in the AISC specifications.

Many variations of the Basler incomplete tension field model have been developed; a review can be found in the SSRC Guide to Stability Design Criteria For Metal Structures (1988). A model which gives better results than Basler's model was developed by Rockey (1972-1975); and it has been adopted in codes in Europe. The model is shown in figure (2); near failure the tensile membrane stress together with the buckling stress causes yielding, and failure occurs when hinges form in the flanges to produce a combined mechanism that includes the yield zone ABCD. The vertical component of the tension field is added to the shear at buckling and combined with the frame action shear to calculate the ultimate shear strength. The ultimate shear strength is given as a function of ø, and ø is calculated by trial and error to give the maximum strength value.

Recently, Ajam and Marsh (1991) developed a simple model to calculate the shear capacity of the web. After buckling, the shear stress distribution changes to the form shown in figure (3). When the shear force increases the shear stress at corners "O" remains constant at the initial critical value, and the shear stress at the corners "A" rises until it reaches the yield stress of the web material. In a ductile web, a further increase in the shear force causes the web to distort as it yields locally in shear, thus forcing the flanges to bend. If the flange has bending stiffness, it resists and stresses normal to the boundary are introduced, which contributes to the shear resistance. Ajam and Marsh developed a model of the collapse mechanism in which the shear capacity is due to contribution by the shear in the web and by the bending of the flanges.

Evans and Mokhtari (1992), based on their own test results and results from tests conducted by others, have concluded that unstiffened webs possess a considerable reserve of post-buckling strength. The incomplete diagonal tension field approach, however, is *only reasonably accurate up to a maximum aspect ratio (stiffeners spacing / web depth)* equals to 6. Evans and Mokhtari, further, concluded that the model developed by Hoglund (1971) can not predict the ultimate capacity to a good degree of accuracy and research work is required to develop an appropriate method of predicting the post-buckling strength of unstiffened girders. Ajam and Marsh (1991) stated that their model can predict the ultimate capacity of unstiffened webs to a good degree of accuracy.

3. STEEL PLATE SHEAR WALLS:

In steel plate shear walls, the boundary members are stiff and the plate is relatively thin; in such a case Wagner's complete tension field can be developed. Thorburn, et al (1983), and Timler and Kulak (1983) modeled the plate as a series of tensile bars inclined at an angle ø. The angle of inclination ø is function of the panel length and height, the plate thickness, the cross-sectional areas of the surrounding beams and columns, and the moment of inertia of the columns, and is determined by applying the principle of least work. Although this model can predict the ultimate capacity to a reasonable degree of accuracy it fails to depict the load-deflection characteristics to the same degree of accuracy. In the aforementioned work by Thorburn et al and Timler and Kulak the material for the tensile bars were assumed to be elastic-perfectly plastic.

Elgaaly, Caccese, et al (1993) based on their test results recommended the use of a trilinear material model in order to depict the load-deflection characteristics to a reasonable degree of accuracy. Based on the test results and Finite Element Analysis conducted by Elgaaly and Caccese, the stresses in the inclined tensile plate strips are not uniform but are higher near the supporting boundaries than the center of the plate and yielding of these strips starts near their ends and propagates towards the mid-length. Although that the aforementioned trilinear material model was developed empirically, Elgaaly and Liu (will be published) were able to develop analytically a very similar model.

Although the post-buckling behavior of plates under monotonic loads has been under investigation for more than half a century, this behavior under cyclic loading has not been investigated until recently. Tromposch and Kulak (1987) tested one specimen with a story height of 2200 mm, a bay width of 2750 mm, and a plate thickness of 3.25 mm. It was intended to load the specimen with a gradually increasing fully reversed cyclic load to failure. Due to failure of some parts of the loading system, only a total of 28 cycles were completed and the maximum applied load was about 67% of the ultimate load subsequently attained. A one third scale model of Tromposch and Kulak was tested by Elgaaly, Caccese, and Martin (to be published) and the results from the model and the prototype were in good agreement.

At the University of Maine, Elgaaly, Caccese, Chen, and Martin tested a total of 18 scaled specimens, ten were 1/4 scale and eight 1/3 scale; in addition to the aforementioned scaled model of Tromposch and Kulak specimen. The 1/4 scale models were for three story single bay shear walls and the 1/3 scale models were for two story single bay shear walls; see figure (4). The parameters considered included the connection between the plate and the beams and columns (bolted vs. welded), the effect of the column axial compression, the plate thickness and column cross section, and the presence of openings in the wall. The specimens were loaded in a gradually increasing fully reversed cyclic load and the tests were displacement controlled. Drifts starting with 0.25% up to 2%, in increments of 0.25% and three cycles each were applied. Analytical studies were conducted which include, studying the behavior of the test specimens and developing a simple model to depict the test results to a practical degree of accuracy, and studying the behavior of the shear wall as a component in a building. The analytical work considered equivalent static loads, response spectra modal analysis, and time-history analysis and included parametric studies. The results to date indicate that thin-steel plate shear walls are very effective in seismic design, and in the near future the research results will be formulated for possible adoption by the applicable specifications and codes.

4. STABILITY OF THE COMPRESSION FLANGE:

The compression flange of a plate girder subjected to bending usually fails in lateral torsional buckling, local buckling, or yielding. If the web is slender the compression flange can fail by vertical buckling into the web. Basler and Thürliman (1963) developed a limiting value for the web slenderness ratio to preclude this mode of failure. This limiting value may be too conservative since vertical buckling of the compression flange into the web occurs only after general yielding of the flange. This limiting value, however, can be helpful to avoid fatigue cracking under repeated loading due to out-of-plane flexing and it also facilitate fabrication. The results of Basler's work has been adopted by the specifications in the United States back in the sixties; the author is not aware of further research work in this area since then.

Lateral torsional buckling does not govern the design of the compression flange in a box girder. Unstiffened flanges of narrow box girders can be treated as long plates supported along their longitudinal edges and subjected to uniaxial compression. The post-buckling capacity can be determined using the effective width concept developed by von Kàrmàn (1932). The flanges in wide box girders, which are not commonly used in the United States, are stiffened in both the longitudinal and transverse directions. Methods to predict the ultimate strength of such flanges are reviewed in chapter 7 of the SSRC Guide (1988) by Dowling. The presence of the in-plane compression in the flange magnifies the deflection and stresses in the flange from local bending due to traffic lateral loading. The amplification factor $1/(1-P_a/P_{cr})$ can be used to increase the deflections and stresses due to local bending.

5. ADDITIONAL PROBLEMS IN BOX GIRDERS

Care must be exercised in applying the tension field models developed primarily for welded plate girders to the webs of a box girder. The thin flange of a box girder can provide very little or no resistance against movements in the plane of the web. If the web of a box girder is transversely stiffened then one can use Basler's tension field model to predict the web ultimate capacity. However, as mentioned earlier Basler's equations are based on a full tension field rather than a limited band as his model implies; in such a case Basler's ultimate strength equation can over predict the web strength. Hence, it is advisable to use Rockey's model assuming the plastic moment capacity of the flange to be negligible.

In box girders intermediate diaphragms are provided to limit cross-sectional deformation and load bearing diaphragms are used at the supports to transfer loads to the bridge bearings. Diaphragm design is treated in the BS 5400: Part 3 (1983) and discussed by Dowling in Chapter 7 of the SSRC Guide. Local crippling of unstiffened or stiffened diaphragms over the bearing pads will be addressed in the following sections of this paper.

6. IN-PLANE COMPRESSIVE EDGE LOADING

Webs of plate and box girders can be subjected to local in-plane compressive loads. Vertical (transverse) stiffeners can be provided at the location of the load to prevent web crippling; however, this is not always possible such as in the case of a moving load and it involves higher cost. During the past 60 years analytical and experimental studies of elastic buckling and ultimate strength of webs subjected to this kind of loading were performed, a summary of the work performed can be found in Elgaaly (1983).

During the last seven years research work has been conducted at the University of Maine to study the problem of web strength when subjected to compressive edge loading applied downward at the top flange between the supports or upward at the bottom flange at the supports. The studies included experimental and analytical work and some of the results were published by Elgaaly and Nunan (1989), Elgaaly, Sturgis, and Nunan (1989), Elgaaly and Salkar (1990 and 1991), Elgaaly, Salkar, and Du (1991), and Elgaaly, Salkar, and Eash (1992). Some limited work to examine the fatigue strength of the web when subjected to repeated loading of this type were carried out by Elgaaly and Thorat (to be published).

The results from the aforementioned research work lead to the following conclusions. Failure of the web under this loading is always due to crippling; in thin webs crippling occurs before yielding of the web and in stocky webs after yielding, see figure (5). The formula developed by Roberts (1981) predicts the crippling load to a reasonable degree of accuracy; this formula was adopted by the AISC specification. It has to be noted, however, that this formula was developed from tests where the loads were applied downward on the top flange of the girder near mid span. Applying the formula, by taking half the ultimate capacity, for web crippling over the supports will yield conservative results. A modified formula for this case has been adopted in the new edition of the AISC specification to eliminate some of this conservatism. In the AISC specification two formulas are given to check web crippling; one if the load is applied at a distance not less than half the member's depth from its end and another if the load is at a distance less than half the member's depth. The results from the aforementioned research indicate that this issue need to be addressed in a better fashion.

7. ECCENTRIC EDGE LOADING:

Eccentricities in loading with respect to the plane of the web are unavoidable. The previously mentioned research work, which was conducted at the University of Maine, included a study on the effect of small eccentricities on the web strength. It was found that there is a reduction in the web capacity due to the presence of an eccentricity; for example, in one case, an eccentricity of 0.5 inch reduced the web ultimate capacity to about half its capacity under in-plane load. Furthermore, it was found that the effect of the load eccentricity in reducing the ultimate capacity decreases as the ratio of the flange to web thickness increases. A deformed beam subjected to eccentric load near failure is shown in figure (6). Web strength reduction factor for various eccentricities as a function of the flange width and for various flange to web thickness are given in figure (7).

The failure mechanism in the case of eccentric loading is different from that for in-plane loading. The flange twisting moment acting at the web flange intersection can cause failure due to bending rather than crippling of the web, if the eccentricity is large enough. In most of the cases, however, the failure mode is due to a combination of web bending and crippling. Failure mechanisms were developed by Elgaaly and Salkar (to be published) and formulas to calculate the ultimate capacity of the web under eccentric edge loading were derived. Currently, the specifications are not addressing the effect of the eccentricity on the reduction of the web crippling load. Eccentricities can arise also due to moments applied to the top flange in addition to vertical loads. An example would be a beam resting on the top flange of another beam and the two flanges are welded together. Rotation of the supported beam will impose a twisting moment in the flange of the supporting beam and bending of its web, which will reduce its crippling load.

8. LOAD BEARING STIFFENERS:

Webs of girders are often strengthened with transverse stiffeners at points of concentrated loads and over intermediate and end supports. The AISC specifications require that these stiffeners must be double sided, extend at least one-half the beam depth, and either bear on or be welded to the loaded flange. The specification, further, require that they shall be designed as axially loaded members with an effective length equals to 0.75 times the web depth; and a strip of the web, with a width equals 25 times its thickness for intermediate stiffeners and 12 times the thickness for end stiffeners, shall be considered in calculating the geometric properties of the stiffener.

Analytical and experimental studies were performed at the University of Maine by Elgaaly et al. In these studies several parameters, such as the stiffener thickness and depth, width of the loaded patch and the load eccentricity with respect to the stiffener's vertical centerline, were considered. The studies covered intermediate stiffeners under a load which was applied at the top flange, as well as end stiffeners over supporting pads under the bottom flange. In the later case the load was applied at the center of the beam span.

For intermediate stiffeners the load was applied as a concentrated or a patch load. In some cases the load was applied through an I-beam placed on the top flange perpendicular to the loaded beam; in such case the behavior was similar to that of a concentrated load. The failure, in cases where the stiffener depth is less than 75% the depth of the web, was due to crippling of the web below the stiffener, see figure (8). The failure, otherwise, is due to global buckling of the stiffener provided that the thickness of the stiffener is adequate to prevent local buckling. Based on the results from the studies, it appears that the optimum depth of the stiffener is 0.75 times the web depth.

Similar conclusions can be reached for end (or support) stiffeners, see figure (9). It appears that the specification requirements are not adequate and do not account for factors such as the stiffener depth and load eccentricity. The experimental and analytical results are being examined and failure mechanisms are being considered with the ultimate goal of providing design recommendations.

9. WEB OPENINGS:

Openings are frequently encountered in the webs of plate and box girders. Research work on the buckling and ultimate strength of plates with rectangular and circular openings subjected to in-plane loads has been performed by many investigators. The research work included reinforced and unreinforced openings.

Narayanan and Der Avanesian (1983), developed a theoretical method of predicting the ultimate capacity of slender webs containing circular and rectangular holes, and subjected to shear. The solution is obtained by considering the equilibrium of two tension bands, one above and the other below the opening. These bands have been chosen to conform to the failure pattern observed in the plate girders with holes tested at University

College, Cardiff. Experimental results showed that the method gives satisfactory and safe predictions. The calculated values were found to be between 5 and 30% below the test results.

The ultimate strength of plate girders with perforated webs under shear and bending was investigated by Lee (1991). The girders considered were transversely stiffened with aspect ratios (b/d) between o.7 and 1.5. The holes in the webs are centrally located and they are circular, elongated circular and rectangular shapes. The ultimate strength was determined using an analytical model developed on the basis of stress fields and load-carrying mechanism observed experimentally from tests (1989) and numerically from finite elements analysis (1990) performed by Lee. The solution allows for the variation in panel and hole size and the predicted results proved to be accurate by comparing with the experimental (results from 70 tests) and numerical (using finite elements) results, see figure (10). The solution is applicable for webs with depth to thickness ratios of 120 to 360, panel aspect ratio between 0.7 and 1.5, hole depth greater than 1/10th of the web depth; and for circular, elongated circular, and rectangular holes.

Elgaaly, et al (to be published) tested three shear wall specimens with two rectangular openings per panel; stiffened and unstiffened openings were considered. Elgaaly and Liu (to be published) developed an analytical model to predict the behavior of the shear wall with openings to a good degree of accuracy; the model is based on the diagonal tension field theory. The results from the aforementioned research on plate girders and shear walls are in a form which can be easily incorporated in design guides, codes, and specifications.

10. GIRDERS WITH CORRUGATED WEBS:

Corrugated webs can be used in an effort to decrease the weight of steel girders and reduce its fabrication cost. Studies have been conducted in Europe and Japan and girders with corrugated webs have been used in these countries. The results of the studies indicate that the fatigue strength of girders with corrugated webs can be 50% higher compared to girders with flat stiffened webs. In addition to the improved fatigue life, the weight of the girders with corrugated webs can be as much as 30 to 60 percent less than the weight of girders with flat webs and same capacity. Due to the weight savings, larger clear spans can be achieved. A summary of the research work conducted, and the use of girders with corrugated webs can be found in a paper by Elgaaly et al (1991).

Research work on girders with corrugated webs have been conducted by Elgaaly, Smith, and Hamilton (1992) at the University of Maine. Tests were performed to study the behavior of girders with corrugated webs under uniform shear and uniform bending. The shear tests were performed on web panels of various aspect ratios. Four different corrugation configurations and two different thicknesses were considered. The web panels tested under uniform bending were square and four different corrugation configurations and two thicknesses were considered. Failure in the shear test specimens were due to elastic buckling of the web and in the bending test specimens failure was due to yielding of the compression flange and its vertical buckling into the corrugated web which buckled. The test results and its correlation to results from empirical models will be discussed later.

Finite element models were developed for the test specimens and the analytical results were found to be in good agreement with the test results, Elgaaly and Seshadri (to be published) . The program utilized in this analysis was ABAQUS; this program is very efficient in determining the buckling strength of thin-plate structures. Some of the Finite Element Analysis results will be presented later in this paper. After the calibration of the

Finite Element model, the model will be used to perform parametric studies considering different corrugation configurations, thicknesses, and loading conditions. The results from the parametric studies will lead to design rules and recommendations. With the advancements in welding technology girders with corrugated webs are possible to fabricate and very efficient to use. Engineers, however, will not use them without design rules and recommendations in the applicable codes and specifications.

11. GIRDERS WITH CORRUGATED WEBS UNDER UNIFORM SHEAR:

Elgaaly and Hamilton (to be published) tested a total of 42 specimens using four different trapezoidal corrugation configurations and two different thicknesses; namely 0.024", and 0.03". The depths of corrugation used are 9/16", 1", 21/16", and 2", and the corresponding pitch was 2.5", 5", 5.14", and 6", respectively . The panels considered were 12x12 (width x depth), 18x12, 24x12, 12x18, 18x18, and 12x24 inches. The specimens were fabricated by Lincoln Electric Company in Cleveland, Ohio. In all specimens the load was applied over the central stiffener at mid span, thus loading the panels on each side in uniform shear. The thickness of the material on each side was different and the thinner panel failed first. After the first failure the failed panel was reinforced using cross bracing and the thicker panel was tested to failure. At no time over the course of testing did a weld between the 1/2" flanges and stiffeners and the thin corrugated panel break.

Failure of all specimens was due to elastic buckling of the web; the web material yield strength is about 90 ksi. The buckling mode was global for the dense corrugation, local for the course corrugation and combinations of local and global for corrugations in between, see figure (11). Usually, local buckling occurs first followed by global buckling. The load carrying capacity of the specimens dropped after buckling; the specimens, however, demonstrated a residual load carrying capacity after failure.

The ultimate capacities of the specimens tested were calculated using the formula in the Swedish Code for Light Gage Metal Structures (1982). The formula in the Swedish Code yielded results comparable to those obtained from the tests, however, on the conservative side. The test specimens were able to carry loads of 32%, on the average, more than what the Swedish formula predicted. For the dense corrugations (depth of 5/16"), however, an orthotropic plate solution developed by Easly (1975) and modified by Elgaaly and Hamilton yielded results which correlates better with the test results.

As stated earlier Finite Element models are being used to increase the data base and examine the effect of various parameters. When the analytical work is completed it is hoped that a semi-empirical formulae will be developed to predict the shear ultimate capacity of corrugated webs. These formulae with design recommendations can be employed by applicable codes and specifications thus allowing engineers to use corrugated webs.

12. GIRDERS WITH CORRUGATED WEBS UNDER PURE BENDING:

Although the behavior of corrugated webs under pure shear has been examined by many investigators in addition to the author; no studies were conducted to examine the corrugated web behavior under uniform (or pure) bending. Elgaaly and Hamilton (to be published) tested six specimens in which a 12"x12" panels were loaded in uniform bending up to failure. Four of the specimens were made of the thinner material and included the four corrugation configurations previously tested in shear. The 21/16" and 2" deep corrugations were tested also for the thicker material.

All specimens failed at a moment exceeding the plastic moment capacity of the specimen ignoring the web contribution. The mode of failure observed was that of the compression flange yielding and vertically buckles into the web, see figure (12). It has to be noted that the failure was sudden with no appreciable residual strength. As the flange buckles into the web the vertical component of the flange force was resisted by the corrugated web, thus adding to the moment capacity of the girder. The final collapse occurs when the web buckles under the vertical component of the flange force. Based on the test results, a simple analytical model was developed to predict the moment capacity of the test specimens to a reasonable degree of accuracy.

The results from the research work conducted in the USA and Europe are suitable for adoption in design guides, codes, and specifications in the United States; European codes and specifications include provisions for the design of girders with corrugated webs. Furthermore, due to their light weight and low cost of fabrication because of the absence of stiffeners, girders with corrugated webs are recommended for use in bridges. Limited tests were conducted in Hungary by Korashy and Varga (1979) to examine the fatigue strength of these girders and the results are very encouraging, some limited tests will be performed by the author at Drexel University in the near future. A comprehensive study program to investigate the fatigue strength of girders with corrugated webs is overdue

13. SUMMARY AND CONCLUSIONS

The paper is an attempt to provide a summary of recent research work on problems related to plate and box girders and steel plate shear walls. The results are briefly described and the transfer of these results to practice by incorporation in design guides, codes, and specifications is addressed. Since buckling of flat plates which are adequately supported along their edges does not mean failure, buckling of plates was not addressed in this paper other than when related to the ultimate strength. One can find in the technical literature many solutions for plate buckling problems, a good reference is the book by Bulson (1970).

Problems which are addressed in the specifications but the solutions need to be updated, new problems for which solutions are available and need to be included in the specifications, and problems which require further research work have been identified. New technologies in plate and box girders applications such as steel plate shear walls and girders with corrugated webs have been addressed. The research in steel plate structures in general and plate and box girders in particular is very active. The specifications are catching up and making the results of the research available for practicing engineers.

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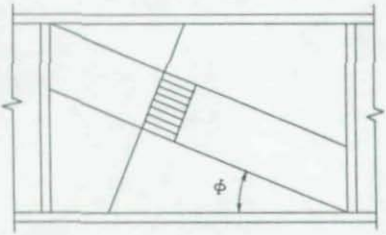

Figure (1) Basler's Tension Field Model

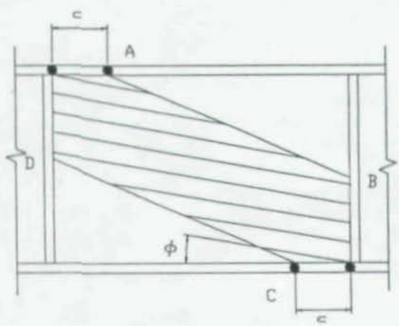

Figure (2) Rockey's Tension Field Model

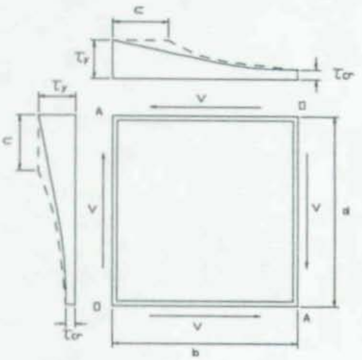

Figure (3) Shear Stress Distribution After Buckling
Ajam and Marsh (1991)

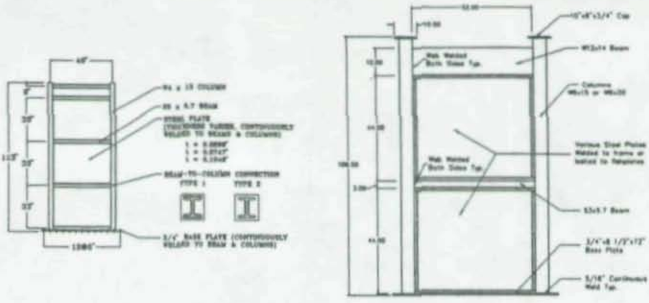

Figure (4) Steel Plate Shear Wall Specimens
(Elgaaly and Caccese)

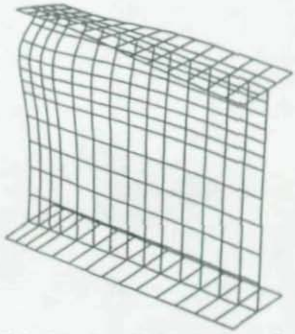

Figure (5) Web Crippling Under In-Plane Compressive Edge Loading, (Elgaaly and Salkar)

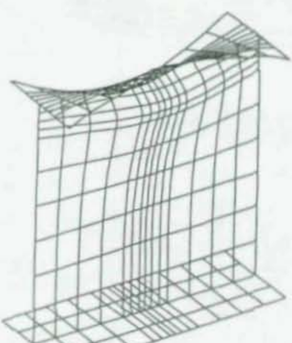

Figure (6) Web Failure Under Eccentric Edge Loading
(Elgaaly and Salkar)

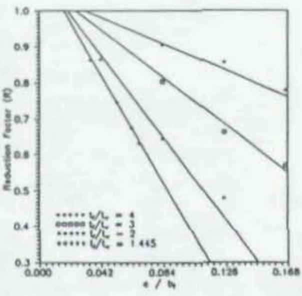

Figure (7) Web Strength Reduction Due To Eccentricities
(Elgaaly and Salkar)

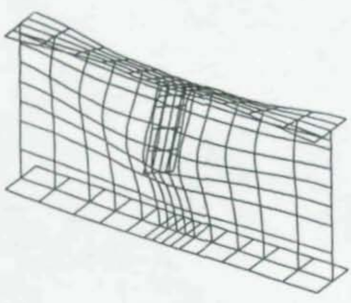

Figure (8) Web Failure of A Stiffened Web With Shallow
Stiffener - Half Web Depth
(Elgaaly and Salkar)

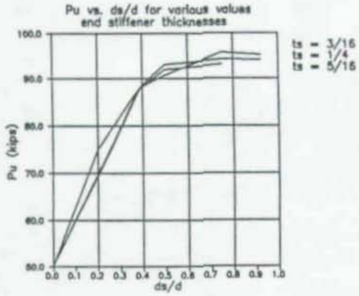

Figure (9) Ultimate Web Capacity vs End Stiffener Depth
(Elgaaly and Eash)

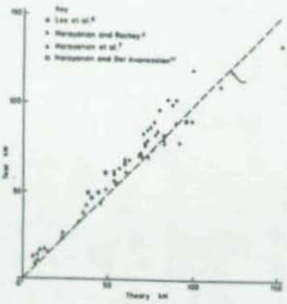

Figure (10) Comparison Between Tests and Theoretical Results (Lee - 1990)

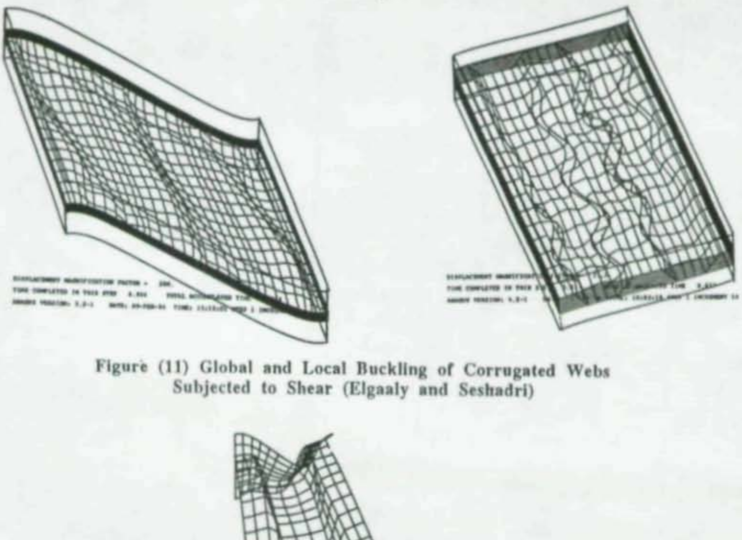

Figure (11) Global and Local Buckling of Corrugated Webs Subjected to Shear (Elgaaly and Seshadri)

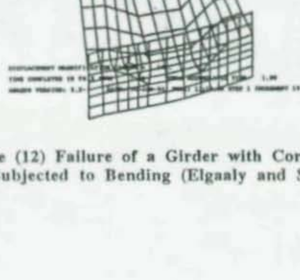

Figure (12) Failure of a Girder with Corrugated Web Subjected to Bending (Elgaaly and Seshadri)

THIN-WALLED METAL CONSTRUCTION:
RESEARCH, DESIGN AND CODIFICATION

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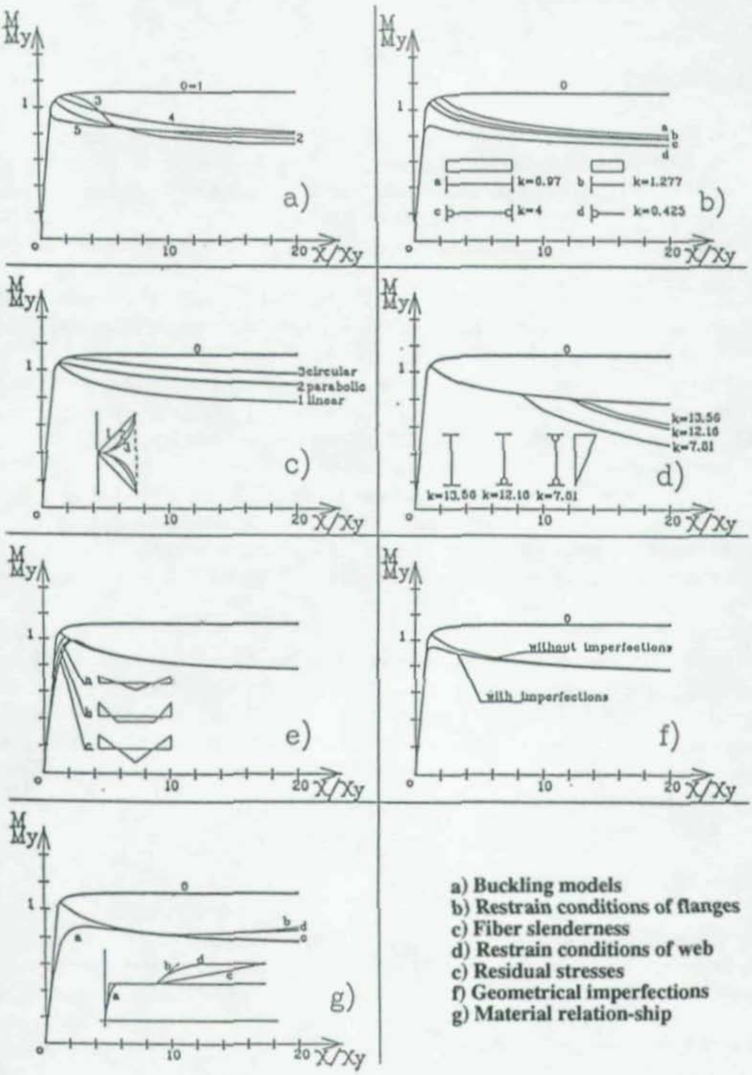
1. INTRODUCTION

One of the main activities in the future of metal structures will be the use of cold-formed thin-walled structures in civil and industrial buildings. This trend justify the development in the last years of many theoretical and experimental researches, together with the assessment of new calculation methods and design provisions.
The structural behaviour of cold-formed steel sections and trapezoidal sheetings is a research subject which has been carried out in the last decade at the Engineering Faculty of Naples. This activity is framed into the national and international developments which are related to many subjects including the bending behaviour of trapezoidal sheets, both standard [1 to 3] and long-span [4 to 10], the diaphragm action of cladding panels [11 to 15], the flexural collapse of cold-formed beams and the interaction between local and lateral instability [16 to 25], their seismic behaviour [26], the comparison among the design provisions [27,28] which are recently issued [30,31,32], the design aids [33].
The moment versus curvature relation-ship for a given section belonging to these typologies has been evaluated by means of an appropriate simulation programme [17,18,19], which allows to take into account all phenomenogical aspects influencing the bending behaviour up to the collapse, such as the different models for interpreting local buckling, the restrain conditions of flanges and web, the presence of geometrical and mechanical imperfections (see fig.1).

2. STANDARD SHEETINGS [1,2,3]

A systematic analysis of the existing trapezoidal steel sheets communly used in the market has been performed in order to compare their carrying capacity at the light of their ultimate behaviour and then to optimize the shape.



a) Buckling models
b) Restrain conditions of flanges
c) Fiber slenderness
d) Restrain conditions of web
c) Residual stresses
f) Geometrical imperfections
g) Material relation-ship

fig.1

The complete study has been carried out through the following phases:

a) Execution of bending tests on selected trapezoidal sheets with different dimensions and thicknesses;
b) Interpretation of experimental results;
c) Numerical evaluation of the bending process of the section taking into account the local instability phenomena of compressed parts by means of the general simulation procedure;
d) Evaluation of bending strength according to the main provisions and comparison with experimental and numerical results.

The experimental measurement system was designed so as to allow the evaluation of both the global response and the local behaviour of the midspan section (fig.2a).

In order to interpret the obtained test results, the specimens have been subdivided in two groups (figs.2c and d). By inspiring to the behavioural classed proposed in EC3, as classification parameter the b/t ratio of the compressed flanges has been assumed. These groups are:

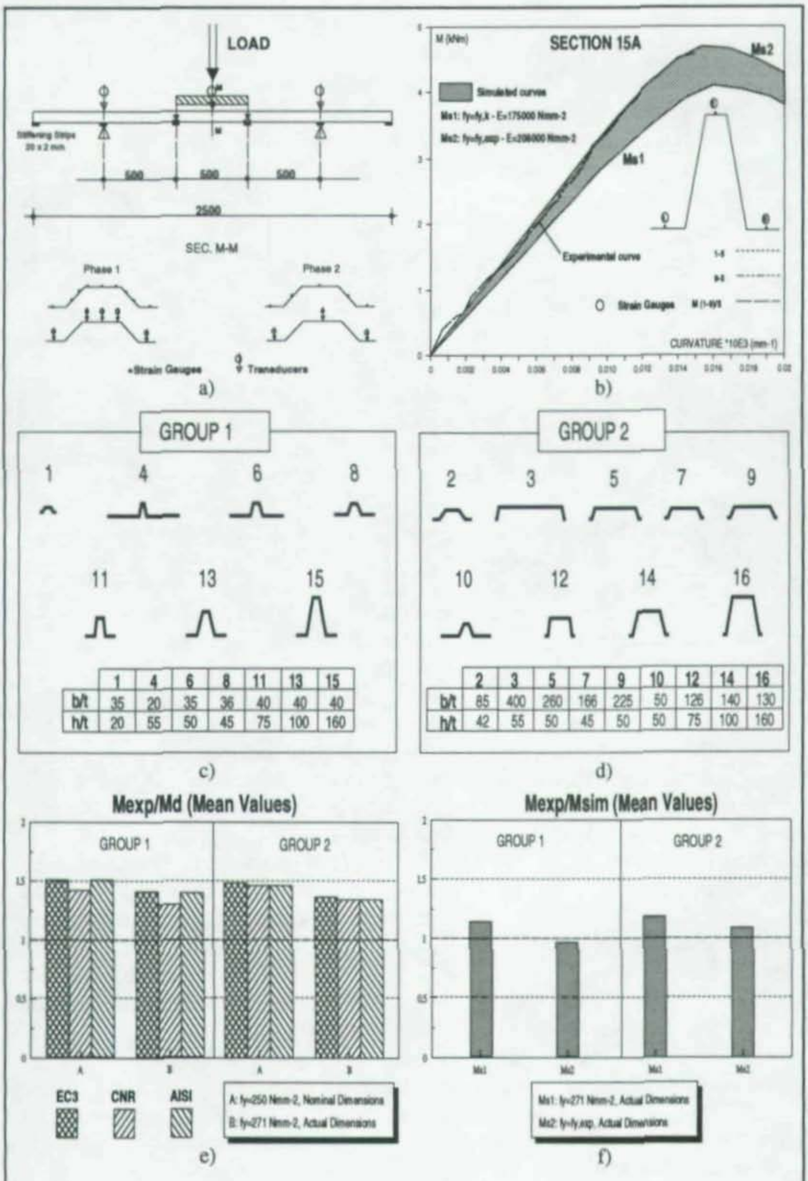
1th group, including specimens with $b/t<41$

2th group, including specimens with $b/t>41$

The shapes belonging to the second group are certainly of class 4 (slender sections), the complete classification of specimens of the first group requires the evaluation of the web slenderness. Anyhow the two groups are representative of two different bending behaviours due to the influence of local buckling. The experimental response in term of force versus displacement relation-ship for all specimens belonging to the first group emphasized an elastic behaviour almost linear until the failure, whereas for specimens of second group a decreasing stiffness is observed due to the spread of local buckling phenomena.

The moment-versus curvature behaviour is primarily influenced by the value of strain in the compression flange, since the tensile one virtually remains elastic. For specimens of first group these curves show an initial linear trend which later on becomes non linear due to yielding occurring in the compression flange and local buckling in the web, the last phenomenon occurring in case of deepest shapes.

The second group of specimens provided very irregular curves, being influenced by both local buckling of the top flange and yielding of the tensile one. Local buckling usually affects the behaviour even for low values of loads. Yielding of the bottom flange affects the behaviour for large curvature values, sometimes leading to quasi-flat branches.


LOAD
SEC. M-M
Phase 1
Phase 2
500
500
500
2500
Strain Gauges
Transducers
a)
SECTION 15A
M (kNm)
Simulated curves
Ms1: fy=fy,k - E=175000 Nmm-2
Ms2: fy=fy,exp - E=206000 Nmm-2
Ms1
Ms2
Experimental curve
Strain Gauges
CURVATURE *10E3 (mm-1)
b)
GROUP 1
1 4 6 8
11 13 15

	1	4	6	8	11	13	15
b/t	35	20	35	36	40	40	40
h/t	20	55	50	45	75	100	160

c)
GROUP 2
2 3 5 7 9
10 12 14 16

	2	3	5	7	9	10	12	14	16
b/t	85	400	260	166	225	50	126	140	130
h/t	42	55	50	45	50	50	75	100	160

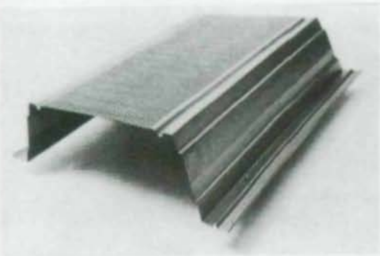
d)
Mexp/Md (Mean Values)
GROUP 1
GROUP 2
EC3
CNR
AISI
A: fy=250 Nmm-2, Nominal Dimensions
B: fy=271 Nmm-2, Actual Dimensions
e)
Mexp/Msim (Mean Values)
GROUP 1
GROUP 2
Ms1: fy=271 Nmm-2, Actual Dimensions
Ms2: fy=fy,exp, Actual Dimensions
f)

fig.2

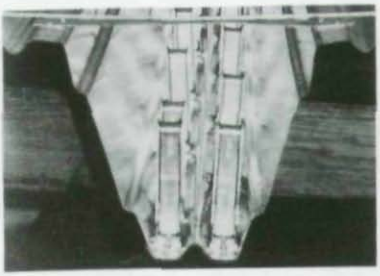
The comparison between experimental and simulation results (fig.2b) has been done by using a scatter band defined by two values of the elastic modulus: the true one for the upper bound and a conventional reduced value (15 % reduction) forfettary taking into account the lowering in stiffness due to initial geometrical imperfections. This approach allowed a good interpretation of the experimental moment versus curvature results.
The scatters between the ultimate moment given by test M_{exp} and by simulation M_{sim} have been evaluated, considering both the nominal value of the yield stress and the experimental one (fig.2f): the simulation give always a good agreement. The same ratio M_{exp}/M_d has been evaluated by means of the italian (CNR 10022 [31]), american (AISI [32]) and european (EC3-part.1.3 [29]) codes. The recommended values are very similar and they are always on the safe side, ranging the scatter from 1.3 to 1.5 (fig.2e).

3. LONG-SPAN SHEETINGS [4,5,6,7,8,9,10]

The development of steel sheetings over the past years has been characterized by three generations of shapes, according to their capacity to cover more and more large spans.
To the third generation belongs the trapezoidal unit with both longitudinal and transversal stiffners, which privides suitable solution for spans up to 12 m without purlins. The so-called TRP 200 is one outstanding exponent of this class (fig.3a). It is designed by Planija in Sweden and recently experienced also in Italy in the field of both industrial and multistory buildings; in the last case the cross-section is completed with casted concrete and with appropriate types of connectors (fig.3b).
Several series of tests have been carried out at the University of Naples, by considering different loading conditions (concentrated and distribuited) for different spans (figs.3c and d). The considerable number of experimental data allowed for a statistical interpetation and the characteristic values of ultimate bending moments have been compared with the nominal ones, which are in most cases on the safe side.
The perforated type with small holes for approximatively 17% of the surface of the profile for improving acoustic performance has exhibited a small reduction of bending capacity, approximatively 5%.



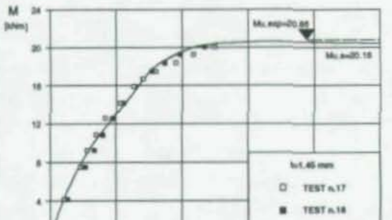
a)



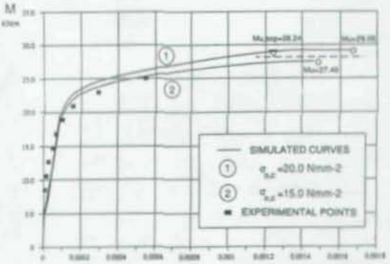
b)

c)

d)


M
0
4
8
12
16
20
24
0
2
4
Curvature
TEST n.17
TEST n.18
simulated curve

e)


M
SIMULATED CURVES
-20.0 Nmm-2
-15.0 Nmm-2
EXPERIMENTAL POINTS

f)

fig.3

The behaviour of TRP200 as a composite structure has been also analysed by means of experimental test on simply supported beams (fig.3c), by considering three type of systems:

- type A obtained by simply casting concrete inside the section up to 4 cm of slab above;
- type B including two steel reinforcement bars at the bottom of the section before casting;
- type C using special fastners (such as Hilti), connected by *rivets at the bottom flange*.

The three systems exhibit very different behaviours. It is brittle for type A, where the collapse occurred because of the formation of a plastic hinge at midspan. Higher strength and more ductile behaviour have been shown by type B, certainly due to the reiforcements. The behaviour of type C is quite similar to type A, but the collapse mechanism was different due to the formation of three plastic hinges in sections equally spaced of L/4.
The numerical results by using the general simulation programme have been in good agreement with the corresponding experimental ones (fig.3e). The analytical model has confirmed also in this case to be a suitable tool to improve the knowledge of the bending behaviour of this kind of structures.
A different simulation programme has been used for the composite systems. The numerical model has been calibrated on the base of the experimental results, leading to a good approximation in interpreting both elastic and collapse behaviour (fig.3f).

4. DIAPHRAGM ACTION [11,12,13,14,15]

The diaphragm action due to the interaction of cladding panels and framed structure is usually considered in the well known methodology so-called "stressed skin design".
The effect due to cladding becomes particularly important when analysing the seismic behaviour of steel structures. The degree of collaboration between cladding panels and the main structure can be used to classify the structures as follows:

a. The main structure is designed to resist vertical and seismic loads, while the panel cooperation is taken into account in the serviceability limit state only, when checking the maximum sways and story drifts. In this case rigid beam-to-column joints are required.
b. The whole structure is designed so that panels and frames together have to resist vertical and seismic loads. Semi-rigid connections can be, therefore, accepted.

c. The frame has the task to resist vertical loads only, while horizontal forces due to earthquake or wind are supported by cladding panels, which play the role of bracings. In this case pin-ended connections can be used, providing the maximum economy in reducing both structural weight and manufacturing cost.

The definition of the analytical model to be considered in the stressed skin design requires particular attention to the type of panel and its connecting system.

The shear forces can be trasferred from the frame to the panel by means of a continuous connection along the perimeter or by means of a limited number of connections generally placed at the corners of the supporting structure. Each panel itself is composed by a given number of sub-panels which are connected together. These connections can be made using bolts, rivets, screws or welds.

There is therefore a wide range of cases which behaviour has to be studied in experimental way by sub-mitting the panels to shear forces both in monotonic and cyclic range (fig.4a). In this field is very important to underline the need to codify testing procedures for panels in order to allow an unified interpretation of the results.

The present knowledge in test results [13] shows that there is a wide difference of behaviour mainly due to the type of fasteners connecting the sub-panels each other.

Riveted connections cause a very brittle behaviour. The hysteresis loops are in general unsymmetric due to large slips and the energy dissipation capacity is very low.

A little improvement can be obtained by using screwed connections with a sensible increase of ductility and energy dissipation.

The use of spot welds for connecting sub-panels and the insertion of the complete panel into a perimetral frame provide an important increase of strength, ductility and energy dissipation.

These results have shown the importance to calibrate numerical models on experimental data in order to analyse structures completely braced by claddings.

From exhisting test results, two type of simplified analytical models have been set-up, according to the different types of fasteners connecting sub-panels:

- one for riveted and screwed connections (fig.4c);
- one for spot welded connections (fig.4d).

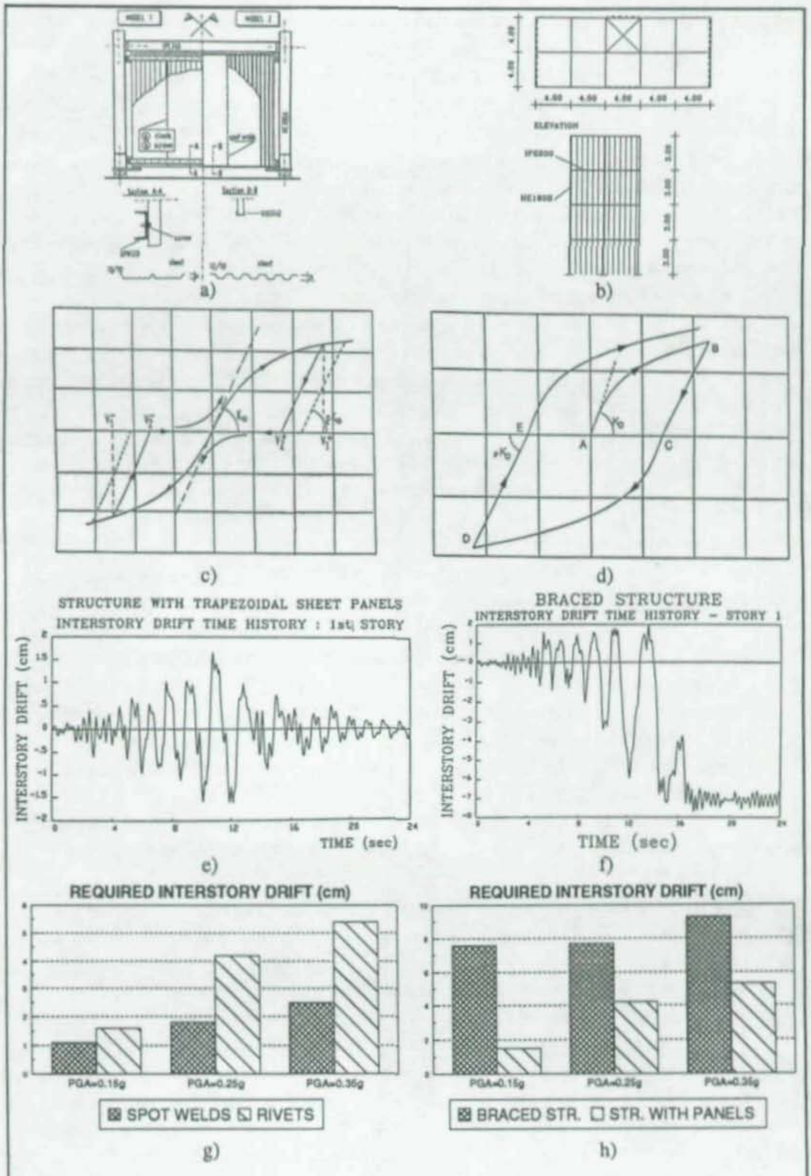

ELEVATION
a)
b)
A
B
C
D
E
c)
d)
STRUCTURE WITH TRAPEZOIDAL SHEET PANELS
INTERSTORY DRIFT TIME HISTORY : 1st STORY
INTERSTORY DRIFT (cm)
TIME (sec)
e)
BRACED STRUCTURE
INTERSTORY DRIFT TIME HISTORY - STORY 1
INTERSTORY DRIFT (cm)
TIME (sec)
f)
REQUIRED INTERSTORY DRIFT (cm)
PGA=0.15g
PGA=0.25g
PGA=0.35g
SPOT WELDS
RIVETS
g)
REQUIRED INTERSTORY DRIFT (cm)
PGA=0.15g
PGA=0.25g
PGA=0.35g
BRACED STR.
STR. WITH PANELS
h)

fig.4

Both are based on an increasing branch described by means of a curve of Ramberg-Osgood type.
From the dynamic analysis of a given four story building (fig.4b), interesting behavioural aspects have been observed. For a pin-ended structure the bracing effect can be obtained in two ways: traditionally, by means of X-bracings, or, in a more advanced concept, by means of claddings.
The seismic response of this building is quite different according to the two solutions. The comparison between the time history of the interstory drift at the first floor shows that in case of panel the drift values are quite symmetrical (fig.4e), but on the contrary in case of X-bracing the response is strongly unsymmetrical (fig.4f). The second behaviour is, obviously, unsatisfactory due to the worsening of the ductility demand for a fixed amount of energy dissipation.
For three different seismic zones, the maximum required interstory drift for a given structure has been evaluated (fig.4h). It represents also the required ductility, which has to be compared with the available one which is strictly dependant on the connecting system both for diagonals and panels.
In case of X-braced structure the ductility demand is considerably high for all seismic zones, so that the choice of the connecting system of bracing members is determinant and welded connections only seem to be suitable from the experimental evidence.
From these results the use of panel bracing for pin-ended structures in low sismicity zones at a first glance seems therefore to be convenient, due to the good dynamic performance and the reduction in structural weight, which can reach 20-25 % according to the different typologies.
The second aspect to be analysed is the influence of the type of connecting system in building-up each panel from the point of view of ductility demand, because the available ductility is strictly dependant on the behaviour of panels. The above analytical models (figs.4c and d) have been used to give an answer to the last question.
The comparison between the consequences of the use of rivets and of weld (fig.4g) immediately shows that panels composed by riveting performe a required ductility in term of interstory drift much more higher than in case of welding. As a conclusion, in case of structure braced by means of cladding panels, in low seismicity zones both panels with riveted and screwed connections can be used; on the contrary in higher seismicity zone only panels with welded connections and inserted into a perimetral frame are able to provide the required ductility.

5. COLD-FORMED PROFILES [16,17,18,19,20,21,22,23,24,25,26]

The European codes greatly penalize thin-gauge sections because of their thickness. The Eurocode 3 [29] subdivides the steel cross-sections into 4 classes depending on the b/t ratio of compressed flange and web; the European Recommendations for Steel Structures in Seismic Zones [30] allow their use in dissipative zones only for low values of such ratio. In this way, cold-formed profiles are mainly included in class 4 (slender sections) and their dissipative capacity is consequently not considered, their use being practically not allowed in seismic resistant structures. However, the classification of cross-sections in behavioural classes doesn't seem to be completely assessed, as being not confirmed by a sufficient number of theoretical and experimental investigations.

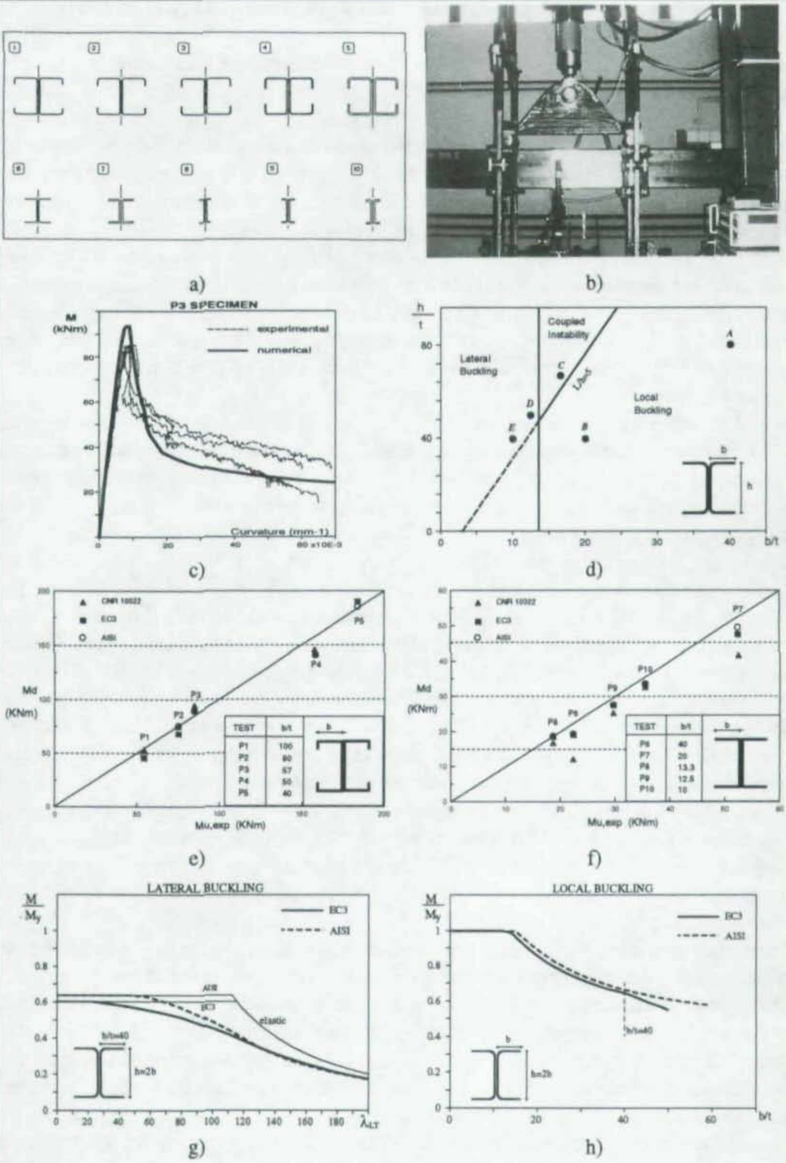
The aim of the research activity was to contribute to achieve a better comprehension and a systematical arrangement of this topic. The main purpose of the investigation was to study the flexural behaviour of cold-formed thin-gauge open cross-sections, in order to contribute to reduce the actual lack of knowledge.

Such an objective has been pursued both in theoretical and experimental way. In the first case the study consisted in simulating the flexural elasto-plastic behaviour of a section, by developing a numerical procedure able to follow its deformation history, taking into account local buckling phenomena (see fig.1). The experimental tests are also finalized to calibrate the proposed model, in order to make it available for extrapolation to simulate the behaviour of other sections of the same type, with different geometrical and mechanical characteristics.

Consequently the informations to get through experimental tests have to deal both with the overall behaviour parameters of the member (ultimate moment, rotational capacity, vertical displacement) and the local behaviour parameters, measured in the most stressed section.

In order to obtain a significant description of the dissipative behaviour, it has been necessary to test the specimens also in the post-critical field, until the overall collapse. The monitoring of characteristics was continuous until the end of test. In order to avoid sudden increases of deformation due to buckling phenomena, tests were performed imposing displacements and measuring the corresponding forces.

The experimental investigation has been carried out on 10 specimens (fig.5a), made of Fe360 steel, composed by two coupled cold-formed channels (with or without edge stiffeners). Cross sections were


a)
b)
P3 SPECIMEN
M
(kNm)
experimental
numerical
Curvature (mm-1)
c)
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t
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Lateral
Buckling
Local
Buckling
A
B
C
D
E
b/t
d)
CNR 10022
EC3
AISI
Md
(KNm)
Mu,exp (KNm)
TEST
b/t
P1 100
P2 80
P3 67
P4 50
P5 40
e)
P6 40
P7 20
P8 13.3
P9 12.5
P10 10
f)
LATERAL BUCKLING
M
My
EC3
AISI
elastic
b/t=40
h=2b
λ_{LT}
g)
LOCAL BUCKLING
b/t=40
h)

fig.5

designed to cover a wide range of b/t ratio values and then to represent all the behavioural classes provided by EC3, according to which they can be classified as follows:

1st class (plastic section): specimen P10
2nd class (compact section): specimen P5
3rd class (semi-compact section): specimens P8 and P9
4th class (slender section): specimens P1,P2,P3,P4,P6, P7.

Monotonic tests were perfomred on simply supported beams, 3.00 m of span; two concentrated loads were applied in two points, 1.00 m distant, across the midspan (fig.5b).

In case of stiffened sections the moment-curvature curves (fig.5c) appeared to be generally in a remarkable agreement with the experimental data both for the maximum moment value and for the shape of the decreasing branch.

On the contrary the experimental behaviour of some unstiffened specimens was affected by their flexural-torsional buckling. In fact the arising of local buckling in the compressed flange gave rise to a lateral displacement which gradually increased together with the imposed vertical displacement.

Figure 5d shows the fields of local, lateral and coupled buckling for the limit L/h ratio equal to 5, which corresponds the L/h ratio of the central unrestrained part of the tested beams. The location of the experimental points indicates that only the case C exhibits a coupled behaviour, as emphasized by test.

From the codification point of view, the three codes european [29], italian [31] and american [32] have been compared with the experimental results.

The ratios $M_{y,red}/M_{exp}$ are also plotted in figs. 5e and f, for stiffened and unstiffened sections, respectively. Design values for stiffened sections show a substantial agreement among different codes, while the reduced stress approach of the italian code CNR for unstiffened sections seems too much conservative in comparison with EC3 and AISI. Only for two specimen (P3 and P5) the design moments are slightly unsafe with respect to experimental values; however these differences may be covered by the partial safety factor γ_M for steel, which in the last version of EC3 is taken equal to 1.1. For every other profile the design moments are always smaller then the experimental ones.

6. CODIFICATIONS [27,28,29,30,31,32,33]

The recent approval of part 1.3 "Cold-formed thin-gauge members and sheeting" of EC3 stimulated the preparation of an ad hoc programme to help the designers in their practical activities

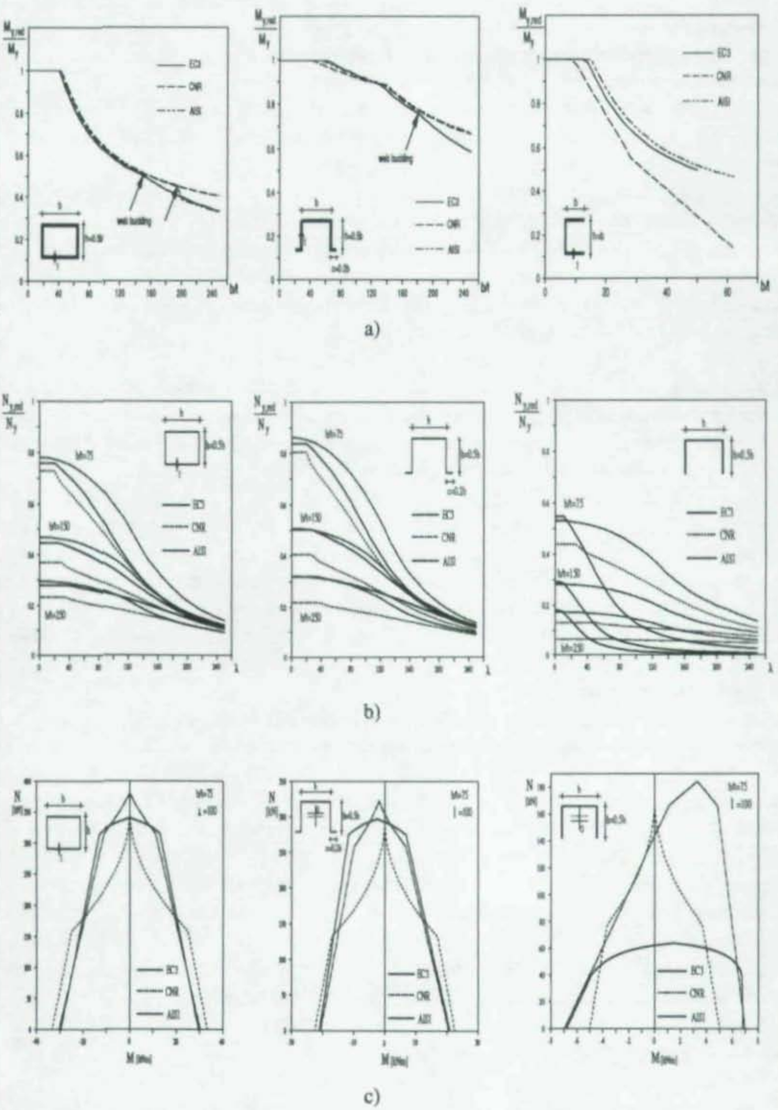

fig.6

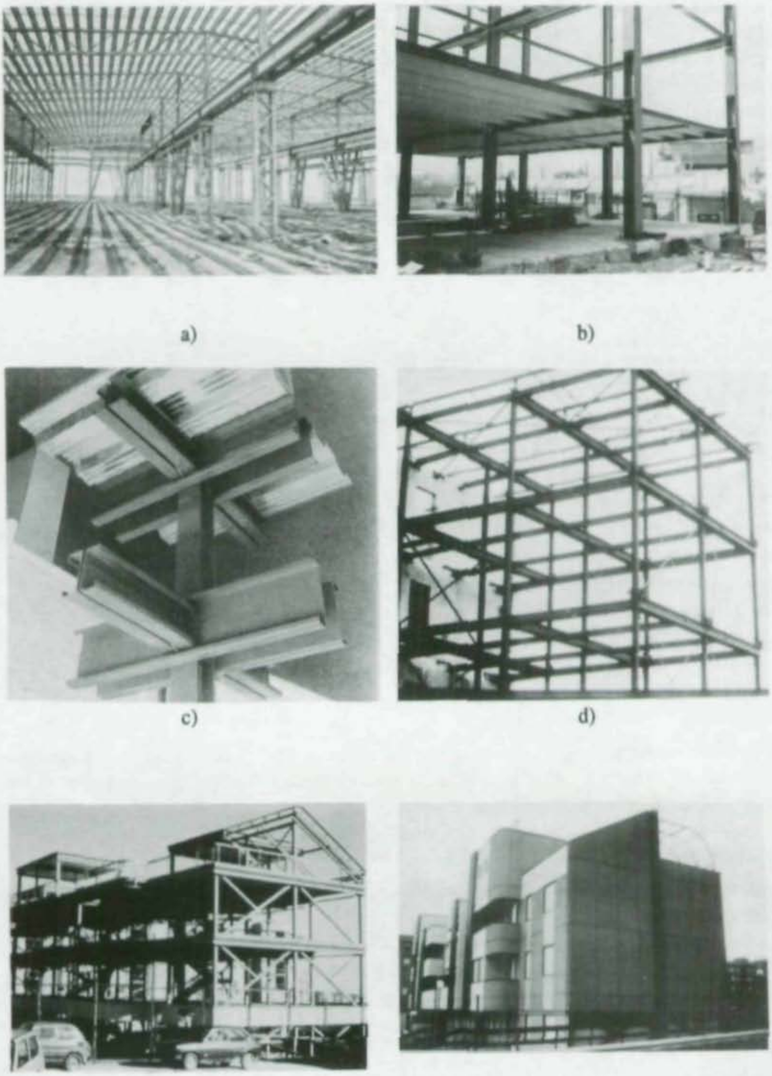

fig.7

[33].
It allows the comparison of EC3 provisions with the ones from the application of italian CNR and american AISI codes. The rules supplied for the analysis of each element in the section and for the evaluation of the load bearing capacity of members in bending (fig.6a), in compression (fig.6b) and under combined axial force plus bending (fig.6c) are discussed and compared.
Some differences between EC3 and AISI relating to webs and lipped flanges show the need for further investigations in order to assess the best way to model the actual behaviour of these elements.
The wider gap was observed in checking members subjected to coupled buckling and even more in case of combined axial force and bending.

7. APPLICATIONS

Third generation profiles have been used in Italy for roofing in the field of industrial buildings (fig.7a) and in floor structures of multistory buildings by completing its cross section with casted concrete (fig.7b). Some applications have been also proposed for the rehabilitation of old roofs and floor structures.
The constructional metasystem named BASIS (Building Activities Steel Integrated System) was presented in 1982, as a proposal in the field of industrialized buildings based on the use of steel elements. The main components of this system are (fig.7c):

- one type of column, made of HEB 140 rolled section, single for buildings of up to 4 stories and double for those up to 8;
- cold-formed steel sections for the main and secondary beams made of back-to-back channel in ranges of thickness from 4 to 8 mm and depth from 120 to 280 mm;
- corrugated steel sheets integrated with reinforced concrete slab for deck.

This constructional system can be used to resist horizontal loads both with steel bracing structures and with concrete core (fig.7d).
The BASIS has been used in Italy and abroad for apartment buildings.
In order to avoid the typical box-shape of the prefabricated buildings, some variations have been introduced to the system, giving a better architectural look. It is the case of the Civic Center of Civitavecchia (fig.7e), where the feature of the facade is characterized by curved balconies (fig. 7f).
Many applications of this system have been done in seismic zones in the new settlements after the earthquake of 1980.

A new important activity is devoted in Italy to reiforce and to upgrade the hystorical heritage of masonry buildings, continuously deteriorated by earthquakes. With this purpose, the corresponding research activity has been sponsored by the Ministry of Research and University in order to find out the suitable way of application of steelworks in the refurbishment operations. Cold-formed shapes and trapezoidal sheets have been considered as new materials, which can be conveniently used instead of the traditional technologies to give integrated structural systems.

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SHELL STABILITY
THE LONG ROAD FROM THEORY TO PRACTICE

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INTRODUCTION

The investigation of shell buckling will always have an appeal to researchers in solid mechanics. Not only are shell structure used in many fields, but their analysis brings one to the forefront of nonlinear analysis and sophisticated constitutive equations. Most research work in shell theory has been carried out in the aerospace industry. This is quite natural when one considers the necessity of using thin shells in flight vehicles. This paper will consider the problems involved in the transfer of this work to Civil Engineering structures.

BUCKLING OF SPHERICAL SHELLS UNDER INTERNAL PRESSURE

The field of shell stability is so broad that one must focus on some standard problem if one is to demonstrate the physical principles involved. Since the author is more familiar with the investigation of spherical shells, this problem will be reviewed. Buckling of spherical shells may be said to be the vogue topic in Solid Mechanics during the years between 1950 and 1970. Numerous papers were written on the topic; an excellent review of this research is presented in (16).

The basic problem may be stated simply. Zoelly's linear buckling analysis of a complete sphere under uniform internal pressure is described in Timoshenko and Gere (23). This analysis yields the buckling pressure

(1)
$$p_{cr} = \frac{2E}{\sqrt{3(1-\mu^2)}} \left(\frac{t}{R}\right)^2$$

In addition, it is found that the shell buckles in a large number of waves such that the wave length number can be approximated by the equation

(2)
$$n = 1.8\sqrt{\frac{R}{t}}$$

Therefore, if the length of the shell's meridian is large enough for the buckled wave length to develop, the boundary conditions at the support of a spherical shell segment should not affect the buckling pressure.

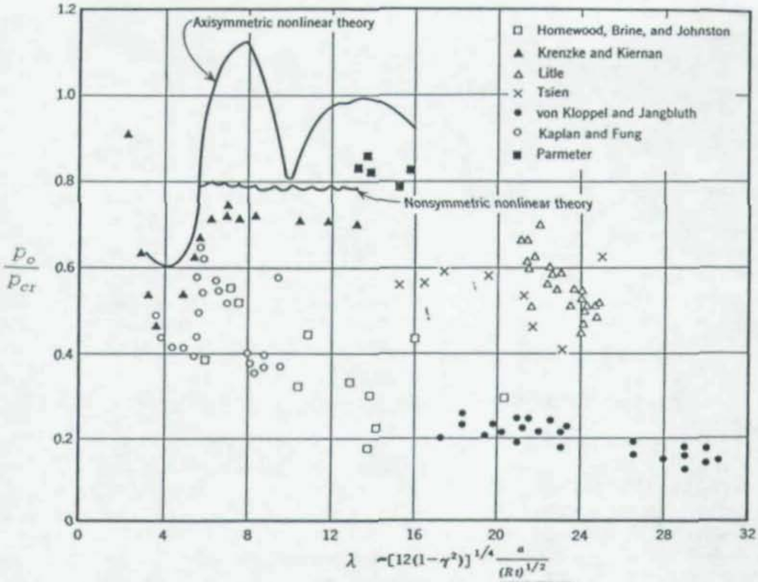

FIGURE 1. RESULTS OF EXPERIMENTS ON SPHERICAL SHELLS

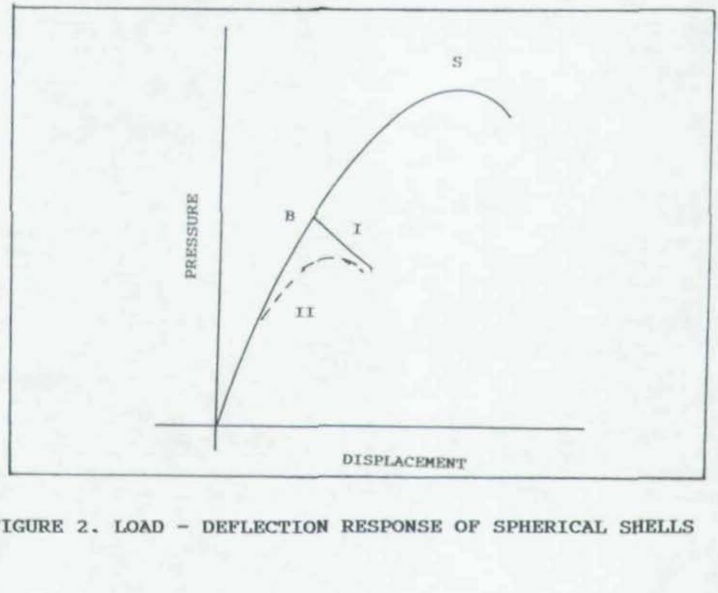

FIGURE 2. LOAD - DEFLECTION RESPONSE OF SPHERICAL SHELLS

Unfortunately, tests on spherical shells under internal pressure do not come even close to the classical buckling pressure presented in Eq (1). Figure 1 presents some of the experimental results. As can be seen, most experimental values are very low when compared to the linear buckling value for a complete sphere.

A similar problem exists in the investigation of a cylinder under longitudinal compression (7). There, it was found necessary to apply nonlinear shell theory. The same approach was taken for the spherical shell problem. Most researchers investigated the buckling of shallow spherical caps because the shallow shell equations were a little easier to solve. (It might be mentioned in passing that this problem was investigated in the early days of computer applications and tested the capacities of the then existing computers.) In addition, since the boundary conditions were not expected to play a critical role, only a shallow portion of the shell was expected to control stability. A typical nonlinear load-deflection is shown in Figure 2. Symmetric buckling occurs at the horizontal tangent at point S ; such a point is called a limit point.

The results of Budiansky's symmetric nonlinear analysis (5) are shown in Figure 1 . As can be seen, except for small values of λ, the analytical results obtained deviate even further from the experimental values. Budiansky also introduced symmetric imperfections into his shell equations, but their inclusion in the analysis did not reduce the buckling pressure very much.

Huang (14) solved the problem as a nonlinear bifurcation problem. That is, the nonlinear symmetric equations were solved and used to form eigenvalue problems for infinitesimal asymmetric deflections at selected points along the load-deflection curve. A typical bifurcation point is point B in Figure 2. If a bifurcation point occurs prior to a limit point, the corresponding load is taken as the buckling load. The results of this nonlinear asymmetric analysis are also shown in Figure 1. There is fair agreement between Huang's results and the experiments of Krenzke and Kiernan (18) on shells fabricated to a great degree of precision. Therefore, Huang's analysis can be considered as valid. Nonetheless, it does not explain the large scatter in experimental results.

Three reasons can be given for the poor correlation between theory and experiment: the effect of the boundary conditions, the effect of a plastic zone in the bending area around the shell support, and the effect of initial imperfections. Litle (19) carried out a systematic series of experiments on shallow shells. He came to the conclusion that the support conditions do affect the buckling pressures found. This is due to the large bending zones in shallow shells. Galletta (11) solved the asymmetric elastic-plastic problem using deep shell theory. While yielding in the bending zone does reduce the buckling stress somewhat, this effect is not large, particularly for the thin shells used in most experiments.

The main reason for the poor agreement between theory and experiment is the existence of asymmetric imperfections which trigger the asymmetric bifurcation load at lower pressures then those followed by Huang .This is made plausible by the postbuckling analysis due to Koiter (15). If one starts at an asymmetric bifurcation point B shown in Figure 2 , one can attempt to follow the new equilibrium path by using a perturbation theory. Let the buckled displacements and loading be written as (22)

$$(3) \quad p = p_o\,(1 + a\,A + b\,A^2 + \ldots..); \quad u = u_o + u_1$$

where p_o is the buckling pressure corresponding to point B on the load deflection curve and u_o is the displacement vector at this point. a and b are the constants to be determined; they describe the postbuckling behavior of the shell.u_1 is an infinitesimal displacement vector which can be written as the following series

$$(4) \quad u_1 = A\,\phi_1 + A\,(p - p_o)\,u_{11} + A\,(p - p_o)^2\,u_{12} + \ldots\ldots + A^2\,u_{20} + A^2\,(p - p_o)\,u_{21} + \ldots..$$

A is an infinitesimal amplitude, ϕ_1 is the asymmetric mode shape, and u_{1j} are displacement vectors to be computed. Their computation requires solution of the general linear shell equations subjected to progressively more complicated loadings .The factors a and b require knowledge of these functions : to compute a one must know u_{11} while to compute b one must know u_{11}, and u_{20}.

The value of a for a spherical shell is zero while b is negative. Therefore the postbuckling load - deflection curve is curve I in Figure 2 . There is a fall off in load as the curve is followed.

This analysis is exact and gives some reason for the experimental results. One can surmise that if ϕ_1 already exists even in an infinitesimal form , the load-deflection curve can follow the curve II shown in Figure 2. The postbuckling analysis can be extended to demonstrate this. Instead of the displacement vector given by Eq(3), u is assumed to be

$$(5) \quad u = u_o + u_1 + \epsilon\,V_1$$

where ϵ is an infinitesimal imperfection amplitude and V_1 is the imperfection pattern , taken in the same form as the asymmetric buckling mode, ϕ_1 . u_1 again describes the postbuckling path. A similar perturbation analysis gives the relationships between the buckling load and the imperfection (22)

$$(6)\quad -0.25\,\epsilon\,a = \frac{\left[1-\frac{p}{p_o}\right]}{\frac{p}{p_o}} \qquad (a = 0)$$

and

$$(7)\quad 1.50\sqrt{-3\,b}\,|\epsilon| = \frac{\left[1\ \frac{p}{p_o}\right]^{\frac{3}{2}}}{\frac{p}{p_o}}$$

As can be seen if a is negative there is a decrease in critical load with imperfection amplitude . If a = 0 , which is the case with spherical shells under a uniform pressure, the imperfection sensitivity occurs if the b term is negative . It is negative for spherical shells under internal pressure (8).

The application of Koiter's imperfection sensitive theory leaves open the problem of what happens when the asymmetric response becomes large; the higher order perturbation terms may stabilize the system Therefore, the analysis tends to give conservative results. Another defect in the procedure is the way in which asymmetric imperfections are brought into the analysis,eq(5). Since u_1 and V_1 are considered to be infinitesimal, they do not affect the symmetric response. This cannot be true; the response must be asymmetric from the start since V_1 exists before load is applied. A different approach to the problem of buckling of a spherical shell was carried out by Freskakis (9). It is interesting because it is not restricted to small imperfections.

Freskakis used a nonlinear deep shell theory and incorporated initial imperfections of the form shown in Figure 3. The integer j was taken to correspond to the nonlinear asymmetric buckling mode for perfect shells. Freskakis's equations are too complicated to present here. Suffice to say, the shell response is governed by three simultaneous nonlinear partial differential equations. What is distinctive about the equations is the incorporation of terms arising from the nonlinear geometry due to both the elastic deformations and the initial imperfections; it is not assumed that the imperfections are infinitesimal. Therefore, the symmetric and asymmetric responses are not uncoupled as they are in Koiter's theory.

Freskakis solved his equations by using Fourier series expansions in the circumferential direction. In order to make the computations tractable, in addition to the symmetric response terms, he described the asymmetric response by only a single term in the series, using the same trigonometric term as used in defining the imperfection. This is an approximation because all the asymmetric Fourier series terms are coupled by the nonlinear

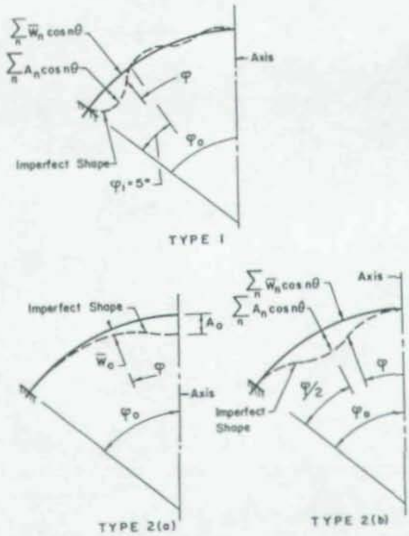

FIGURE 3. IMPERFECTION PATTERNS FOR SPHERICAL SHELLS

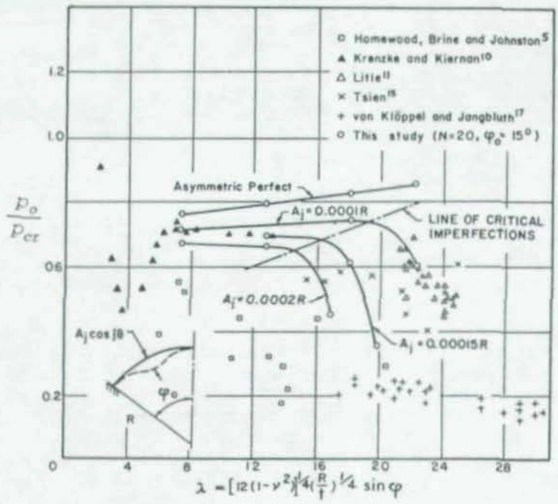

FIGURE 4. EFFECT OF IMPERFECTIONS ON BUCKLING PRESSURE

terms. However, since the equations are coupled and the buckling load must be found by solving the complete load-deflection curve, this assumption does seem reasonable. No attempt was made to follow the declining path of the load deflection. Buckling was taken to occur at the horizontal tangent of the curve. Hence Freskakis' work is not a post-buckling analysis.

A typical set of Freskakis's results is shown in Figure 4 where the results using different size imperfections is superimposed on the experimental results. The imperfection pattern of any spherical shell can be expanded in a Fourier series. If the term corresponding to the asymmetric buckling mode is large, the buckling pressure can be reduced drastically. The question, however, is whether shells are fabricated to this degree of tolerance. There are two ways of looking at the problem. One can adopt the viewpoint taken by most design codes. The tests depicted in Figure 1 were carried out under controlled conditions in laboratories. One cannot expect that fabricated shells will be more perfect than those built in testing laboratories. Therefore, one should base estimates of buckling load on the lower bounds of these test results. Note that if this approach is adopted there is no need for nonlinear analyses at all. One is rejecting all such theory. A second approach is to point out that laboratory shells are, of necessity, very thin, much thinner then would occur in Civil Engineering structures. Therefore, very small geometric imperfections in these shells are large when compared to the shell thickness, but this would not necessarily be the case for actual shell structures. Adoption of this approach makes nonlinear analysis worthwhile, but it also requires data on shell imperfections. In addition, elastic-plastic effects will be more important and should be included in the analysis.

CURRENT STATUS

Some other recent work on spherical shell theory is important for the designer. Yamada(24) carried out an extensive series of tests on stiffened spherical shells under uniform pressure. He also solved the nonlinear equations considering imperfections , somewhat in the manner of Huang.Design equations for stiffened and orthotropic spherical shells under internal pressure are presented in (24).

An interesting conjecture concerning the loss of stability for shells is that it is due to loss of membrane stiffness . Acting on this assumption, one can compute lower bounds to buckling loads for spheres and cylinders (2). The conjecture cannot be completely true because it will lead to absurdly low buckling values in some cases ,but it does seem to agree with experiments for a large range of shell geometries.

There has been a great deal of development in procedures for the nonlinear analysis of shell structures (6,17,20). Nonetheless, much of this work is of very little direct use to the designer. Some available computer codes do provide for a Koiter analysis of postbuckling response (21) so one can use them to determine if imperfection sensitivity is a problem . If it is , and if some data on imperfections is available , a nonlinear analysis using these imperfections can be made to determine the buckling load. These analyses are,however, very time consuming . Examples of this approach are given in the work of Arbocz on cylindrical shells (1) and Galletty on torispherical shells (10).

USE OF DESIGN CODES

The writer can describe some of the difficulties faced by the designer of shell structures . He served as a consultant to Harstead Engineering Associates in their investigation of the stability of the Catawba Containment Shell, shown in Figure 5. This work is reported in detail in (13) so only a sketch of some problems encountered will be presented here. The first questions to be decided on was what approach should be taken to the stability analysis. The ASME Code Case N284 for the design of shells had been published and was available for use. Should it be used, or would it be better to attempt more refined nonlinear analysis based on assumed imperfections similar to a Koiter-type analysis ? While the latter approach is more attractive to the theoretician, and could have been done at the time, it is not one which should be taken in a design such as this. Even though the equations for imperfect shells can be solved, the assumption of any imperfections is always open to question. There is the problem of the reviewing agency, or anyone else, questioning the assumed imperfection pattern and proposing new ones. The review process could then become open-ended. To avoid this prospect, the designer had to use whatever code was available; the ASME Code Case was chosen.

The Code Case essentially requires the designer to compute the linear bifurcation loads for the membrane states due to various service load combinations. A nuclear containment shell is subjected to various thermal, static and dynamic loadings. The membrane stress states found are not symmetric. However, it is the practice to treat the asymmetric stress states as if the maximum stresses were distributed uniformly around the shell. This permits one to employ shell of revolution computer codes which are usually restricted to computing asymmetric buckling modes due to symmetric membrane states. It is of interest to note in passing that many different compressive states arise during a dynamic shell analysis and it is sometimes not possible to know which will yield the lowest buckling load. Therefore, a large number of stress states must be analyzed for stability. This is another reason why nonlinear analyses were impractical.

The linear bifurcation load, p_{cr}, is reduced by plasticity reduction factors, η, and capacity reduction factors, α, so that the buckling load is taken as follows

$$p_B = \eta \, \alpha \, p_{cr} \tag{8}$$

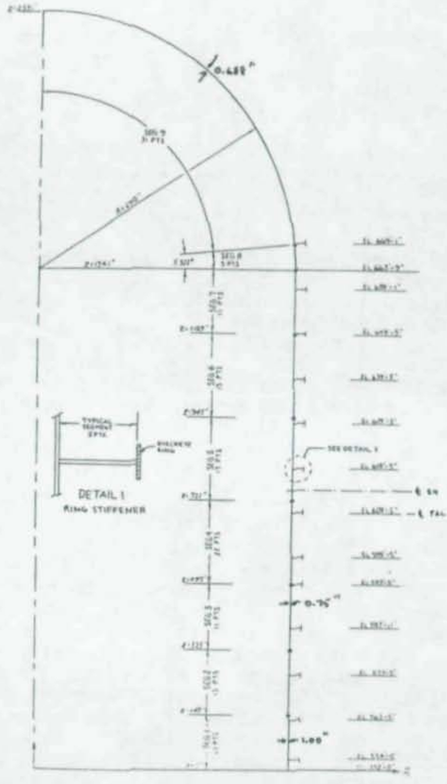

FIGURE 5. CATAWBA CONTAINMENT SHELL

α depends on the shell geometry and the type of loading. As an example, for a spherical shell under uniform internal pressure,

$$\alpha = 0.837 - 0.14\,M \quad if \quad 1.5 < M < 1.73$$

$$\alpha = \frac{0.826}{M^{0.6}} \quad if \quad 1.73 < M < 23.6 \tag{9}$$

$$\alpha = 0.126 \quad if \quad M > 23.6$$

where $M = \frac{L}{\sqrt{Rt}}$. The value of η less than 1.00 did not arise in the design so it is not relevant to the discussion to follow.

It must be emphasized that no factor of safety has been applied to Eq (3). The capacity reduction factor is not a factor of safety. It is more in line with a reliability factor such as used in load and resistance factor design. Eq (3) represents the actual buckling capacity of the shell.

The load capacity reduction factors are based on shell buckling experiments so, if experiments are not available, the values do not exist. This difficulty is shown by the investigation of the spherical dome of the containment shell. There were two critical membrane stress states for the dome (1) $N_\phi = N_\theta = 0.641\ K/in$, (2) $N_\phi = 1.88\ K/in$; $N_\theta = -1.88\ K/in$. The bifurcation buckling values, N_ϕ, for the case of equal unit membrane forces is 11.50 kips/in. for this shell, in agreement with Eq (1). Since the capacity reduction factor is 0.126, the actual buckling value for case 1 is 1.449 k/in. The factor of safety is, therefore, 1.449/0.641 = 2.26. The linear bifurcation value for equal and opposite unit membrane forces is 12.55 kips/in for the shell. This result is, at first glance, surprisingly low in view of the fact that the influence of the tension force is so small. The question, however, is what should the capacity reduction factor be for this case? One would not expect this stress state to be as imperfection-sensitive as the state due to uniform internal pressure. Therefore, the capacity reduction factor should not be as small. However, no value is available. The decision was made to choose the factor by analogy to the case of a cylinder under shear, $\alpha = 0.60$. If this is the factor used, the actual buckling value for case 2 is 0.60 (12.554) = 7.53 k/in. The factor of safety = 7.53/1.88 = 4.00.

The difficulty that arose in this dome analysis occurs quite often. It can arise in the case of thermal buckling where a compressive stress acts in one direction while tension acts in the other. If the capacity reduction factor based on internal pressure is used, the conclusion is reached that even very thick shells can buckle under slight changes in temperature.

If the spherical dome were stiffened with meridional stringers spaced so close together that the shell plate cannot buckle, the designer would again be faced with the problem of choosing the capacity reduction factor. This fact has lead some workers in the field to introduce the split rigidity concept (3,4). Buchert represents the stiffness and shell plate combination by equivalent bending and membrane thicknesses, t_B and t_m. These are then used to describe available data from shell buckling experiments.

A more direct approach to stiffened shells is presented in Yamada's work mentioned above. However, unlike Buchert's work , Yamada's analysis is only valid for spheres under uniform pressure.

RETICULATED SHELLS

Reticulated shells are widely used. Although ,in the past,it was necessary to use a shell analogy to compute the stresses in such structures,it is now possible to carry out complete nonlinear analyses of these structures using available space frame programs (12). Nonetheless, one can expect some problems to arise in their design.The basic problem is that of initial imperfections.

If one did not have knowledge of previous work on shell stability, one might think that design could be carried out by insuring that the individual members of a lattice dome satisfy appropriate design codes for member buckling, and ,in addition, by insuring an adequate factor of safety against overall frame stability. However, if the discrete shell really does behave like a continuous shell, this approach to design is not adequate. The structure might be sensitive to initial imperfections . If this is so,nonlinear analyses will not be sufficient to insure a satisfactory factor of safety against shell buckling.

The basic stability problem in the design of reticulated shells is their imperfection sensitivity. Not all discrete curved domes behave like continuous shells. Therefore the designer must be able to determine when his or her dome does so. Some discussion of this problem is presented in (3); Buchert claims that a reticulated frame behaves like a shell when several members fall inside a wave of the buckling mode shape.This seems to be reasonable. However, more work must be done in this area.

If the discrete system is truly shell-like , it will be sensitive to initial imperfections and the designer must employ capacity reduction factors. There is little published experimental data to go on, so the only available recourse is to employ analogy with the known continuous shell experiments. This approach may lead to very conservative designs.

Another problem which may arise in design of reticulated structures is the need for elastic-plastic analyses if one is to obtain the correct factor of safety.

CONCLUSION

Shell stability is a field of endeavor where sophisticated theories are described by complicated mathematical equations. Yet , for the designer, much of this analysis provides little help. The field is largely semi-empirical. Shell structures do not seem to satisfy our intuition because slight changes in input data cause large changes in output. It might be the only type of structural system where this is the case. However, engineers who work in shell theory are familiar with similar problems. Almost all problems concerning flow in pipes or channels are such that small changes in channel or pipe roughness cause large changes in the flow characteristics.This is usually accepted without question because the empirical results are learned prior to study of the Navier-Stokes equations.

It can be expected that the current trend toward sophisticated analyses procedures for shells will continue. This will be very useful for engineers who have a firm grasp of both theory and experiment. However, if the user lacks either aspect, these computer codes will be of little use and may even be dangerous.

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ADVANCED PROGRESS IN THE DOMAIN OF STRUCTURAL STABILITY

International Cooperation of Stability Studies

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Summary

The paper summarized the historical background of the international colloquia of stability, the development of the general principles of structural safety and the application of general principles of safety to members in danger of instability.

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1. History of International Colloquia on Stability

The historical review begins with year 1944, when the Column Research Council was organized in the United States of America and 1955 when the European Convention for Steelwork (ECCS) was founded. Professor H. Beer, from Austria was, at that time, Chairman of the Committee 8 (Instability) of ECCS.

In Commission 8, Professor J.Dutheil was very influential by insisting with tenacity that the Commission had to abandon design formulae based on the theory of instability by bifurcation of ideally perfect axially compressed bars and could not continue to correct this approach by "varying sentimentally" the safety factor with the value of the slenderness ratio Kl/r, but had, on the contrary, to take into account the random imperfections of these bars and include them in the design formulae.

Two Subcommittees were formed in Commission 8: The first (Buckling experiments) under the leadership of Dr. Sfintesco, launched an extremely large series of experiments on axially loaded columns. Seven countries were involved and about thousand buckling tests have been performed. The second subcommittee, (Theory of buckling) under the leadership of Professor Ch.Massonnet, discussed several years about the possibility to develop analytical computations taking account of the elasto-plastic behaviour of straight columns having a certain crookedness and various distributions of residual stresses. The answer came when using the electronic computer to solve above problem.

Professor Beer and G. Schulz developed at the Technical University of Graz a computer program that was efficient in giving the collapse load. The many (σ_{cr},λ) curves at Graz were compared with the experimental results obtained by the first Subcommittee and became the basis of the so-called "European buckling curves". These curves were then very slightly modified to take account of some new information provided by Dr. B.W. Young in the United Kingdom.

Professor Beer had the idea to organize in 1971 an international Colloquium in order to compare the ECCS approach of buckling curves with those prevailing in Eastern Europe, United States of America and Japan. After the sudden death of Professor Beer, in 1972, it was decided to hold above Colloquium in Paris [1].

In 1973, as a result of the links of some of its members with the Column Research Council of the United States, Commission 8 was reorganized and divided in nine task groups.

With the gradual development of rules for designing against instability emerged, in London, in 1974, the idea to hold an International Colloquium treating every aspect of structural instability of steel structures. Dr. Sfintesco and Professor L.S. Beedle proposed to enlarge the geographical scope of the Colloquium and transform it into a "Travelling Colloquium".

Starting with the first one in Paris in 1972, and with the last one in Istanbul in 1991, there have been 13 International Colloquia Stability Sessions around the world. Each session was organized locally with co-sponsoring organizations from other countries participating.

Table 1. provides a summary of these 13 sessions with references [1]-[21] documenting the associated publications resulting from the conferences.

It is interesting to note that these conferences resulted in 29 volumes of technical papers contributing to substantial technology transfer around the world.

In 1982 the first edition of the "World View" [22] and in 1991 the second edition of the "World View" [23] evaluated specifications and codes, compared and contrasted them, and explored some of the major reasons for their differences.

In 1995 and 1996 the next sessions of the "Travelling Colloquia" will focus on "The *Future Direction for Stability Research and Design". This Fifth Colloquium is intended to* culminate to work of the first Four Colloquia, comprising 29 sessions. The goals of the Colloquium are to consolidate the status of available knowledge and to plane and prioritize the need for future research.

2. General Principles of Structural Safety

The increasingly powerful experimental and computational tools of structural design require well-defined design philosophy. As its basis seemingly the concept of limit states is accepted in many countries, requiring the check of the (small enough) risk that the given structure be brought in its ultimate state (failure) and the (somewhat bigger) risk of the occurrence of phenomena restricting its regular use (serviceability).

The design methods may be distinguished by the type of safety conditions: ***deterministic design methods,*** in which the basic parameters are treated as non-random; ***probabilistic design methods,*** where basic parameters are considered as random.

Table 1
INTERNATIONAL COLLOQUIA ON STABILITY

International Colloquium	Dates	Location	Conference Organizer	Volumes of Pub.	Ref. Nums.
1st	Nov 23,24, 1972	Paris	IABSE	1	1
2nd	Sept. 9, 1976	Tokyo	ECCS	3	2,3,4
2nd	April 13-15, 1977	Liege	ECCS IABSE	2	5,6
2nd	May 17-19, 1977	Washington, D.C	SSRC	1	7
2nd (1st Hungarian Int. Conf.)	Oct. 19-21, 1977	Balatonfüred	Technical University of Budapest, Hungarian Academy of Sciences	2	8,9
3rd	Oct. 16, 1982	Timisoara	The National Council of Engineers, Romania, Polytechnic Institute of Timisoara	2	10,11
3rd	May 9-11, 1983	Toronto	SSRC	1	12
3rd	Nov. 16,17, 1983	Paris	Center Technique Industrial de la Construction Metallique	2	13,14
2nd Hungarian International Conference	Sept. 25, 26, 1986	Tihany	Technical University of Budapest, Hungarian Academy of Sciences	5	15,16
4th	April 17-19, 1989	New York	SSRC	3	17,18
4th	Oct. 10-12, 1989	Beijing	Tsinghua University, Institute of Structural Stability and Fatique (China), China Steel Construction Society	1	19
4th (3rd Hungarian Int. Conf.)	April 25-27, 1990	Budapest	Technical University of Budapest, Hungarian Academy of Sciences	5	20
4th	Sept. 16-20, 1991	Istanbul	Bogazici University, Istanbul Technical University, SSRC	1	21

2.1. Historical remarks

The Safety Concept, as adopted by most East European countries was originally worked out and formulated firstly in Germany at 1926 by Mayer and mostly by Soviet experts in the 30's and 40's of this century (see Baldin 1951).It was officially introduced first by the Soviet Building Code in 1946. Similar concepts were adopted in Hungary and Poland around 1950. This initiatives had great influence on the work of other international organizations (ISO, CEB) in their standardizing activity as well.

A governing idea of the above mentioned specifications was to modify the main features of design philosophy common for structures of different materials (reinforced and prestressed concrete, steel, aluminium, brick, timber), ensuring thus a uniform attitude in formulating their design rules.

The background, development and details of this process are summarized in several monographs (Mayer 1926, Gvozdev 1949, Baldin 1951, Broude 1953, Streleckij 1952, Korányi 1952, Bolotin 1965, Murzewski 1970).

The Safety Concept - called by different names as "limit states approach", "method of divided safety factors", "semi-probabilistic method" and lately as "*level I.* method in a probabilistic approach", indicating thus the different stages of it's development - was originally offered as an alternative instead of the previously generally adopted "allowable stress approach" and was motivated by it's criticism. Based on contemporary comments (Baldin 1951) this can be summarized as follows.

2.2. Limit states

With broadening knowledge about performance of steel structures in the inelastic range and reinforced concrete elements in their different working stages, it seemed necessary to re-interpret the original and newly accumulated ideas hidden behind the design formulae of the allowable stress approach:

$$\sigma(F_K) \leq \sigma_w; \qquad e\,(F_K) \leq e_w \tag{1}$$

$$(a)\;\; \sigma_w = \sigma_y / n_1; \;\; (b)\;\; \sigma_w = \sigma_{cr} / n_2; \qquad (c)\;\; \sigma_w = \sigma_f / n_3; \tag{2}$$

expressing comparison of stresses (and deflections) due to code-specified loads F_K to their allowable values σ_w (and e_w), the former one derived from yield stress σ_y, critical stress σ_{cr} and fatigue strength σ_f by safety factors n_1, n_2, n_3 respectively. First of all Eq. 2b proved to be problematic, as it represents *in some cases* the requirement of elastic behaviour under normative loads - giving thus but an indirect and often irrelevant information about margin of safety (Kazinczy 1914) and bearing little relevance upon actual behaviour of reinforced concrete elements (Gvozdev, 1949), - *in other cases* (bearing in mind the design practice of trusses, connections and alike) it includes at least quantitatively the consequences of ductile behaviour and refers directly to failure. To avoid ambiguities and unreasonable differences in actual safety it was suggested

- to take systematically into account all known phenomena rendering the structure unsuitable to satisfy anticipated design requirements, bringing it thus to a "limit state",

- to select parameters Φ^i (loads, load-effects, stresses, deflections and so on), suitable for quantitative description of the limit states by their values $\Phi_L{}^i$ depending on geometrical, strength and stiffness characteristics of the structure,

- to adopt adequate computational methods to monitor the changing values of $\Phi_F{}^i$ of this parameters in the loading process, and

- to judge safety by comparison of the most unfavourable value of $\Phi_F{}^i$ during erection and life-time to $\Phi_L{}^i$, considering the consequences of reaching the limit states.

Two categories of limit states were suggested (for steel)

A. Ultimate limit states (rendering the structure unfit for further use), caused by phenomena as
- loss of equilibrium as a whole,
- instability,
- fracture (brittle or due to fatigue),
- formation of a collapse mechanism,
- substantial change in geometry,
- other phenomena (excessive yielding, slip in connections, and alike) making further functioning impossible.

B. Serviceability limit states (restricting regular use of the structure), caused by phenomena as
- excessive deformations, deflections,
- excessive vibration.

Most attention was paid to category *A.*, where in general the load F and carrying capacity R can be regarded as parameter $\Phi_F{}^i$ and $\Phi_L{}^i$ respectively.

2.3. Divided safety factors

Progress in understanding structural behaviour and in reliability of material was manifested by a successive increase of allowable stresses (as for instance the raise of σ_W for mild steel from 140 MPa to 160 MPa during the second world war in the Soviet Union). It was of common opinion that further general increase would jeopardize safety and further progress can be achieved by differentiating among structures and loading cases only. As main view-point of differentiation (within a certain class of structures and materials) differences in variability of loads offered themselves; reflecting the general experience, that structures with high proportion of (unchanging) selfweight are less vulnerable and superior in longevity, than light structures with high proportion of strongly varying live or climatic loads. This motivated the early Soviet specifications for reinforced concrete to apply different safety factors depending on the ratio of dead load to live loads, and subsequently to the splitting up general safety factors in Eqs. 2. into two parts:

$$n = \gamma_F \cdot \gamma_m \quad \cdot \tag{3}$$

the first one expressing uncertainties due to loads, the second one those due to resistance.

2.4. Semi-probabilistic approach

In this relation it was necessary to clear up the actual meaning of traditional terms as "characteristic" (normative, code-specified, nominal, guaranteed) values of load (F_K) and resistance (R_K). This brought about the concept (Mayer, 1926; Chocialov 1929; Streleckij 1935, 1947; Wierzbicki 1936), that they can be interpreted by regarding load and resistance as random variables, characterised by a probability density function, mean values $(F_{max})_m$, R_m; standard deviation s_F, s_R; and coefficient of variation δ_F, δ_R; respectively (Fig.1).

Following the interpretation of Rzanicin (1947, 1949, 1954), the risk of failure (P_f) can be expressed by the probability

$$P\{R-S<0\}=P_f \tag{4}$$

(shadowed area in Fig.1.b), which - in case of Gaussian distribution - depends on safety index:

$$\beta=\frac{(R-F_{\max})_m}{s_{R-F}}=\frac{R_m-(F_{\max})_m}{\sqrt{\left[R_m\times\delta_R\right]^2+\left[(F_{\max})_m\times\delta_F\right]^2}} \tag{5}$$

allowing to establish correlation between risk of failure and "central" safety factor

$$n_C=\frac{R_m}{(F_{\max})_m}=\frac{1+\sqrt{\beta^2(\delta_R^2+\delta_F^2)+\beta^4\delta_R^2\delta_F^2}}{1-\beta^2\delta_R^2}. \tag{6}$$

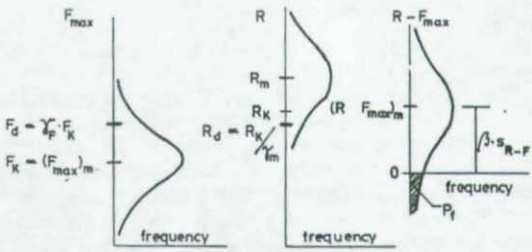

Figure 1.

This can be - somewhat arbitrarily and approximately - split up:

$$n_C \sim n_F \cdot n_R; \qquad n_F=1+\bar{\beta}\delta_F; \qquad n_R=\frac{1}{1-\bar{\beta}\,\delta_R}. \tag{7}$$

and thus - in case of arbitrarily chosen characteristic values - by

$$\gamma_F = n_F \frac{(F_{\max})_m}{F_K}; \qquad \gamma_m = n_R \frac{R_K}{R_m} \tag{8}$$

the basic design formula can be established:

$$\gamma_F F_K \leq \frac{R_K}{\gamma_m} \text{ ; or } \gamma_F F_K = F_d; \qquad \frac{R_K}{\gamma_m} = R_d; \quad F_d \leq R_d \tag{9}$$

In case of combined loading - adopting different γ_{Fj} values according to different coefficients of variation of the components

$$\sum_j \gamma_{Fj} F_{Kj} \leq \frac{R_K}{\gamma_m}; \tag{10}$$

or more generally:

$$\Phi_F^i(\gamma_{Fj} F_{Kj}) \leq m \Phi_L^i(\sigma_y / \gamma_m, A) \tag{11}$$

where m represents correction factor for special circumstances in fabrication, tolerances, structural behaviour and A stands for geometrical quantities.

2.5. Resistance factors

Originally (Baldin, 1951) $\overline{\beta} = 3$ was suggested. For characteristic value of yield stress usually

$$(\sigma_y)_K = (\sigma_y)_m - 2s_\sigma \tag{12}$$

is chosen, equal approximately to traditional "guaranteed" minimum value. Thus for most steel grades

$$\gamma_m = \frac{(\sigma_y)_m - 2s_\sigma}{(\sigma_y)_m - 3s_\sigma} \sim 1.1 - 1.2. \tag{13}$$

For ultimate tensile stress $\gamma_m \sim 1.45 - 1.6$.

2.6. Load factors

The characteristic value of loads is chosen as the mean value of maxima:

$$F_K = (F_{\max})_m. \tag{14}$$

F_{max} being the greatest load on individual structures of the population or similar structures, or the maximum load within certain characteristic time interval (for instance one year for snow load). Having very few data about δ_F, (Kármán, 1965), load factors were mainly estimated using as measure data of previous specifications and existing structures. Recently most specifications adopt

$$\gamma_F = 1 + 1.65\,\delta_F \tag{15}$$

or in case of non-Gaussian distribution the value $F_d = \gamma_F\, F_K$ characterized by being surpassed with probability less than 5 %.

Examples for γ_F values for limit states *A*:

- Dead load: $\gamma_F = 1.1$ (0.9); exceptionally 1.2.
- Live load, if intensity q N/m2:
  - $q \le 2000$: $\gamma_F = 1.4$,
  - $2000 < q \le 5000$: $\gamma_F = 1.3$,
  - $q > 5000$: $\gamma_F = 1.2$.
- Snow load: $\gamma_F = 1.4$
- Wind load: $\gamma_F = 1.2$

For limit states *B*: $\gamma_F = 1$.

2.7. Load combinations

To express the reduced probability of coincidence of maxima of more components in a loading process, loads are categorized as

- Permanent loads (self-weight, earth pressure),
- Variable loads and actions (live load, climatic loads, imposed deformation), subdivided wholly are partially to
  - sustained (long-term) loads or load-parts,
  - instantaneous (short-term) loads or load-parts.
- Catastrophe loads (seismic loads and alike).

Two load combinations are suggested:

- Regular combination consisting of permanent and live loads. Including more than one short-term load, combination factors $\gamma_C = 0.9$ (three short-term loads) or $\gamma_C = 0.8$ (more than three short-term loads) are to be applied.
- Irregular combination, including one catastrophe load. Combination factor $\gamma_C = 0.8$ to all short-term loads can be applied.

2.8. Structural safety of standards and codes

There are many approaches to the probability-based methods of evaluating the reliability of structures, and these are discussed in a number of textbooks (see, e.g., Bolotin, 1965; Benjamin and Cornell, 1970; Hart, 1982; and others). The most useful of these methods are those which use only the mean and standard deviation of the random parameters of design (Ang and Cornell, 1974). These are called "first-order second moment" (FOSM) methods. The FOSM methodology is very useful in structural applications because complete information on the distribution properties of the structural design variables is not usually available. Such methods are applied extensively to develop a new generation of structural design specifications (CEB, 1978; Ravindra and Galambos, 1978; Galambos, 1978; AISC, 1986; CSA, 1974). These new specifications retain the traditional format of limit states design (LSD) with resistance factors and load factors, but these factors are determined by probabilistic principles (Galambos et al., 1982; ANSI, 1982).

EUROCODE 3 adopts modern principles in matter of structural safety based on probabilistic concepts of safety within the framework of a level 1 reliability code format through the use of partial safety factors applied to the load effects derived from a proper structural analysis and to the design resistance. The method of checking structural safety envisaged in EC3 refers to limit states and does not anymore refers to the traditional allowable stresses concept (Brozzetti and Janss, 1992).

2.9. Trends in research

Above specifications allow without giving detailed rules - to use probabilistic design procedures. This means a return to the basic idea outlined in connection to Eqs. 4, 5, 6; to check safety directly by the risk of failure P_f, safety index β ; or using limiting values for central safety factor n_c (Szalai, 1974); or prescribing limiting values $\Phi_L^{\,1}$ depending on coefficients of variation both of resistance and loads. Actual application needs further research, for instance:

- To find theoretically well-founded probability density functions for different steel-grades (Mistéth, [9]) and for loads.

-To find appropriate probabilistic procedures for non-linear (instability) problems *(Murzewski, [8], Bolotin 1972), and gather statistical information about imperfections.*

- To complete probabilistic approach by regarding the structure as an assemblage of elements and concentrate on safety of the whole instead of that it's separated parts (which can differ substantially in both safe and unsafe sense) (Bolotin 1965).

- To find compromise between requirements of safety and economy in the choice of risk of failure, or safety index (Rzanicin 1961, Murzewski 1970, Mistéth 1974).

2.10. Reliability considerations

The limit states approach is - in its traditional form - mainly concerned with failure due to exceptionally high loads and for this sake it suffices to "condense" the loading history into a probability density function (Fig.2.a), containing no information about actual number of load-cycles or time-effect (except that longer life-time involves unfavourable F_{max} distribution).

Some limit states - as those caused by fatigue, accumulation of damages, residual deformations - are closely connected to actual loading history, and so instead of "safety" rather "reliability" should be mentioned, interpreted as probability of survival of the structure P(t), depending on required time of operation (Fig.2.b) (Bolotin, 1965).

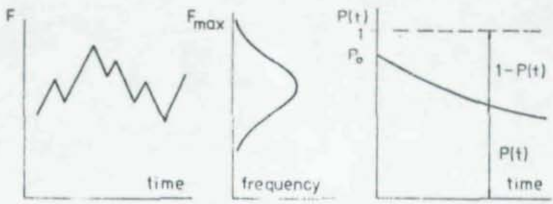

Figure 2.

The way of thinking can be characterized - following Bolotin's (1972) interpretation by Fig.3: from the (stochastic) loading history - described by f(t) - we conclude to structural response u(t) and further to quality parameters v(t); and reliability is given by the probability, that v(t) stays in the region marked out by the "limit surface" Γ.

In this sense the procedure of fatigue analysis according to the specifications - using mostly a load combination of "service loads" and limiting stresses for fatigue depending on load-cycles - needs further refinement.

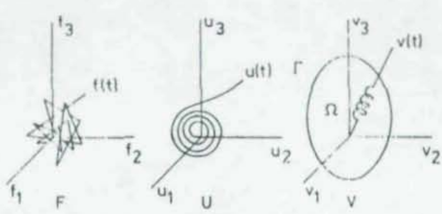

Figure 3.

Economical considerations - as the choice of safety index β - can be based on reliability as well, containing time-dependent elements (as amortization). In this sense optimization using cost-function of the form (Bolotin 1965)

$$C = C_0 + C_1(P)(1-P) \tag{16}$$

may give answer, establishing an important correlation between operational life and required safety.

3. Application of the General Principles of Safety to Members of Steel Structures

In case of steel structures the designed ideal structure differs from the one completed according to the plans, and this difference can be characterized by numerous larger and smaller defects, irregularities, imperfections.

The equilibrium of the designed structure is stable if small imperfections and defects cause sufficiently small differences with respect to the ideal operational conditions.

If small imperfections cause disproportionately big differences the equilibrium is unstable.

The term stability (instability) represents the relation between disturbing factors and the requirement due to them.

Stability condition in case of structures can be expressed as follows: the equilibrium condition of ideal structure - during operation - must be stable against all these disturbances that differentiate the ideal structure from the real one. This way stability has a close relation with the choice of the calculation method, the calculation model.

(A) ***The traditional stability analysis*** methods assume ideal models and determinate bifurcation (equivalent effective length).

Let's assume that the calculation model is homogeneous, perfectly straight, concentrically loaded bar (Figure 4.a).

The ideal model differs from the real one in the initial out-of-straightness and the eccentricity of the compression force. The statement, that straightness of bar (ideal model) is stable in regard with the above differences (disturbances), is equivalent to the statement, that by limiting the initial out-of-straightness and eccentricities under adequately small value of displacements perpendicular to the axis of bar be less than a given value. If compression force N is smaller than the Euler force N_E the straight bar is stable. If $N>N_E$ the least disturbances might cause infinite displacements.

(B) Trials with the goals to demonstrate the loss of stability with bifurcation (buckling, lateral buckling, plate buckling) have the information that theoretically assumed behaviour (bifurcation) can be achieved only with specimen (perfectly straight bar, perfect cylinder or sphere, etc.) made with extreme care; behaviour of civil engineering structures might be significantly different. Inverse of this basic statement hints that in cases where analysis of model for such exaggerated abstractions (simplifications in structural geometry, conditions referring to the symmetry of arrangement and loading), from which the least deviation (small geometric defects of structure, small asymmetry, etc.) - at least in certain load spectrum - can lead to a significant deviation in the assumed and real behaviour of the structure to be modelled. Aim of the up-to-date stability analysis - using the before mentioned term for stability of equilibrium - is the limitation of application of the above mentioned, too much simplified ideal models and the searching for such - real models (so-called *imperfect models*) that can reflect the generally negligible, but in given cases (so-called disturbing) effects of significant consequences.

Therefore ***up-to date analysis*** specifications are based on the analysis of real models. In case of buckling and lateral buckling analysis slightly crooked and eccentrically loaded bars with residual stresses, while at buckling of plates and shells surfaces with geometrical imperfection are to be taken into consideration.

Let's assume that initial crookedness and eccentricity are given and calculation is carried out on the model of crooked and eccentrically loaded bar (Figure 4b). If the problem is expressed this way stability problems of the straight bar in respect with the given disturbances are eliminated and are replaced by determination of stress and deformation states by a more accurate calculation model. (However, in this case such a new problem may occur that analyzes the stability of this state in respect with certain new classes of disturbances.)

Application of the general principles of safety will next be illustrated as it applies to axially loaded columns to demonstrate its use in the development of design criteria for instability.

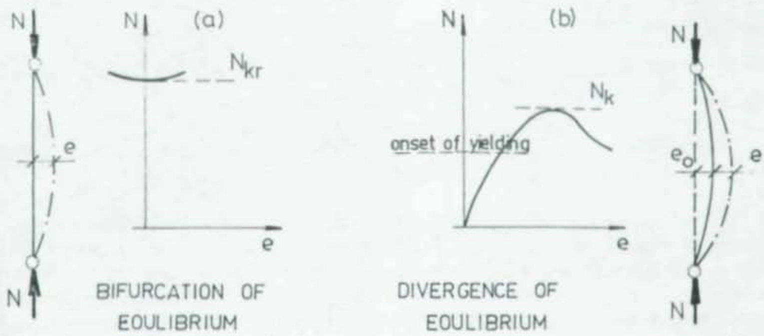

Figure 4.

3.1. Experimental Data-Base Approach for Column Curves

For the preparation of design codes and for the verification of new theoretical approaches for predicting the resistance of structure, test data relating to the behaviour of steel structures and structural components throughout the entire range of loading up to ultimate load are an essential requirement.

Prof. Y. Fukumoto and Y. Itoh (1983) have been developing a numerical data-base for the ultimate strength of steel structures. Following this Prof. Fukumoto and Itoh initiated an extensive survey of column tests and stored the test data on a Numerical Data-Base for Steel Structures (NDSS). After various computational and statistical manipulations, the information on steel column tests have been evaluated and comparisons made statistically between the surveyed test data and the EC and SSRC multiple column curves.

The basic column strength formula in the EUROCODE 3 and Hungarian Code (MSZ) is compared graphically with the experimental curve (Figure 5.).

The limit load N_c of a centrally compressed steel member of uniform cross section governed by flexural buckling is given in the form

$$N_c = A \cdot \chi \cdot \frac{\sigma_y}{\gamma_m} \tag{17}$$

A denoting the cross-sectional area, χ buckling factor for column and σ_y/γ_m the specified yield stress.

3.2 Mathematical Expression of the Buckling Curves (Maquoi, R. and Rondal, J., 1978)

A two-pinned member possessing a sinusoidal imperfection of amplitude e_0 and subjected to a normal load N, is deflected halfway along by a moment of the second order given by the relation:

$$M^{II} = \frac{N\ e_0}{1 - N / N_{cr}} \tag{18}$$

where $1/(1\text{-}N/N_{cr})$ represents the coefficient amplification of the deflection e_0, N_{cr} being the Eulerian buckling load of the member. The ultimate load N_K of such a column is reached when the normal stress σ in the most loaded fibre reaches a maximum value σ_{max}, which is written:

$$\frac{N_k}{A} + \frac{M^{II}}{W} = \sigma_{max} \tag{19}$$

where A and W are respectively the area and the section modulus of the cross-section of the member. Assuming that the maximum stress σ_{max} is equal to the yield stress σ_y of the material, and designating by σ_k and σ_{cr}, respectively, the ultimate stress and the Eulerian buckling stress , the relation (19) is written:

$$\sigma_k + \frac{\sigma_k e_0}{1 - \sigma_k / \sigma_{cr}} \frac{A}{W} = \sigma_y \tag{20}$$

or, in a different form:

$$(\sigma_y - \sigma_k)(\sigma_{cr} - \sigma_k) = \eta\ \sigma_k \sigma_{cr} \tag{21}$$

where the parameter $\eta = e_0\ A/W$ represents the effect of the only geometric imperfection of the member.

One can imagine for the sake of simplicity, keeping the general expression above but representing by the factor η, not only the effect of a geometric imperfection but also that of the distribution of residual stresses - due to any rolling, cold straightening, welding and flame cutting which might have occurred - and the spreading of the values in the section.

By setting $\eta = \alpha(\bar{\lambda} - 0.2)$, the mathematical expression

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 + \bar{\lambda}^{-2}}} \tag{22}$$

where: $\Phi = 0.5\,(1 + \alpha\,(\bar{\lambda}-0.2) + \bar{\lambda}^2)$

In Figure 5. the column curve b ($\alpha = 0.35$) is showed.

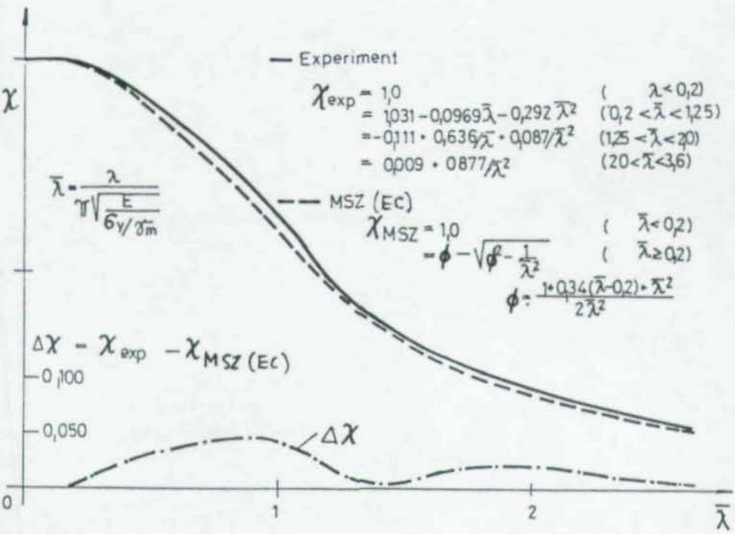

Figure 5.

3.3 Reliability of Axially Loaded Columns

3.32 Load and Resistance Factor Design Curve

As a result, the AISC Load and Resistance Factor Design (LRFD) Specification adopts the following curve

$$\frac{P}{P_y} = \begin{cases} \exp\left[-0.419\,\lambda_c^2\right] & \lambda_c \le 1.5 \\ 0.877\,\lambda_c^{-2} & \lambda_c > 1.5 \end{cases} \tag{23}$$

to represent column strength. Note that only one curve is recommended for the whole range of possible column strengths. In the development of this curve, the following assumptions were made:

1. The column has small end restraints corresponding to an end-restraint parameter G =10 or an effective length factor K = 0.96.
2. The column has an initial crookedness sinusoidal in shape and has an amplitude of (1/1500)L at midheight.
3. The axial force is applied at the centroid of the column end cross sections.

This LRFD curve is plotted in Fig. 6 together with other curves.
Note that the LRDF column curve as represented by Eq. (23) is comparable to SSRC Curve 2, especially in the range $0 \le \lambda \le 1.0$. The LRFD format is

$$\Phi R_n \ge \sum_{i=1}^{m} \gamma_i Q_m \tag{24}$$

where R_n = nominal resistance
Q_n = nominal load effects
ϕ = resistance factor
γ = load factor

Note that the LRFD format has the features in that factors of safety are applied to both the load and resistance terms to account for the variabilities and uncertainties in predicting these values. Furthermore, these load and resistance factors (ϕ, γ) are evaluated based on *first-order probabilistic approach*. Since different types of loads have different degrees of uncertainties, different load factors are used for different types of loads (e.g., 1.6 for live load, 1.2 for dead load, etc.); therefore, the LRFD format represents a more rational design approach.

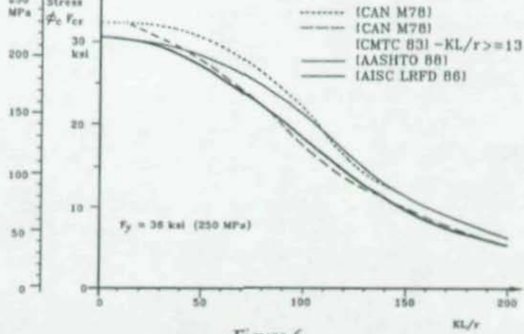

Figure 6.

3.3.2 Quasi-Probabilistic Formulae of Buckling Curves

Formulae of that kind are sometimes attributed to Rankine and Merchant. Some historical remarks may help to follow the development of the idea of a uniform approach to elastic-plastic buckling.

a) W.J. Rankine in 1858 gave an empirical formula for strength R_u of real columns

$$1/R_u = 1/R_{pl} + 1/R_{cr} \tag{25}$$

b) W. Merchant proved hundred years later that the formula (25) may be applied to frames and it is always conservative in comparison with the known experimental results. In this case R_{cr} is the critical carrying capacity of a perfectly elastic frame and R_{pl} is relative to the rigid-plastic frame of the same configuration.

c) Murzewski has considered a random ultimate compressive load of a column

$$R_u = \min(R_{pl}, R_{cr}) \tag{26}$$

He assumed the Gauss probability distributions for R_{pl} and R_{cr} in 1973 and one year later - the Weibull distributions. So a formula for the characteristic minimum $\breve{R}_u$ is derived

$$\sqrt[\nu]{1/\breve{R}_u} = \sqrt[\nu]{1/\breve{R}_{pl}} + \sqrt[\nu]{1/\breve{R}_{pl}} \tag{27}$$

where ν - the Weibull coefficient of variation. The Rankine-Merchant formula (25) is a particular case of (27) when $\nu = 1$. But statistical tests indicate that ν is less than 1 and it depends on slenderness ratio λ of the member. The formula (27) with $\nu = \nu(\bar{\lambda})$ has been taken into consideration recently for the level-2 probability-based design.

d) The formula (27) is transformed as follows

$$R_u = R_{pl} / \sqrt[n]{1 + \bar{\lambda}^{2n}} \tag{28}$$

where $\bar{\lambda} = \sqrt{R_{pl}/R_{cr}}$ and $n = 1/\nu = \text{const}$.

The formula (28) may be considered as an extended Rankine-Merchant formula. It was recommended to checking of lateral buckling of beams by ECCS recommendations (1975). The relative slenderness ratio $\bar{\lambda}$ in formula (28) is modified so that both R_{pl} and R_{cr} are taken with equal values of central safety factors $\bar{\gamma}_R$, i.e. safety factors relative to the mean values $\bar{R}_{pl}$, $\bar{R}_{cr}$.

e) Z. Mendera [20] (Vol. I, pp. 33-42) extends the quasi-probabilistic formula (28) to the cases when the critical resistance R_{cr} of a structural member (a bar or shell) is defined by either theoretical or empirical formula for slender members.

3.4. A Unified View on the Strength of Columns through Catastrophe Theory

Niwa, Y., Watanabe., Isami, H. (1983) [13] presented a principle to determine the realistic load-carrying capacity of steel columns, beams and plates as related to the structural stability on the basis of catastrophe theory (Thom 1972) and discusses the effect of the initial out-of-plane deflection. It is possible to present the ultimate strength or the imperfection sensitivity on the basis of the "fold" bifurcation set.

Ultimate strength of the actual members is affected not only by the elasto-plastic buckling load and the elasto-plastic postbuckling path, but by the initial imperfections, and is controlled by the plastic unloading curve corresponding to the failure mechanism.

Now, the equivalent bifurcation point may be defined by the intersection (w^*,σ^*) of the elasto-plastic postbuckling equilibrium path with the plastic unloading curve, that is, the failure mechanism. (Figure 7.)

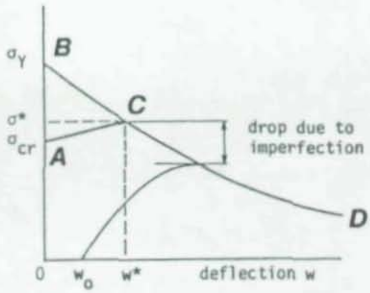

Figure 7. Equivalent bifurcation point

Figure 8. illustrates the equilibrium space M_V in 3-dimensional space $(w\text{-}w^*, w_0\ ,\ \sigma)$, and its projection to three orthogonal planes.

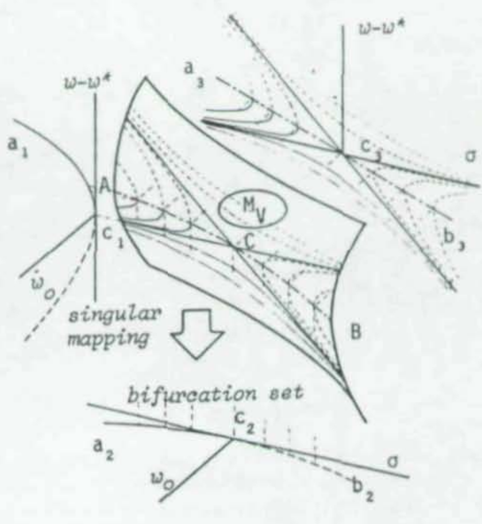

Figure 8. Equilibrium space M_V and singular bifurcation set

Therefore, the ultimate strength σ_m of the imperfect member can be evaluated in terms of the bifurcation set. The bifurcation set can be defined graphically by the curve a_2c_2 in the σ - w_0 space projecting vertically the singular fold line AC on the equilibrium surface M_Y. It takes the following form of the 1/2-power of the imperfection:

$$\frac{\sigma_m}{\sigma^*} = 1 + \alpha^* w_0 - \sqrt{2\alpha^* w_0 (1 + \alpha^* w_0/2)} \tag{29}$$

Several numerical examples are demonstrated to evaluate the strengths of columns, beams, and the compressed plates. From these examples, the ultimate strengths are found to be reasonably well expressed in terms of the bifurcation set characterized by the 1/2-power of the imperfections.

Fig. 9. compares the proposed bifurcation set with several column strength curves such as the elasto-plastic buckling, the ECCS, Perry-Robertson and the interaction curves for weak-axis buckling of columns.

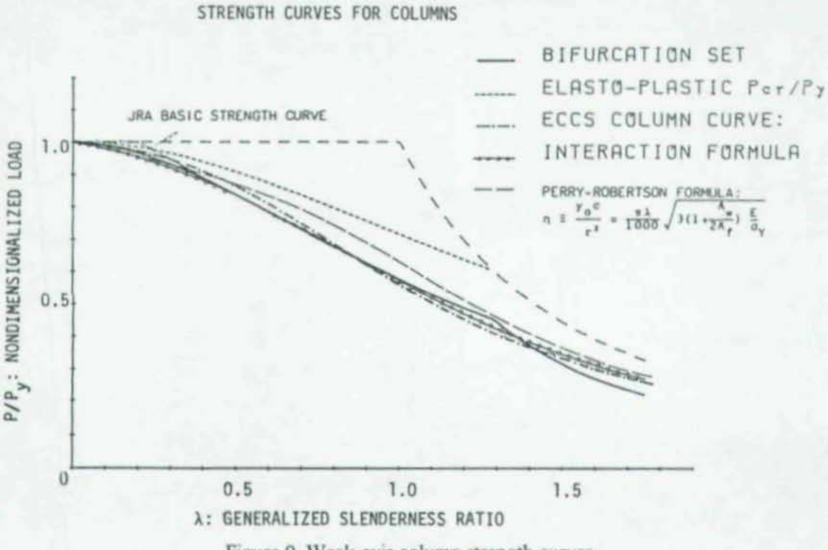

Figure 9. Weak-axis column strength curves

4. Final remarks

It seems to be a common attitude of modern specifications to set growing requirements for the designer: to look at his structure more and more in its real complexity - starting with the way and unavoidable tolerances and defects of fabrication, going through the mode of erection with its inherent imperfections, ending with the performance under regular and exceptional loading conditions, thus accounting for a series of factors of random character, governed by circumstances beyond his authority. So he has to regard his structure on the drawing table as a member of a multitude of similar structures differing slightly in form, strength, loading, - not identified in the stage of design - and so defining safety supposes the use of statistics and the laws of the theory of probability and reliability.

Additionally analysis of structural response is growing more sophisticated; specifications have to incorporate an increasing number of design rules, sometimes in form of tables and diagrams (often derived directly from experiments), making specifications voluminous, the number of their appendices increasing.

Although we are in possession of all facilities to carry through any complicated calculations, it seems necessary to regard them mainly as a tool for research and to find a good balance between the real needs of a safe and economic everyday design praxis and the way of feeding back the accumulating information gained by research.

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STABILITY ANALYSIS FOR COMPOSITE AND HYBRID STRUCTURES FOR SEISMIC LOADS

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INTRODUCTION

Over the past three decades composite and hybrid structural systems have provided the lateral stability for many tall landmark structures such as the Bank of China in Hong Kong [1], Two Union Square in Seattle, First City Tower in Houston, and Norwest Center in Minneapolis. Composite systems incorporate members in which steel and concrete act together through mechanical interlock, friction and adhesion. Hybrid structures, on the other hand, contain combinations of steel, concrete and composite members as part of their main lateral load resisting systems. In the rest of this paper both hybrid and composite systems will be referred to as "composite" since their design shares many similarities.

Composite construction is an area where practice has consistently progressed faster than research and code writing, and where the designer's ingenuity and structural concepts are at the forefront in the design process. This is particularly true for high rise construction in areas where large seismic loads or hurricane winds govern the design. The design approach is based on visualizing alternate load paths, providing ample strength capacity in both key members and connections, and careful detailing to insure ductile behavior.

Most of the use of composite structural systems has been in situations where the lateral stiffness of the building is provided by a few large members. A typical example is the structural system for the 60-story Norwest Center in Minneapolis designed by CBM Engineers (Fig. 1). The lateral load resisting system consists of four very large composite columns tied together by 5-story Vireendel trusses in the North-South direction and by concentric braces in the East-West direction [2]. This system was dictated by the large number of changes in floor plan and the resulting need to concentrate the lateral load resisting system in the tallest part of the building.

Because (1) there is little analytical and experimental data on which to base the design of structures with very large composite members and connections, and (2) the design is generally governed by drifts at the service loads, these building are consistently overdesigned for strength. Much of the conservatism stems, in addition, from the need to provide simultaneously ductility and toughness to resist large, not well-characterized seismic and wind forces.

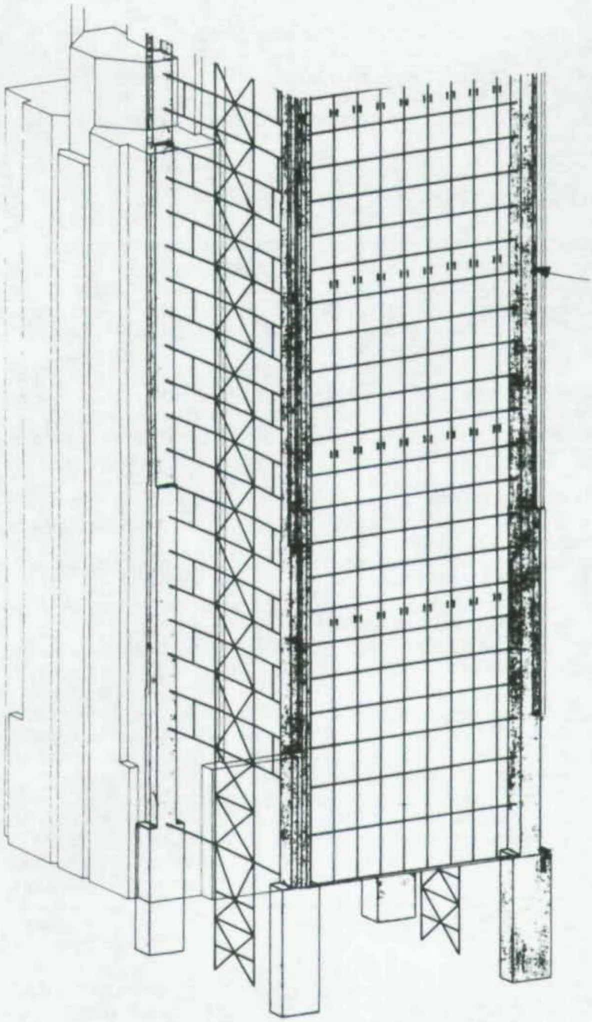

Figure 1 - Structural system for the Norwest Center.

The design of both composite members and systems has reached the stage where they are recognized as separate from steel and concrete systems. There are currently two sets of model codes for buildings, the NEHRP [3] and the Eurocode 4 [4], which treat composite design separately. The documents offer some striking similarities and differences. The NEHRP document, which is strictly speaking a "resource document" and not a model code, deals primarily with structural systems and connections. It is a short document intended to highlight structural systems for use in zones where seismic loads govern the design. It concentrates on describing the distribution of forces and makes reference for most of the detailing to existing provisions in the ACI [5] and the AISC new LRFD specification for seismic design [6].

The Eurocode, on the other hand, is much longer, more detailed, and concentrates on member design for the usual gravity and lateral loads. The Eurocode 4 does not deal with seismic design since that is part of Eurocode 7. Both documents, however, are aimed at promoting the use of composite systems in both low- and high-rise construction. This paper summarizes the NEHRP provisions for stability while the Eurocodes approach to this problem is described in another paper in these proceedings [7].

BACKGROUND TO NEHRP

There is a wide array of codes currently used for seismic design in the United States. This situation is a direct consequence of the progressive recognition of the seismic risk in different areas of the country. In the West Coast, and particularly California, seismic design regulations have been in effect since the early 1930's when the 1933 Long Beach earthquake seriously damaged many school buildings. For this area both the Structural Engineers' Association of California [8] and the Uniform Building Code [9] have provided since the 1960s strict guidelines for the design and detailing of buildings to insure good performance during earthquakes. In the Eastern part of the United States, where other codes such as the BOCA [10] and SBCCI [11] are used, is has not been until recently (late 1980s) that seismic provisions have been put into effect. It is interesting to note that while all these codes agree in the fundamentals, their design forces and levels of detailing vary widely, particularly for areas of low to moderate seismicity.

All the current codes for seismic design share three important problems:

(a) Our knowledge of the seismic performance of structures increases with every new major earthquake and with experimental and analytical research. Codes have become reactive documents that incorporate this knowledge. Thus new provisions are brought in haphazardly, when portions of the existing code are deemed unsafe. There has not been a major rewriting or restructuring of these codes for many years and thus the level of safety implied varies with the type of member and structural system being designed.

(b) Many of the codes were originally conceived in allowable stress terms because that was the prevailing design method from the early 1900's through 1950's. When ultimate strength methods became popular, they were calibrated to the allowable stress provisions. Thus several of the codes remain a hybrid between ultimate strength and allowable stress design irrespective of the fact that seismic design has traditionally been treated as an ultimate limit state.

(c) Current codes purport to address only the life safety of the occupants and do not intend to deal with limiting structural damage. Both the 1989 Loma Prieta and the 1994 Northridge earthquakes have shown that while the structural damage that occurred was primarily to older, brittle structures, the costs due to non-structural damage far exceeded those for structural damage in most newer construction. Thus limiting damage has arisen as a new challenge for code writers, a challenge that will not be easily met.

Following the 1971 San Fernando earthquake, the government recognized on that there should be some uniformity on codes and that a single entity should serve as a clearinghouse for processing damage information and new research. This led the Federal Emergency Management Agency (FEMA) to award a contract to the Applied Technology Council (ATC) in the early 1970s to develop a resource document that could serve as the foundation for a nation-wide seismic code. ATC assembled a large number of designers and researchers with the objective of developing a new, more consistent model code based on ultimate strength design (items (a) and (b) above). The result was the ATC 3-76 document [12] which became the basis for today's National Earthquake Hazard Mitigation Program (NEHRP) Recommended Provisions for the Development of Seismic Regulations for New Buildings [3]. In 1979 FEMA gave the Building Seismic Safety Council (BSSC), and arm of the National Institute of Building Sciences, the task of maintaining and developing these provisions. Major new editions of the NEHRP provisions were published in 1985 and 1988, and revised in 1991. A new edition, incorporating a separate chapter on composite construction, is slated to appear in September 1994 [3]. BSSC is hard at work on a new version that may address the issue of non-structural damage scheduled to be released in the late 1990s.

The major differences between the way NEHRP provisions and other classify structures are that NEHRP uses the concepts of Seismic Performance Categories (SPC) and velocity-related acceleration. The latter is an equivalent acceleration derived from velocity maps and intends to account for the better correlation observed between peak ground velocities and damage as the distance from the earthquake source increaes. This effect is ascribed to filtering of different frequency components of the earthquake with distance and soil profile. Depending on the use of the structure and the level of velocity-related acceleration, NEHRP classifies structures as SPC A through E, with SPC A requiring basically no seismic design and SPC E requiring a great deal of seismic detailing. The level of analysis required (equivalent lateral load, modal analysis, or other) is also based on the SPC and the presence of discontinuities in both vertical stiffness and horizontal plan.

STRUCTURAL SYSTEMS FOR HYBRID AND COMPOSITE CONSTRUCTION

One of the major differences between the 1991 and 1994 edition of NEHRP is the incorporation of a new chapter dealing with composite construction. In developing the recommendations for the NEHRP 1994 edition, BSSC Task Group 12 - Composite Construction attempted not only to develop provisions for existing systems but also to encourage the development of new systems that promote structural efficiency. The idea is that many of the newer system could be alternatives to all steel or concrete systems in low to moderate height buildings (4 to 30 stories). The new NEHRP provisions recognize seven types of composite structural systems. Since many of them have a counterpart in steel and concrete the prefix "C-" has been used to name the corresponding composite system:

(a) **Composite Partially Restrained Frames (C-PRF)**: C-PRF consist of steel columns and composite beams joined by composite semi-rigid connections [13]. This is a very interesting system from the seismic standpoint because the flexibility of the connections can be used to adjust the natural period of the structure and thus reduce seismic demand. In addition, the connections provide a "fuse" since they are weaker than either beams or columns, and thus a true capacity design approach can be used in their design [14]. Because of this flexibility, however, these structures would also seem to be stability (P-Δ) critical, at least under a simplified equivalent lateral load design approach. On the other hand, because the joint strength is predictable and capped, the stability of the individual members can be predicted better.

(b) **Composite Ordinary Moment Frames (C-OMF)**: C-OMF include a variety of configurations where steel or composite beams are combined with steel, composite or reinforced concrete columns. The term ordinary is used to indicate that little of the detailing required for critical structures (SPC D and E in NEHRP) is envisioned in this type of structure.

(c) **Composite Special Moment Frames (C-SMF)**: C-SMFs are similar to C-OMFs except that much more stringent detailing is required in order to provide behavior similar to that of a steel SMF. In this case the columns, if composite, are required to both meet all AISC requirements for b/t and h/t ratios and have all the transverse reinforcement required for columns by Chapter 26 of ACI. As in most ductile frames, the columns and joints are required to develop the full strength of the beams so that a stable strong column-weak mechanism develops. Clearly the design of the connections is a key element in this system, and recently detailed provisions for a system incorporating steel beams and concrete columns has been proposed [15]. Fig. 2 shows a typical connection in the Norwest Center building which were designed basing on extrapolating from this research. C-SMF are very efficient in resisting earthquake loads as shown by Japanese SRC (steel-reinforced concrete) system in which an entire steel skeleton is covered with reinforcing cages containing large amounts of transverse reinforcement.

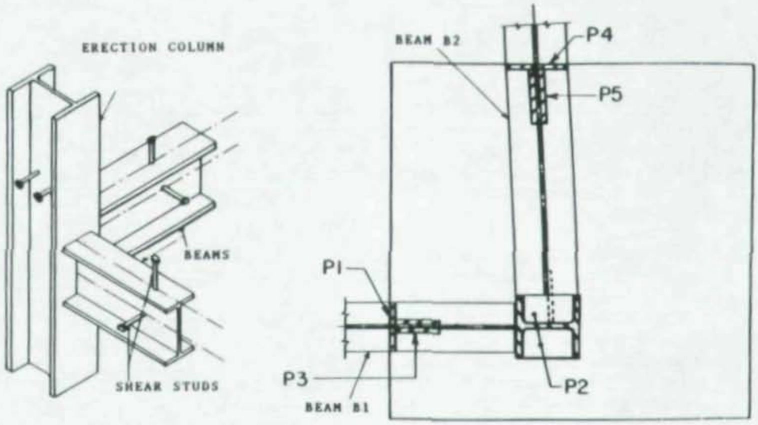

Fig. 2 - C-SMF connection.

(d) **Composite Concentrically Braced Frames (C-CBF):** C-CBFs are similar to their steel counterparts except that some of the members (beams, columns, and braces) are composite. There is considerable debate on the applicability of braced frames in areas of high seismicity because the tendency of the braces to buckle results in poor energy-dissipation characteristics if the structure goes inelastic. To alleviate the buckling problem several researchers have proposed to utilize composite braces (either encased shapes or concrete-filled tubes) where the stiffening effect of the concrete prevents local buckling. In many cases the composite action is required only within the brace and presents a problem at the connections since ideally they should act as truss members. In this case careful detailing is required at the ends of the braces to insure that ductile yielding in the member and not a fracture at the connection is the failure mode.

(e) **Composite Eccentrically Braced Frames (C-EBF):** C-EBFs, as the name implies, are analogous to the usual eccentrically braced frame except that some of the members are composite. When the EBF concept was originally developed there was some concern as to whether the floor beams, which are in effect composite beams, could accommodate the large rotational ductilities demanded by the system without causing local failures. Extensive research has been carried out in this area indicating that the floor elements are capable of withstanding the very large shear deformations required by short links [16].

(f) **RC Walls Composite with Steel Elements:** At least three possible variations of this system exist, and they correspond to cases of hybrid structures. The first utilizes concrete panels as infills in steel or composite frames. The second is where large steel sections are used as boundary elements in concrete shear walls. The third one is where steel or encased composite beams are used to tie two reinforced concrete shear walls [17,18].

(g) **Steel Plate Reinforced Shear Walls:** Since the early 1980's the concept of utilizing steel plate shear walls has been popular [19]. The concept is very similar to the use of plate girders in bridges, except that the main element is vertical rather than horizontal. These systems have been used successfully as retrofits in critical steel structures (hospitals) where access to the structure was severely limited by the need to keep it operating during the retrofit. The system basically behaves as a CBF with the tension field action taking the lateral loads. Composite steel shear walls, in which the steel plate is covered with concrete and composite action activated by mechanical connectors, have been postulated as a system with better energy dissipation capacity. Another variation could be a sandwich configuration where the space between two thin steel plates containing studs is filled with concrete. In this case the steel plates act as the formwork and could be welded directly to an existing steel frame. Great care is needed in connecting the plates to the boundary elements since the shear wall is such an efficient structural element that it can easily overstress the adjacent columns and beams.

NEHRP STABILITY PROVISIONS

Stability provisions for application in seismic areas are generally of two types, local and global. Local failures are controlled by requiring some minimum member dimensions to insure that members can reach their plastic capacity and rotate sufficiently to allow for force redistribution. Insofar as steel sections are concerned, local stability is enforced by limiting the b/t and h/t ratios for the elements making up the steel shape. For concrete members local buckling is generally not a problem because of the large moments of inertia involved. On the other hand, local buckling of bars is a common failure mode observed in both past earthquakes and laboratory tests to large deformations. Minimum requirements for cover and transverse reinforcement are used to control this phenomena and delay it until large rotations have been achieved.

Most of the data used to derive local buckling provisions was developed from laboratory tests on steel and reinforced concrete members. Relatively few tests for local buckling are available for encased or concrete-filled members although it could be argued that any strength test for these members is also a local buckling test. In developing the new NEHRP recommendation BSSC Task Group 12 chose to use the existing AISC (Section B.5, [6], and ACI (Chapter 26, [5]) provisions as they were seen as a conservative estimate. This decision was reinforced by the fact that only a scant amount of this data was obtained under strain rates similar to those that occur in a major earthquake. Past performance and our understanding of strain rate effects lead us to believe that static tests constitute a safe lower bound insofar as local buckling. In composite systems, due to the reliance on flexible mechanical connectors, this may not necessarily be the case. Crushing of the concrete could lead to larger slips between the steel and concrete portions than in the static case and losses of strength at comparable deformations.

Overall or global stability failures are prevented by requiring that P-Δ forces be properly accounted for in the analysis. Several methods, including those in Chapter B of the recent editions of the AISC Specifications [6], can be used to check this condition. The NEHRP provisions separate the required analysis according to the height of the structure and the regularity of the structure with respect to vertical stiffness and plan geometry. For most office low-rise buildings, it is likely that only an equivalent lateral force (ELF) analysis will be required. In this case the NEHRP provisions require that a more thorough analysis be done if the P-Δ moments exceed one-tenth of the overturning moments at any story. This is expressed as:

$$\theta \le 0.10 \tag{1}$$

where,

$$\theta = \frac{P_x \, \Delta}{V_x \, h_{sx} \, C_d} \tag{2}$$

In this equation P_x is the axial force in the columns in the story above, Δ is the story drift, V_x is the story shear, h_{sx} is the story height and C_d is the displacement amplification factor. C_d is the ratio of inelastic to elastic displacement, and its is given by the provisions for different structural systems. It appears in this equation because Δ is equal to $C_d\delta_{el}$, where δ_{el} is the drift calculated form an elastic analysis.

To limit the overall P-Δ effects, the NEHRP provisions also require that:

$$\theta \le \theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \tag{3}$$

where β is the ratio of the story shear demand to the story shear capacity at that story. β can be taken as 1.0. If θ is greater than 0.10 and less than θ_{max}, P-Δ effects must be checked by other rational analysis. The NEHRP commentary suggests that a rational analysis can be carried out by increasing the lateral loads at at floor by $(1+a_d)$, where:

$$a_d = \frac{\theta}{1+\theta} \tag{4}$$

Note that this multilplier is really the summation of the series: so that successive increments in Δ are included. If θ is greater than θ_{max}, the structure must be redesigned. If the structure is

$$(1 + a_d) = 1 + \theta + \theta^2 + \theta^3 + \ldots \tag{5}$$

irregular, a modal analysis is required to obtain forces and deformations. After these have been obtained, the P-Δ effects can be checked by the same procedure as for ELL

It should be understood that the code-type provisions for checking stability are a considerable simplification of a very complex problem. The seismic demand is not well known, our ability to calculate deformations is poor, and the use of factors to convert from elastic to inelastic deflections is questionable for this application. On the other hand, the alternative is to conduct a series of full non-linear dynamic analysis that includes both P-Δ and P-δ effects, member plastification, panel zone deformations, and the effect of non-structural elements. This analysis would have to be run for several input motions, including probably some white noise ones. While this option may be acceptable for a major structure, it will certainly not be so for a low or moderate height office building.

SEMI-RIGID COMPOSITE CONSTRUCTION

To llustrate the stability provisions of the NEHRP document, a design example of a composite partially restrained frame will be discussed. The example frame is a four-story, three bay C-PRF designed for an area with velocity-related accelerations of 0.15g. In this region wind loads for 90 mph and Exposure B by ASCE 7-88 governed the design and only the detailing of the connections and columns was done for NERPH SPC C.

The design forces and members selected are shown in Fig. 3. A companion rigid frame was also analyzed. This rigid frame had the same members as the C-PRF, but the connections were much stiffer. The moment-rotation curves for the two cases are shown in Fig. 4 for the upper stories (W21x44 beams). The curves for the first two stories (W24x55 beams) are similar but the strengths are about 15% higers.

It should be noted that the rigid frame is probably not a particular good design since the columns are weaker than the beams. On the other hand the frames are controlled by drift, and thus have a substantial overstrength (by a factor of 3 to 4) over the design strength. The size of the composite beams in the first two floor were increased to help control drift in the C-PRF case so that the column sizes remained reasonable for a four-story frame.

The frames were analyzed for the load case 1.2D + 0.5L + X.X (W or), where X.X is the load factor plotted in Figs. 5 and 6 against the interstory drifts for the four levels. The 1.3W combination that governed this design for lateral loads corresponds to a total lateral load of 60.2 kips.

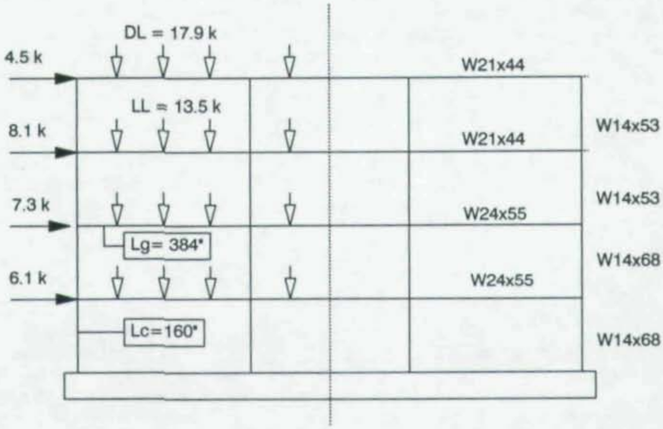

Figure 3 - Details of example frame.

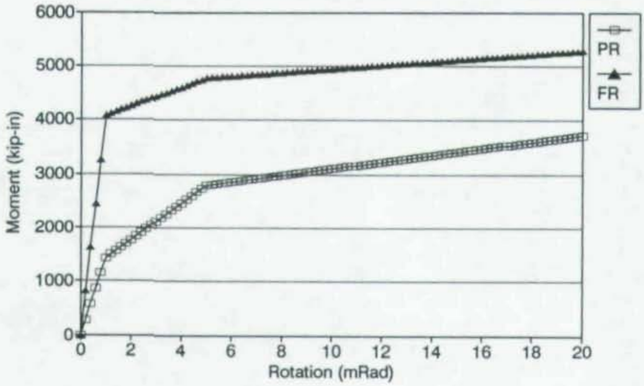

Figure 4 - Moment-rotation curves used for the connections.

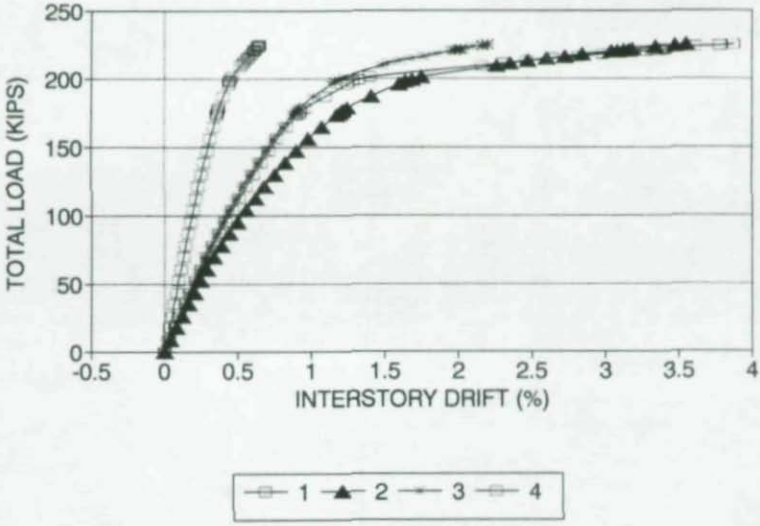

Figure 5 - Load-interstory drifts for C-PRF case.

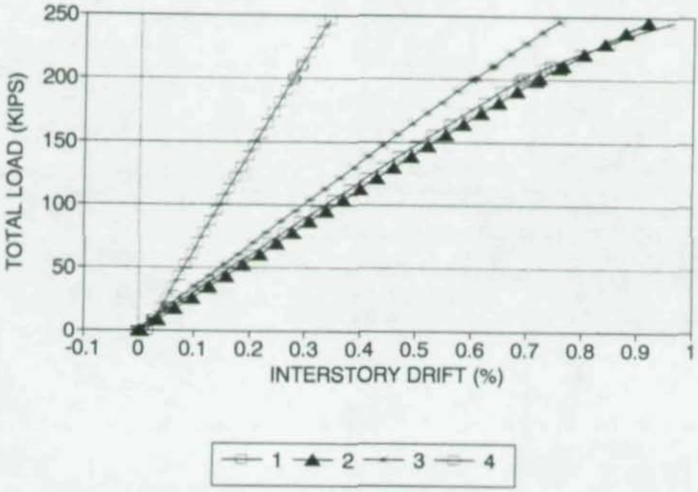

Figure 6 - Load-interstory drift for the rigid frame.

Stability checks for individual members were carried out by assuming that the effective length factor should include the effect of the spring. This can be accomplished by assuming a modified I_{eff} for the girder given by:

$$I_{eff} = I_{eq} \left(\frac{1}{1+3\alpha}\right) \tag{11}$$

$$\alpha = \frac{2 \; E \, I_{eq}}{L_g \, k_{ult}} \tag{12}$$

where L_g is the length of the beam, E is the modulus of elasticity of the steel beam, and k_{ult} is the secant connection stiffness at ultimate. The choice of a secant stiffness, as opposed to a tangent stiffness is justified on the basis of some research on SDOF inelastic systems [20].

For the C-PRF the hinges began to form at the base of the columns at a total load of 180 kips or a load factor close to 3. All four hinges at the base had formed by the time the total load reached 201 kips. At this stage most of the connections had rotated at least through 10 mRad, and were softening considerably. While the columns had been designed under a strong column-weak beam philosophy with an overstrength factor of 1.25 over the beams, the P-Δ effects soon became important, leading to hinging in the columns. This hinging occurred at the bottom of the second and third story interior columns and on right exterior column (the one with the most compression). At this stage the left exterior column (the one with less compression), was stabilizing the whole structure. The collapse occurred when the first interior column hinged at its top at a total load of 225 kips or 3.72 times the design load. The sway at this point was very large (about 4%) indicating that the ratio of gravity/lateral loads was low.

For the rigid frame the sequence of hinge formation was different. A story sway mechanism formed at the bottom story at relatively low deformations due to the overstrength of the beams. The hinges began to form at the bottom of the columns at a total load of 180 kips (L.F. = 3.0) and drift of about 0.6%. The hinges at the top of the columns began forming at a load of 260 kips and an interstory drift of 0.7%, with collapse occurring at a load factor of 4.43 and drift of 1.6% in the bottom story.

Figure 7 shows a plot of the value of θ (Eq. (2)) against the interstory drift for the C-PRF case. As can be seen θ is small from the beginning of the loading, with a value of 0.04 at the design load (60.2 kips). The value of θ does not indicate that this is a P-Δ sensitive structure. The correction factor a_d at the design load level resulted in a series of new lateral loads which increased θ to 0.05. The fact that this structure was able to remain stable through very large deformations (up to 4% interstory drift) indicates that θ may be a good indicator of stability under an equivalent lateral load.

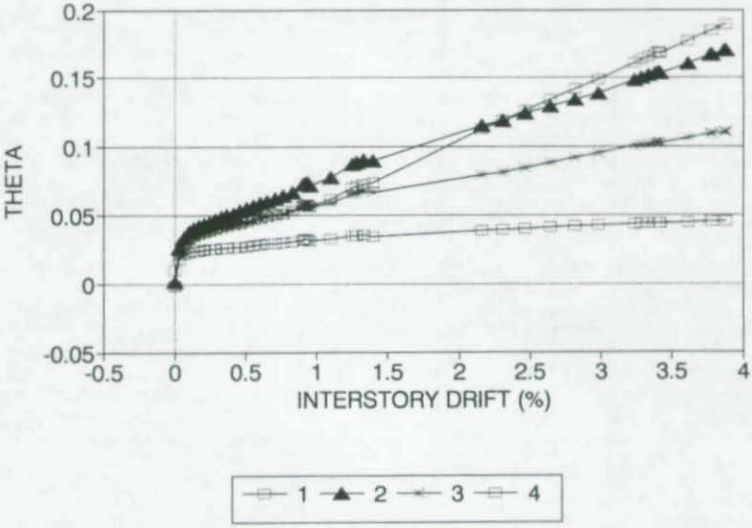

Figure 7

CONCLUSIONS

This paper discussed the new proposed NEHRP provisions with respect to local and global stability of composite frames. The provisions are very approximate, but seem to give a good index of the stability problems even for partially restrained frames. The global stability of structures in areas of high seismicity is definetely an area where much research is needed in order to develop simple but reliable provisions.

Figure 7 - Value of θ with increasing drift.

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COMPOSITE CONSTRUCTION RESEARCH AND PRACTICE: RECENT DEVELOPMENTS IN EUROPE

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ABSTRACT

The first recorded examples of steel-concrete composite construction date back to the end of the 19th century, when bridges and buildings appeared, conceived and designed so that the two materials were acting together. Despite the immediate interest risen among practitioners because of the apparent advantages (including the improved fire resistance), it was only rather recently that composite construction became a popular and successful form of construction. Among various factors, some have to be mentioned, which may be considered of prime importance in the successful development of composite construction: (A) the technological advances (basically the metal sheet decking and shear connectors like the studs weldable through the sheeting), (B) the recent trend towards larger open areas with increasing intensity of servicing requirements, (C) the significant importance of the construction speed on the total cost of the project, (D) the development of increasingly competitive and structurally effective composite systems, and the availability of new and improved materials.

The geographical area where composite construction was traditionally very strong is North America. The tendency towards a growing market for composite systems is now reported as a worldwide phenomenon for both bridge and building structures.
The number of research studies, the proposal of novel structural solutions and the activities for updating and/or developing new Codes (1 - 6) can be assumed as indexes of the importance of a "material" related type of construction. As to the last parameter, and to buildings, it should be noted that, in the vast majority of countries, recommenda tions related to the design and analysis of composite members (mainly beams and slabs) were included in Steel Codes. Besides, no specification was provided for the design of systems. The importance of composite construction is now demanding for a specific and "dedicated" Code, dealing consistently with all the major aspects of the design of composite members and systems. The Eurocode 4 represents the European response to this new requirement [7].

This paper intends to review shortly the main European activities and developments in the last decade in the fields of research, practice and Codes. Although it is recognized that the developments in construction practice, research activity and Codes are deeply interrelated, a separate treatment of these aspects was adopted as being the most effective approach to an overall survey. Construction practice is reviewed first, which represents the starting point of many investigations, Code "advances"are then considered, and finally research studies are presented and discussed. Different aspects of composite construction will be treated further to component and system stability. It is opinion of the Author that focussing the attention on the sole stability aspects would seriously reduce the scope and the interest of the report.

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STABILITY: DIRECTIONS IN EXPERIMENTAL RESEARCH

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INTRODUCTION

The use of experiments in structural engineering predates computers, finite element analysis, limit states design, and Euler's column. The building or construction site was often the experimental laboratory where concepts were tried and retried, designs were modified and engineering practice improved incrementally. In this paper a capsule view of the development of "classic" and "leading edge" testing equipment and techniques is presented. The changing test environment is described in light of the changing roles of analytical and experimental research.

Problems with interpreting experimental data and those of evaluating analytical results have many parallels; neither can claim to be universally superior to the other and each possesses inherent strengths and weaknesses. It is important to realize that the experiment is often the object on which analyses are evaluated for qualification or validity; the quality and limitations of the experiment must be understood to justify this objective. The experiment as addressed here, relates to structural engineering and the stability of metal structures; some points have general application to all structural experimentation and to any material.

Good experimentation on topics of structural stability must include extensive measurements of the geometry and material properties of the structure being studied, and since the structural behavior is so tied to these parameters, full or large scale testing is often favored to bring realistic values into the test. Of equal weight, in creating useable results from experimentation is knowledge of the behavior of the test equipment; stiffness, restraint, friction etc. effects can be dealt with if they have been measured as part of the parametric data. The order of the following discussion does not presume any relative importance; the relative order could only be judged for a specific case. Referred examples deal with experimentation on structural members and assemblies and the author's own experience, but the concepts apply to the broader area of experimental stability.

ANALYTICAL VS. EXPERIMENTAL MODELLING

When the value of an analysis versus an experiment is raised today, the extreme points of view which suggest either as categorically superior to the other, can rarely be defended; the purpose for discussing them here is rather to show their similarities and a potential for a symbiotic relationship. With the emphasis on experiments related to stability of metals, the growth of both analytical and experimental capabilities in research has emphasized the need to recognize this relationship. The "value" here is that associated with learning, discovery, and improvement of engineering practice; no cost factors are addressed.

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Although one strives for truth in studying a phenomenon such as stability, experimental and analytical models are both approximations of reality in structural behavior. Classic analytical models deal with linear elastic material, perfectly straight columns, frictionless bearings, concentric loads, zero self weight and infinite rigidity. And many examples of classical experimental research sought to recreate the mathematical ideals (eg. careful fabrication of models, elaborate lubricated bearings, etc.). The modern trends in analysis and experiment, while using the classical methods for landmarks, are moving closer to reality and the symbiotic nature inferred earlier. An analytical model should ideally be able to include the effect of any measured parameter in an experiment and thus be able to address the importance assignable to it.

Analytical modelling using finite element methods can now solve problems involving large displacements and incorporate non-linear material and geometric behavior; some software is particularly useful for studying post-buckling structural behavior. Large displacements although achieved in structural experiments have presented challenges in measurement, control, and experimental design which strongly parallel the analytical developments. Experience with analytical techniques and limitations can be useful in designing an experiment.

OVERVIEW OF TEST EQUIPMENT DEVELOPMENT

Most of the early development of experimental equipment in the form of "stand alone" test machines or frames was strictly mechanical, using screws and levers to move the parts and induce loads; the load measurement was also "mechanical" in the form of force measurement using the "weighbar". This reflection on early mechanical developments dating back more than a century, is important because the wealth of research information that we use in our recent literature has been influenced by the characteristics and limitations of the equipment. Also, although recently produced, many testing machines and measurement devices retain the characteristics and limitations.

A brief discussion of the universal test frame, still the most popular source of information on material and structural behavior under load, illustrates the legacy and state of the art in experimental structural laboratories. An early test frame had the ability of applying and measuring a self-reacted force in a nearly static (very low rate) to an object positioned to accept the loading. Tensile and compressive coupons or samples are still tested in this fashion for simple mechanical properties evaluation. The universal test frame was and is described first by the maximum force that it is designed to apply and measure and by the size (height/width/depth) of the sample that it can accommodate. With this fundamental definition, the following sequence of developments added an array of capabilities that describe the experimental potential as it is today:

- Hydraulics replaced the mechanically driven screws as an alternative loading method

- Development of the electrical resistance strain gauge brought electronic load sensing which was quicker in response and provided a signal which could be used for control.

- Electronic displacement devices replaced mechanical ones for further improvement in measurement and control.

- Separate and two-way hydraulics provided a way to remove or reverse loading.

- Screws used for control of head position and application of load were replaced with mechanically clamped heads which transferred the force by friction and provided a more rigid frame and a precise means of positioning.

- Hydraulic capabilities were enhanced with the advent of the oil-hydraulic servo-valve used to control the flow of oil under pressure. When combined with two-way hydraulic "actuators" the heart of "servo-controlled closed-loop" testing emerged.

The servo-controlled closed-loop universal testing machine which began to appear in the 1950's is now commonplace in testing and research facilities. This equipment operates with the simplicity of the older equipment or with a sophistication often limited only by the researcher. At the same time that servo-hydraulic testing was emerging, the use of the structural test floor as an integral part of structural testing facilities emerged as a popular means of testing more complex and realistic structural elements and assemblies. No longer centered on the universal test frame but rather on a more versatile system, major structural research laboratories have servo-controlled hydraulic actuators capable of being controlled by the requirements of force, displacement or strain, and many are integrating computers into the control aspects as a complement to computer based data acquisition. Test walls have been added to test floors to permit additional loading directions.

Electronics have, over the same period of time, become miniaturized and the dramatic improvement in speed and reliability of computing has brought about the most significant development in the form of the computer control of experiments. The first regular use of computers in the lab was for the acquisition and processing of data; now, real time processing of data combined with direct feedback control enables experimentation which can simulate changing boundary conditions as they might be influenced by the measured response of the element under test.

A specific example of this process although outside the scope of this paper is found in pseudo-dynamic method which is used to simulate the dynamic loading and response associated with earthquakes. Here the loading/displacement is incrementally calculated on the basis of an analysis of the dynamic response of a larger structure of which the test piece or subassembly structure is a part. The time scale is thus changed to permit the computations and loading to be completed within the limitations of the equipment. In this case, a dynamic event has had the rate reduced and permits large scale structures to be tested for dynamic effects that could only have been simulated previously by high rate tests of small scale models.

This example is presented here because it combines use of state of the art equipment and redefines the specimen archetype so that through the use of non-linear modelling control of the test has been achieved during specimen degeneration. The failure modes in the structure subjected to earthquake motion may well be limited by local or global instability, but the method and equipment can be focused to deal with more subtle aspects of structural stability involving boundary conditions and the response of a structure containing the tested subassembly or element.

MATERIAL PROPERTIES

Two goals are satisfied by material properties tests in experimental research:

(1) identification and measurement of the performance material to calibrate test results,

(2) information on the elastic and plastic performance of a material for the purposes of further experimental investigation and for analytical modelling.

The tensile coupon is always used to satisfy the first goal and is a requirement for most metal material standards such as those of ASTM and CSA in North America; when stability is the objective, one is looking for compression properties, but most materials standards tests for metals are only amenable to tensile testing. If compression properties are likely to be different, alternative tests can be performed on specialized coupons.

Data on yield stress, ultimate stress and elongation at rupture help to identify a material type or product but, in general, do not provide for the second goal. These fundamental tensile parameters define a "box" which contains the desired information, but tell little about what is inside the box. The SSRC Guide[1] provides SSRC Technical Memorandum No. 7: Tension Testing which contains procedures to obtain and report more extensive material properties. It goes farther than most specification requirements for tensile mechanical properties.

The ability to provide complete stress vs. strain behavior from tensile coupons has been available for many years. The use of tangent offset techniques to define yield for materials without a yield plateau may require continuous strain measurement through the defined yield strain but nothing afterward. Researchers should use the capabilities, characteristic of most modern test equipment, to provide a complete documentation of coupon performance. Some additional considerations beyond those generally provided in standards are given here:

- Many clip gauges used to measure strain over a specific standard gauge length do not have sufficient range to measure the elongation at rupture. Two different gauges can be employed simultaneously or the stroke-output of servo-hydraulic machines can be used to calculate the larger strains, approximately, once the elongation at rupture is measured at the completion of the test.

- Care of removal and handling during fabrication cannot be emphasized enough. The removal of a coupon from a larger metal section may be compromised by the presence of residual stresses which cause cutting and machining difficulties. Special care must be taken to prevent the common occurrence of permanent straining the material. Carefully fabricated coupons taken from regions of high residual stress must be clamped flat to perform the coupon machining, and the resulting coupon may be far from flat or straight. Further care must be taken in mounting and aligning the coupon in the testing machine. Once again the independently powered hydraulic grips of modern equipment permit installation of the coupon with the machine load maintained at zero. The resulting stress vs. strain will closely approximate the insitu properties.

- Variation in physical properties of the materials throughout the volume of the a structure or member is generally established through coupon testing. While recognition of this can lead to many tests, a judicious sample to establish the bounds of the variation and direct correlation with average properties from full section tensile or full section compressive tests (stub-columns) can be used to establish representative properties.

- The strain rate varies widely from production/manufacturing to research facility; the rate ranges established in material standards give practical rates for production facilities but often cannot and should not be applied to tensile coupons used for research purposes. If both are required they should be performed separately.

- Zero strain rate tests have been reported in the literature and modern servo-controlled test equipment permits measurement at a strain held constant according to a specific strain signal; in a tensile test this signal is usually provided by a clip gauge, but could be from another source. After a finite time (usually a few minutes) all parameters such as load become constant and the associated values are those at "zero strain rate". If a test had been done at a strain rate approaching zero, in theory, the values obtained would be bounded by those performed at zero strain rate. In large scale testing which is conducted at a low rate or where the strain rate is zero at the time of failure the "zero strain rate" parameters may offer a more meaningful calibration of the results.

- Controlling a test on the basis of strain has been done cautiously because of the potentially dangerous consequences if the sensing gauge was to slip. Later servo-controlled equipment capable of mode switching can mitigate this problem by only remaining in strain control for the required measurements and then returning to load or displacement control as appropriate.

The stub-column test is a compression test of an entire cross section that is used to obtain the nominal stress vs. strain properties of an entire cross section except that length effects associated with column stability are avoided through appropriate selection of specimen length. The SSRC Guide[1] provides Technical Memorandum No. 3 Stub-Column Test Procedure containing extensive description of equipment and measurement techniques. A similar test procedure, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns is included in the AISI LRFD Cold-Formed Design Specification[2]. This document deals with sections likely to have their local strength governed by buckling and the sections may be non-uniform, eg. perforated; calculation techniques for interpretation of data for use in design are included. This specialized test is an example of a formal test to evaluate performance of products which may be too complex to rely on established empirical or analytical methods.

Early research, which established the stub-column testing techniques, used equipment that did not have the rigidity or control mechanisms available on modern test frames using with servo-controlled closed-loop systems. Specimen alignment for the purpose of maintaining uniform application of strain is now easier and generally is accomplished at lower load levels because of the rigid attachment of the crossheads to the columns. As buckling loads are reached the rigorous control of total displacement and the improved rigidity permit slow development of the buckling of the cross section whether before or after yielding. When local postbuckling of the cross section is of interest, the behavior can generally be observed and recorded without any difficulty imposed by testing machine response.

GEOMETRIC PROPERTIES - IMPERFECTIONS - RESIDUAL STRESS

The actual dimensions of structural assemblies and members can be established through direct physical length measurement which will show deviations from specifications and permit calculation of "measured" geometric properties such as area and moments of inertia of cross sections. Measurement techniques generally have to be tailored to the particular geometry that is being established, but today, all have the common capability of electronic sensing, recording and reduction of the data through the use of computer driven data acquisition equipment. Techniques similar to those of topographic mapping the complete geometry of parts can be established and the nature of the deviation from "perfect" or "specified" can be determined. An example of this mapping and a detailed procedure for fabricated tubular members are presented by Prion and Birkemoe[3,4].

For more regular and "constant" cross sectional shapes produced by hot or cold roll-forming local shape dimensions can be mapped from a sampled "slice" of the cross section using techniques which employ scales and micrometer calipers or computer scanning and quantitative graphical imaging of the area. Often the simple technique of weighing a measured length provides the most direct determination of area; this can be combined with buoyant weighing if the density of the material is not known.

Residual stresses are part of the imperfection reality that occurs in of the manufacturing and fabrication process of metal plates and shapes. Their presence is not necessarily negative; recognition of the effect on compressive behavior in columns has resulted in techniques of manufacture to use the residual effects to improve stability. Knowledge of the presence of residual stresses is essential to quantify buckling behavior that involves any observed yielding Techniques for measuring residual strains through mechanical release or removal of the stresses using electric resistance gauges or mechanical measurements are well documented in the SSRC Guide[1] Technical Memorandum No. 6: Determination of Residual Stresses. The relationship between elastic strains and stresses often requires an assumption about the original state of the distribution of stress. If the heat or forming history is known this assumption can be made with the help of analysis. Tensile coupon and stub-column tests cited earlier can yield some knowledge about the residual stress magnitude if carefully conducted.

BOUNDARY CONDITIONS

Problems with boundary effects rank very high in frequency of use to explain unanticipated behavior associated with buckling. Most are attributable to an assumed "ideal" behavior which is not realized. The SSRC Guide[1] Technical Memorandum No. 4: Procedure for Testing Centrally Loaded Columns presents a discussion of common "pinned end" fixtures used for column testing. These mechanical "pinned end" devices all have inherent resistance to rotation when subjected to axial load, and none provides the perfection that is so easily found in an analytical model. Also rollers, bearings, knife-edges, linkages provide a theoretical freedom in one or more directions and theoretical restraint in others (eg. a one-way roller theoretically provides full fixity about one axis and zero about the other). The "zero" fixity should receive most attention and when possible should be measured under load in an independent experiment. In all cases the performance of a boundary condition during an experiment must be monitored to measure its mechanical function and, when feasible, the associated force components.

In most experiments the "ideal" passive boundary condition provides a tempting simplification in an already complex testing environment. When buckling is the behavior studied, the small errors associated with the idealization can cause significantly erroneous results unless the boundary behavior is identified. This is particularly true when the buckling is a departure from an unmoving state. Here, a very small amount of restraint can have a major effect.

An experiment which, because of the slight misalignment or imperfection in shape, tends to mobilise the boundary condition often leads to a more realistic result. Some analytical procedures exhibit similar behavior; the solution to obtaining the buckling result is to provide a small perturbation in the form of a geometric imperfection or a small force to create deformation not generated by the primary forces. A parallel approach works experimentally in some situations; a dithering force applied to the test specimen or test hardware during a test can cause the test structure and boundary conditions to act in the intended manner. These techniques are difficult to quantify and must be determined "by experimentation".

Invariably, restraints with numbers like 0 and ∞ describing their behavior do not perform in this "ideal" fashion. Preferably, known stiffnesses occurring as a result of a connection or attachment can be quantified through analysis or experiment and used in the structural experiment. During experimentation, monitoring of translational and rotational sensors at points of restraint can identify the functioning of the passive restraints. Use of instrumentation to sense force can further evaluate the restraint performance and through careful design, the ambiguity about the performance of boundary conditions is removed.

THE FUTURE WHAT NEXT?

An experiment [3,4] which was part of an ongoing study of the behavior of tubular members at the University of Toronto is pictured at the close of this paper, Figure 1. This study contains many examples of attempts at the techniques discussed here and the practical limits that remain a challenge for the experimental researcher. As an example of a computer controlled experiment within the last decade, it should be noted that much of the computer equipment and interface have been superseded as a result of a rapidly expanding technology in computing. Many other examples of current experimental techniques are found in the latest edition of the Guide[1] ; a new book by Singer and Arbocz [5] compiles extensive information on buckling experiments.

The presence of the computer in the experimental scene will continue to enhance and provoke the development of experimental technique and capabilities. High speed and high quality data acquisition combined with on-line use of the data for control of the loading and/or the response of a boundary condition present a framework for improved experimental demonstrations of stability limits in structures. Improvements in user software for the test environment continue to make the access to computer control easier.

In the ultimate test environment, the performance of restraints or boundary conditions will be modelled by defined performance criteria (a hinge will be a place where the measured moment is maintained at zero) or the boundary may be the computed response of an analytically modelled structure which in fact depends on the performance of the test piece. Digital controllers now in use for complex dynamic applications may offer versatility and improved performance in multi-axis quasi-static testing.

The growing analytical capabilities in the realm of nonlinear behavior and large displacements, combined with the sophistication of experiments which can be performed in conjunction with analytical models offer many potential improvements to buckling experimentation. Evaluation of buckling and postbuckling behavior in a more realistic and known environment will go far beyond the common comparisons of ultimate loads by "analysis versus experiment."

As one dreams, it is prudent to remember that **an experiment is only as good as:**

(1) the knowledge of the parameters describing the specimen,
(2) the capabilities of the experimental equipment for measurement and control,
(3) the completeness of the selection of sensors monitored during the test,
(4) the reporting.

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3. Prion, H. G. L. and Birkemoe, P. C. (1988). "Beam column behaviour of unstiffened fabricated steel tubes." *Department of Civil Engineering Report* No. 88-03, University of Toronto, Toronto, Ontario, 362 pages.

4. Prion, H. G. L., and Birkemoe, P. C. (1990)."Beam -column behavior of fabricated steel tubular members," *J. Struct. Div.* ASCE , Vol.118, No. 5 1213-1232.

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Horizontal axial loading of the member is reacted through the test floor while gravity load simulators are used to apply and react vertical loads causing flexure. This test used spherical end bearings with known characteristics which were established by testing. Although conceptually simple, this research was the first at Toronto to use computer control for structural testing.

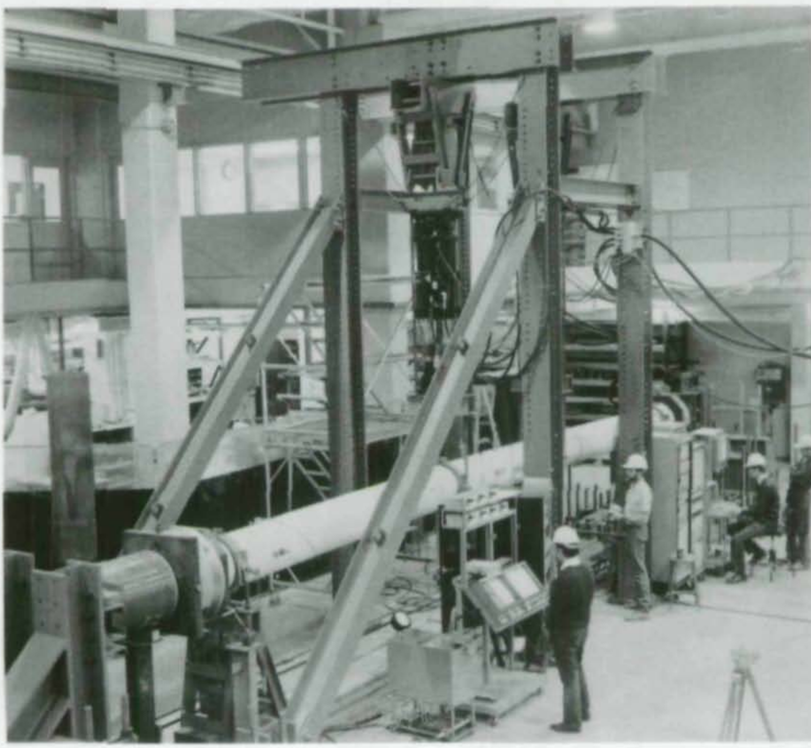

Figure 1 Beam-column testing of a 10 metre tubular member (Ref.3,4)

EXPERIMENTAL DATABASE ON STRUCTURAL STABILITY

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ABSTRACT

Test data are invaluable resource for structural engineering research. In this paper, a numerical database system is built for material properties of steels, steel columns, steel beams and unstiffened steel plates, etc. The database is used to clarify the strength variations of each structural member. In addition, the test data in the database are compared with existing design formulas. The database is further developed in the shape of structural test information-based system. This system includes database functions, test data evaluation functions, reference retrieval functions and technical terms explanation functions. The usefulness of the knowledge base approach for representing the test data is proved and the potential and problems of the system are clarified.

1. INTRODUCTION

(1) Historical Review of Steel Members Tests

Structural elements such as columns, beams and plates always have a number of imperfections, such as i) variations in cross-sectional properties , ii) initial out-of-straightness, iii) residual stresses, iv) variation of material properties, and v) load eccentricity. It is difficult to take account of all the above imperfections in the formulation of design rules. This fact has increased the need for a large volume of good quality tests since such information is essential for the satisfactory calibration which insures an appropriate level of structural reliability. The larger database can justify a more reliable resistance factor.

Experiments are generally expensive and require much effort and good equipment to obtain accurate data. During the past forty years a large volume of structural tests have been conducted in many countries. However, most tests have been performed for various purposes using the deterministic approach, and reports of these works are fragmented and often published papers do not provide sufficient data. Collecting detailed test data and establishing this database on a world-wide basis is required.

Some experiments on steel columns have been carried out from a statistical viewpoint and a comparatively large volume of tests for pin-ended, centrally loaded columns is available. Computer simulations on steel columns were also performed. The European Convention for Constructional Steelwork (ECCS [2]) and the Structural Stability Research Council (SSRC [18]) have both independently recommended the multiple column curves to accurately formulate the column strength.

(2) Requirement of Numerical Database of Steel Structures

A database may be defined as a collection of interrelated data stored together without harmful or unnecessary redundancy to serve multiple applications. The computer industry is improving its

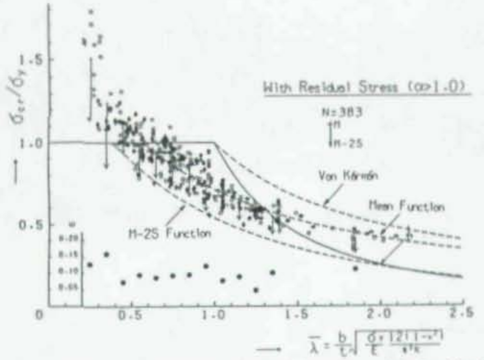

Figure 7: Test Results and Proposed Design Curves – with Residual Stress

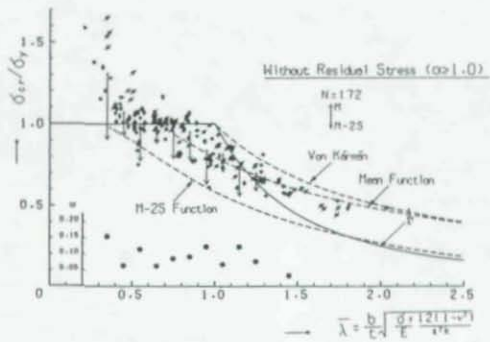

Figure 8: Test Results and Proposed Design Curves – without Residual Stress

plates with residual stresses, a nonlinear regression analysis with a uniform variance was used to develop the following mean function:

$$\begin{aligned} \sigma_{cr}/\sigma_y &= 1.133/\bar{\lambda} - 0.384/\bar{\lambda}^2 + 0.0468/\bar{\lambda}^3 \quad & for\ \bar{\lambda} \geq 0.658 \\ \sigma_{cr}/\sigma_y &= 1.0 & for\ \bar{\lambda} < 0.658 \end{aligned} \tag{4}$$

with the standard deviation $S = 0.104$. The $M - 2S$ curve is:

$$\begin{aligned} \sigma_{cr}/\sigma_y &= -0.208 + 1.133/\bar{\lambda} - 0.384/\bar{\lambda}^2 + 0.0468/\bar{\lambda}^3 \quad & for\ \bar{\lambda} \geq 0.337 \\ \sigma_{cr}/\sigma_y &= 1.0 & for\ \bar{\lambda} < 0.337 \end{aligned} \tag{5}$$

As shown in Fig.8, the M Function is close to the von Kármán curve, when $\bar{\lambda}$ becomes large. The proposed formula (Eq. (4)) agrees quite well with the mean value of the test points for $\bar{\lambda} > \bar{\lambda}_0 = 0.658$. The $M - 2S$ Function underestimates the test points for large $\bar{\lambda}$, since the uniform variance was assumed for much of the range.

capability, and the cost of data storage hardware is dropping more rapidly than other costs in data processing. It becomes cheaper to store data on computer files than to store them on paper.

Test data relating to the behavior of steel structures and structural components throughout the entire range of loading up to ultimate load are an essential requirement both for the preparation of design codes and for the verification of new theoretical approaches for predicting the resistance of structures.

The requirement for good quality test data is also expected to increase with the continuing advancement of powerful, computerized analytical methods since these require careful checking against well documented test results if they are subsequently to be used with confidence.

During the past forty years a large volume of structural tests has been conducted in laboratories both in Japan and overseas. However, reporting of this work is fragmented and often published papers do not provide sufficient data for the purpose outlined above. It is felt that the solution to this would be the establishment on a world wide basis of a computerized database containing detailed results of these tests. The specific benefits that would derive from this include: (a) Prevention of experimental data from being scattered and lost. (b) Effective use of data through the identical evaluation. (c) Rapid retrieval, easy processing and graphic presentation of needed data. (d) Availability to specification writing bodies of all relevant test data. (e) Avoidance of expensive duplication of testing effort. (f) Availability to researchers engaged on theoretical work of well documented test histories . (g) Encouragement to researchers engaged on experimental work to present their results in a consistent manner.

This paper aims at providing a basic framework for the database NDSS (Numerical Database for Steel Structures) regarding the ultimate strength of steel structures and structural components, and presents applications of NDSS and new approaches for dealing with test information and numerical analysis information [4-10, 12-14].

2. OUTLINE OF NDSS

(1) Instrumentation and Data Processing Flowchart of the NDSS System

Instrumentation for the NDSS system is divided into three blocks as follows: (a) Data sources from laboratories and available documents, (b) Instrumentation for data conversion and for input and output of data installed in the Civil Engineering Department. (c) A large computer system at the University Computation Center for source data filing, searching, retrieval, and processing of data.

Data editing, searching, retrieval and management can be done on the university computer system. A part of processing of original data can be performed by using personal computers and workstations.

Figure 1 shows a flowchart of the data processing on the NDSS system. Data collection for original data files may require direct contact with researchers to obtain original test data (first material). Machine readable data can be fed directly into the computer and the data are then stored in original data files through editing programs. Data which may be found in published documents (second material) can be converted into machine readable data and fed into the computer for original data files.

Database files for the NDSS system consist of: (1) main data files for individual items, i.e., mechanical properties of steels, columns, beams, plates and (2) a group of branch files under each of items. The test data relating to plate girders, welded joints for fatigue, beam-columns subjected to cyclic loading have been also dealt with after 1985. Detailed contents of database files will be explained later. In the numerical database, users may often wish to view test data not only in its original format but also in a prescribed format so that users can compare test data with their own theoretical predictions or with proposed design formulas.

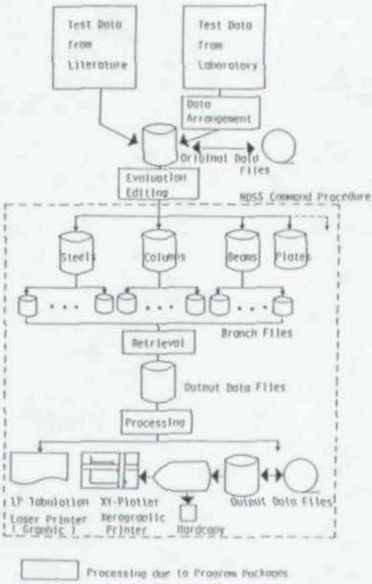

Figure 1: Flowchart of Data Processing in NDSS

Various computational and statistical manipulation programs must be available and these programs may be required for use with any document in the system that contain numerical data. These include a graphics package that permits the presentation of certain data in the form of plots either from a plotter or on screen.

Program packages for output data files presently available in the NDSS system are as follows:

i) Statistical Analysis

(a) statistical assessments of data: mean, standard deviation, coefficient of variation, etc., (b) construction of histograms, (c) estimation of probability distribution functions and goodness-of-fit tests (χ^2-test, Kolmogorov-Smirnov test), (d) regression analysis, and (e) occurrence of random variables.

ii) Analytical Interpretation:

(a) calculation of cross sectional properties of test specimens, (b) data conversion to specified parameters such as buckling parameter, non-dimensional buckling strength, (c) structural analysis, and (d) reference loads calculated by design formulas.

iii) Graphics:

(a) histograms, cumulative distribution curves and fitted probability distribution curves, (b) scattergrams, (c) data plot on various probability papers, (d) graphical presentation of load-displacement relationships, initial crookednesses, residual stress distributions, etc., and (e) plot of test data on various specified buckling strength curves.

Information retrieval system of the NDSS system is developed primarily for on-line processing. Off-line batch processing is also possible in the system. Through time-sharing, the on-line system can be used for searches in which users need data information immediately at a terminal in a conversational style.

(2)Installed Test Data in the NDSS

Material Properties of Steels [4,5]

Material properties are very important parameters to evaluate the strength variation of steel structures [11]. Following mechanical properties obtained from tensile coupon tests are stored in the main database file: (a) upper yield stress σ_{yu}, (b) lower yield stress or static yield stress σ_{yl}, (c) ultimate tensile stress σ_y, (d) yield strain ϵ_y, (e) modulus of elasticity E, (f) initial strain hardening modulus E_{st}, (g) initial strain hardening strain ϵ_{st}, (h) Poisson's ratio ν, and (i) elongation Δl. Original data stored in the database are listed in Table 1.

Table 1: Statistical Data of Mechanical Properties

	N	M	S	ω	Good-fit Distributions (1), (2), (3)
σ_{yu}/F_y	174	1.394	0.161	0.115	Gumb, Log
σ_{yl}/F_y	2054	1.168	0.130	0.111	Gumb, Welb, Log
σ_u/F_u	1619	1.092	0.068	0.063	Nor, Beta, Weib
ϵ_y/ϵ_{yn}	253	1.317	0.173	0.131	Log, Gumb
E/E_n	1024	0.999	0.045	0.045	Log, Nor
E_{st}	265	0.038	0.011	0.289	Nor, Weib
ϵ_{st}	281	20010	4500	0.225	Weib, Nor
ν/ν_n	588	0.937	0.085	0.091	Gumb, Log
$\Delta l/\Delta l_n$	667	1.534	0.302	0.197	Nor, Beta

Nor = Normal distribution
Log = Log-normal distribution
Beta = Beta distribution
Gumb = Gumbel distribution
Weib = Weidbull distribution

Steel Columns [6,7]

A total of 1665 individual test results is categorized by cross-section type and country of origin as given in Table 2.

The test data on each specimen consist of the following items: (a) maximum load, P_{max}, (b) measured cross sectional area, $(A)_a$, (c) nominal cross sectional area, $(A)_n$, (d) measured yield stress, $(\sigma_y)_a$, (e) nominal yield stress, $(\sigma_y)_n$, and (f) slenderness ratio about the test axis. The following test information which belongs to the individual column is also stored in the database files with items (a) to (e) above:

(i) Type of profile: I-shape, box shape, circular tube shape, square tube shape, solid round shape, T shape. name of profile - IAP150, 8WF31, etc. (71 names). (ii) Fabrication process: before assembly — flame-cut, shear-cut , universal mill; during assembly — rolled, welded, drawn, cut-out forming: after assembly — as-delivered, annealed, cold straightened. (iii) Geometry: ordinal section, heavy section; height h / width b; cross sectional dimensions. (iv) Yield stress: steel grade; testing method — stub column test, tensile coupon test. (v) Buckling axis: major axis, minor axis. (vi) Failure mode: flexural buckling, torsional buckling, local buckling. (vii) Source reference:

Table 2: Number of Column Data

Type of Profile		ECCS	USA	JAPAN	TOTAL
	Rolled	502	55	87	644
H or I	Welded	22	31	229	282
	Other	-	5	4	9
Box	Welded	74	14	41	129
Square	Rolled	67	-	-	67
Tube	Welded	*120*	-	-	*120*
Circular	Rolled	99	-	4	103
Tube	Welded	40	-	145	185
Circular	Solid	-	26	-	26
T Shape	Rolled	80	-	-	80
	Riveted	14	-	-	14
Composite		-	6	-	6
Total		1018	137	510	1665

Table 3: Number of Beam Data in NDSS

	Rolled Beams	Welded Beams	Plate Girders	Total
Japan	240	152	28	420
U.K.	35	0	14	49
U.S.A.	31	2	0	33
West Germany	27	0	0	27
Australia	15	0	0	15
Total	348	154	42	544

original paper; country in which testing was conducted. (viii) Initial crookedness. (ix) Column curve according to: ECCS column curves (a_0), (a), (b), (c) and (d); SSRC column curves (1), (2) and (3). (x) Identification: column test index; sequential data number.

Table 4: Number of Plate Data

Type of Plate	Europe	North America	Japan	Total
Single Plate	362		55	417
Welded Square Box	74	8	93	175
Square Tube			49	49
Welded Rectangular Box	20		6	26
Rectangular Tube			22	22
Cruciform	20	12	72	104
Total	476	20	297	793

Steel Beams for Lateral-Torsional Buckling [8]

A total of 544 individual test results is categorized by fabrication process, cross-sectional type and country of origin as given in Table 3. From a deterministic approach, a small number of beams was tested in each institute except the authors' tests. A hundred and forty-three beams were tested from a standpoint of statistical considerations by the authors.

The test data on each specimen consist of the following items: (a) maximum moment, M_u, (b) fully plastic moment, M_p, (c) yield moment, M_y, and (d) elastic critical moment, M_E. The following test information which belongs to an individual beam is also stored in database files with items (a) to (d) above. 1) Type of profile: monosymmetric cross section, doubly symmetric cross section, with or without vertical stiffener, with or without horizontal stiffener; name of profile — 250UB37.2, IPE200, etc. (58 names). 2) Geometry: measured and nominal cross-sectional dimensions such as

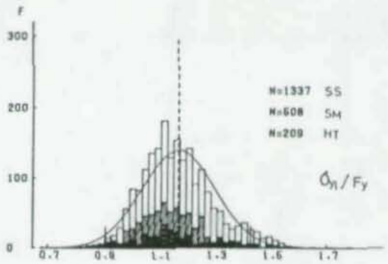

Figure 2: Histograms of Lower Yield Stress

beam height h, flange width b, flange thickness t_f and web thickness t_w. 3) Fabrication process: during assembly — rolled and welded; after assembly — as-delivered, annealed. 4) Yield stress: measured and nominal yield stress. 5) Loading and boundary conditions. 6) Initial imperfections: residual stresses and initial crookednesses. 7) Source reference: original paper, country in which testing was conducted. 8) Identification: beam test index and sequential number. The elastic critical moment, M_E, of the laterally supported beam is calculated by the method of Ref. [1] when exact M_E is not available. Load deflection curves, load-strain curves, material properties and initial imperfections which were obtained by the authors' tests ($N = 164$) are completely stored in database files.

Steel Plates [9,10]

The test data of uniformly compressed plates with clearly defined support conditions may be categorized by the cross-sectional shapes and the support conditions as (a) single plates, (b) square boxes and (c) cruciform shapes. Test data for rectangular boxes are also stored for comparison with the square box test results. Tubular box sections are also surveyed and included in the database. The data for the compression flange of H and box beams are not included because of the small number of test data and unidentified support conditions.

Table 4 shows a total of 793 individual test results which is categorized by cross-sectional shapes, fabrication process, method of testing and country of origin. In Europe, 476 tests are further classified into 323 of U.K., 148 of West Germany and 5 of Sweden.

The database of plates consists of a) σ_{cr} = maximum average compressive stress, b) σ_y = yield stress, c) width-thickness ratio with the following additional items: (a) Cross-sectional shape — single plate, square box column, rectangular box section and cruciform section. (b) Dimensions of plate — thickness, width, length, aspect ratio and cross-sectional area. (c) Fabrication method — welded built up, cold-formed, mechanical cutting, gas cutting; After treatment — as-delivered, annealed and gas heated. (d) Yield stress — nominal yield stress, measured yield stress, method of test (compression or tension). (e) Support condition — unloaded and loaded edges (simple clamped, free and others). (f) Initial imperfections — measured residual stresses and measured initial out-of-flatnesses. (g) Other measured items — load-average strain curve and average strain at the maximum load. (h) References — reference number, reference and the country where the test was conducted. (i) Identification - name of specimens and sequential number.

3. APPLICATIONS OF NDSS [4,5]

(1) Material Properties in NDSS

Table 1 also summarizes the statistical information on material properties of all the test data and goodness-of-fit distribution curves in descending order as tested by the Kolmogorov-Smirnov method.

Table 5: Comparison of Test Results and SSRC Multiple Column Curves

Type of Profile	Symbol	Exp)$_a$/SSRC(1) in Subgroup SSRC(1)		
		N	M	M-2S
Rolled H	△	13	1.079	0.806
Welded H	▲	20	1.054	0.833
Welded Box	■	31	1.037	0.888
Circular Box	○	100	1.028	0.716
Square Tube	□	84	0.998	0.806
Circular Solid	●	26	0.997	0.818
Total		274	1.021	0.775
Type of Profile	**Symbol**	**Exp)$_a$/SSRC(2) in Subgroup SSRC(2)**		
		N	M	M-2S
Rolled H	△	590	1.198	0.870
Welded H	▲	68	1.084	0.820
Welded Box	■	98	1.121	0.782
Circular Tube	○	94	1.192	0.972
Total		850	1.179	0.857
Type of Profile	**Symbol**	**Exp)$_a$/SSRC(3) in Subgroup SSRC(3)**		
		N	M	M-2S
Rolled H	△	15	1.206	0.682
Welded H	▲	16	1.132	0.724
T Shape	T	94	1.379	0.943
Total		125	1.327	0.848

The statistical distribution of σ_{yl}, which is very close to the static yield stress , may be best fitted to 1) Gumbel, 2) Weibull and then 3) Log-normal distributions. The σ_{yl} histogram is in good agreement with Gumbel's in the peak range and with Weibull's at both tails.

Steels can be roughly divided into three groups from the Japanese Industrial Standard (JIS), i.e., (a) SS (structural steels), (b) SM (weldable steels) and (c) HT (high strength steels). Figure 2 shows a histogram of test data for lower yield stress σ_{yl} non-dimensionalized by nominal values guaranteed for each steel grade. The histogram is for three different steels. In Fig. 2, a vertical dotted line represents the mean value and a fitted normal distribution curve is also given for all the test data. The mean value of lower yield stresses $(\sigma_{yl})_m$ is:

$$(\sigma_{yl})_m = 1.168F_y \quad and \quad COV \ \omega = 0.111 \tag{1}$$

for all the test data.

There are 7% of the specimens which have lower values than the minimum specified value. This may be due to the difference of test methods. Since the data in the NDSS were gathered from structural laboratories, the strain rate of loading may be almost zero. Minimum specified values in the JIS may guarantee upper yield stresses.

(2) Steel Columns [6,7]

Comparison between the SSRC Multiple Column Curves and Test Results

The SSRC multiple column curves have been categorized into three groups designated as (1), (2) and (3) in the third edition of the Guide (1976).

The following are results of comparisons between the tests and the appropriate SSRC column curves.

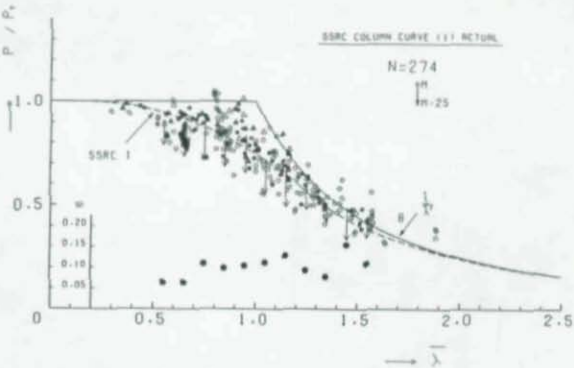

Figure 3: Test Results Compared with SSRC Curve (1)

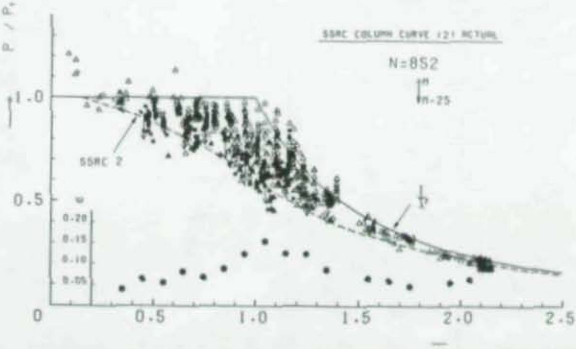

Figure 4: Test Results Compared with SSRC Curve (2)

SSRC column curve (1): Figure 3 shows 274 of the tests that would be allocated to the SSRC column curve (1). The test results are non-dimensionalized using actual yield load. The statistical results of the ratio of the test results to the SSRC column curve (1) are summarized in Table 5. From a comparison between Fig. 3 and Table 5, the SSRC column curve (1) clearly represents the mean value of the test points for much of the range of $\bar{\lambda}$. Table 5 indicates that variation in cross-sectional profile appears to have little effect on the mean column strength. (M-2S) values of all the test points over each 0.1 interval of $\bar{\lambda}$ almost coincide with the minimum envelope of the test points.

SSRC column curve (2): Figure 4 shows 852 of the tests that would be allocated to the SSRC column curve (2). From a comparison between Fig. 4 and Table 5, the test results tend to lie above the SSRC column curve (2). The results from rolled H-sections and circular tubes are the highest of all the test points.

It is obvious from the above comparisons that SSRC column curve (1) is close to the mean of the test data allocated to curve (1), and SSRC column curves (2) and (3) are, however, close to

Table 6: Comparison between Test Results and Various Design Formulas

	With Residual Stress				Without Residual Stress				
	N	M	S	ω	N	M	S	ω	$\bar{\lambda}_0$
Eurocode No.3	249	1.104	0.132	0.119	104	1.269	0.166	0.131	0.70
(draft, 1983)	383	1.089	0.155	0.143	172	1.196	0.192	0.161	
DASt Ri012	248	0.964	0.164	0.170	104	1.102	0.181	0.164	0.70
	383	0.998	0.179	0.179	172	1.095	0.179	0.164	
EDIN 18800/3	252	1.000	0.103	0.103	106	1.139	0.123	0.108	0.69
	383	1.022	0.142	0.139	172	1.119	0.148	0.132	
BS 5400/3	321	1.152	0.177	0.102	136	1.280	0.160	0.125	0.52
	383	1.156	0.135	0.117	172	1.257	0.172	0.137	
SIA 161	156	0.813	0.079	0.097	70	0.941	0.087	0.092	0.90
	383	0.914	0.170	0.186	172	0.999	0.146	0.146	

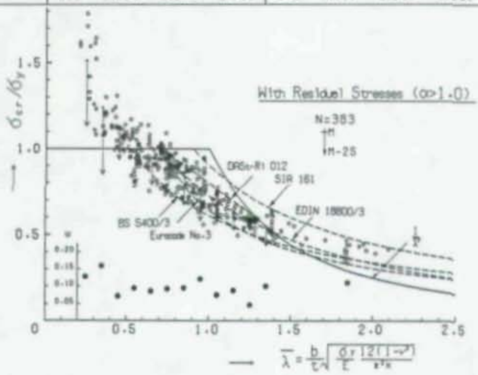

Figure 5: Test Results with Residual Stress and Various Design Curve

the lower bound of the test data allocated to curves (2) and (3), respectively.

(3) Steel Plates [9,10]

Comparison between Various Design Formulas and Test Results

The test data for single plates and square boxes are categorized into two groups depending on the presence of the residual stresses. Figures 5 and 6 show the test results and the various design formulas. Table 6 summarizes statistical results for mean (M), standard deviation (S) and coefficient of variation (ω) of ratios of the test results for single plates and square boxes to various design formulas. In Table 6, the upper line indicates the results obtained when only the test data (N is given in the table) which fall into the range of $\bar{\lambda} \geq \bar{\lambda}_0$ are used where $\bar{\lambda}_0$ is the limiting slenderness parameter for which $\sigma_{cr} = \sigma_y$ according to each formula. The lower line indicates the results obtained using all the data.

The mean ratios of $M > 1$ for BS5400/3 and Eurocode No.3 indicate that the two formulas represent somewhat lower curves when compared with the test data with and without residual stresses. The EDIN 18800/3 formula also locates by the lower bound of test data without residual stresses. On the other hand, $M < 1$ for SIA 161 , indicate that the formula represents the upper bound for the test data. The EDIN 18800/3 formula for the data with residual stresses and the DASt Ri012 for the data without residual stresses are clearly close to the mean values.

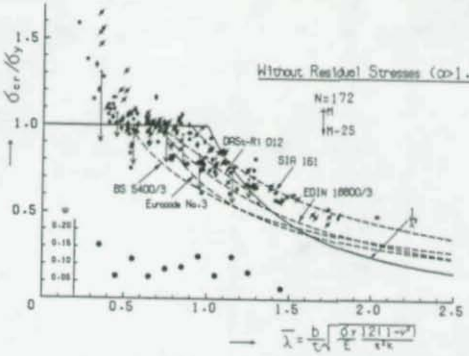

Figure 6: Test Results without Residual Stress and Various Design Curve

Suggestion of New Design Formulas

Since there were significant losses and variations in strength due to the presence of residual stresses, plate strengths are categorized into two groups depending on the presence of residual stresses. Ultimate plate strengths for aspect ratio $\alpha < 1$ are not analyzed hereafter because of insufficient test data.

Figure 7 shows results of 383 plate tests with residual welding stresses. Test data in this category are for single plate with weld beads along the unloaded edges, and for as-welded box columns. In Fig. 7, the nearly equal length of the $2S$ arrows in the range of $\bar{\lambda} > 0.5$ may assume uniform variance of the test data through this range. A nonlinear regression analysis with an assumed uniform variance was therefore performed to obtained a mean curve of the test points. This mean curve is

$$\begin{aligned} \sigma_{cr}/\sigma_y &= 0.968/\bar{\lambda} - 0.286/\bar{\lambda}^2 + 0.0338/\bar{\lambda}^3 \quad && for\ \bar{\lambda} \geq 0.571 \\ \sigma_{cr}/\sigma_y &= 1.0 && for\ \bar{\lambda} < 0.571 \end{aligned} \tag{2}$$

and the standard deviation which is a root of conditional variance is $S = 0.0871$. The mean minus two standard deviation curve is expressed as,

$$\begin{aligned} \sigma_{cr}/\sigma_y &= -0.174 + 0.968/\bar{\lambda} - 0.286/\bar{\lambda}^2 + 0.0338/\bar{\lambda}^3 \quad && for\ \bar{\lambda} \geq 0.389 \\ \sigma_{cr}/\sigma_y &= 1.0 && for\ \bar{\lambda} < 0.389 \end{aligned} \tag{3}$$

The dashed curves in Fig. 7 represent the mean curve (Eq. (2)) and the mean minus two standard deviations curve, (Eq. (3)). From Fig. 7, the proposed formula (Eq. (2)) agrees quite well with the mean value of the test points for $\bar{\lambda} > \bar{\lambda}_0$ for all the sub-groups except for the single plates with the unloaded clamped edges. The $M - 2S$ Function is close to the lower bound of the test data and even close to the Euler curve $\sigma_{cr}/\sigma_y = 1/\bar{\lambda}^2$ when $\bar{\lambda}$ becomes large. The limiting slenderness parameter $\bar{\lambda}_0 = 0.389$ for the $M - 2S$ function corresponds closely to the lower bound of the scattered test points in the vicinity of $\sigma_{cr}/\sigma_y = 1$.

Figure 8 shows results of 172 plate tests without residual stresses. As-cut and annealed single plates and annealed square box sections are included in this category. In the same way as for the

Table 7: Comparison between Conventional Computation and Knowledge-Based Techniques

	Conventional Computation Techniques	Knowledge-Based Techniques
Main Computation Method	Numerical Computations	Logical Computations
Know-How Representation	Programs (Algorithms)	Knowledge Base (if-then rules, etc.)
Data Used in the Processing	Numbers	Heuristical Knowledge (Text, Symbols)
Effort Needed in the System Updating	Large (Program Modification)	Small (Adding or Correcting Knowledge)
Processing Type	Batch Processing (Large Volume of Numerical Computations)	Interactive Processing (Question-Answer Loop)

4. NEW APPROACH FOR TEST INFORMATION

(1) Knowledge-Based System for Information Distribution Using PC [12,13]

Most of the data in NDSS, except the references and some other data, have a specific format. These data are organized in a relational database. However, the relational database can not represent the target of the test or the test method. In addition, the tests related to earthquake research such as the cyclic buckling test and hybrid test have a large number of parameters for each test and it is difficult to represent these data in a relational database. Therefore, the knowledge and experience of the specialists are included in three prototype knowledge-base systems that are easy to maintain. These systems can be considered as intelligent database systems. Table 7 compares the characteristics of the conventional computation techniques and the knowledge-based techniques.

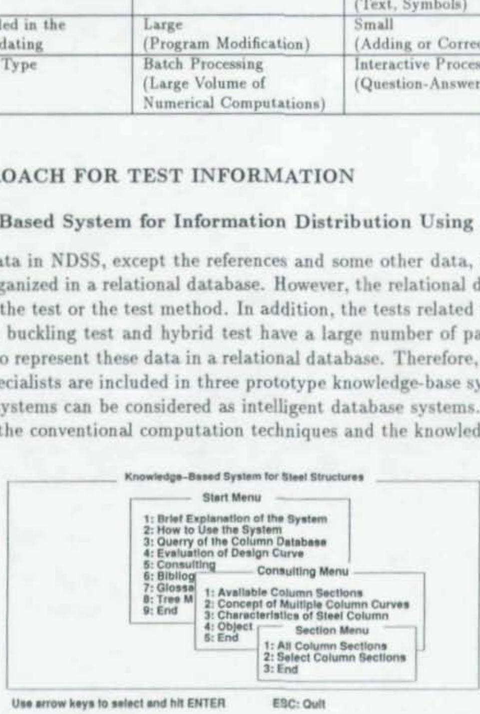

Figure 9: Multi-Window System

Because personal computers (PC) are becoming widely available, there is an increasing demand of accessing the structure tests data directly using the PC. Therefore, the database already established for the main-frame is transferred to the PC. In addition, the knowledge base approach is exploited not only to represent the numerical data, but also the target of the test, the reference, the basic resistance of different specifications, the explanation of technical terms, and other knowledge related to the test. The new system can be considered as a sub-system of NDSS and it is called *Knowledge-Based System for Steel Structures* (KBSS). Although many commercial shells can be used to facilitate the development of knowledge-based systems, these shells do not satisfy the database retrieval and graphical presentation requirements of KBSS. Therefore, this system is developed mainly using Prolog language. Graphics processing and procedural functions are implemented in C language. Figure 9 shows the multi-window and menu-driven screen of the system.

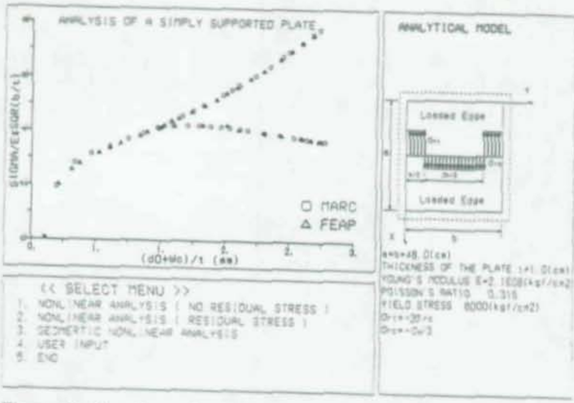

Figure 10: Non-linear Analysis of Steel Plate with Residual Stress

(2) Integrated System for Numerical Analysis Data and Test Data [14]

This system has, in addition to the knowledge about the strength, the deformation behavior, and the ductility of steel structures, the numerical analysis data that can be useful for educational and practical purposes. The results of FEM analysis depend on whether the analysis is elastic or non-linear. The evaluation of the precision of the results is difficult and needs deep understanding of structural engineering and non-linear analysis. In this system, many valuable test data and the related numerical analysis results are included in the knowledge base. In addition, non-linear analysis results of standard structural members and structures are organized in a user-friendly form. Figure 10 shows the geometrical non-linear analysis and compound non-linear analysis results of a steel plate with residual stresses.

(3) Processing System for Earthquake Test Data and Other Tests [3,19]

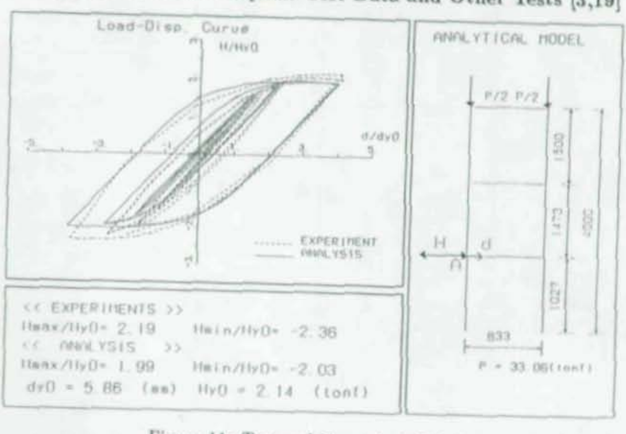

Figure 11: Test and Numerical Calculation

In addition to the unified format database of test data of structural members, a prototype knowledge-based system for earthquake test and other individual cyclic loading tests is developed. In this system, image processing is used to show the position of the measurement points. Figure 11 shows the cyclic loading of a tower and Fig. 12 shows the hybrid test results of the local buckling phenomenon of a bridge pier.

All of the previous systems are developed on a personal computer. However, a new system that will integrate these systems is being developed on a workstation.

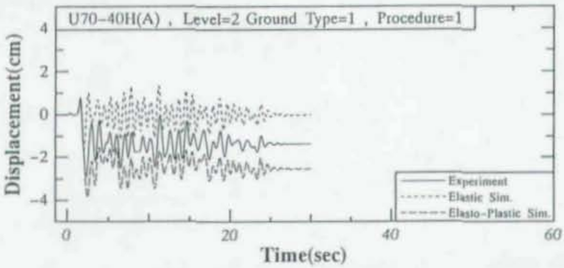

Figure 12: An Example of Hybrid Test Result

5. CONCLUSIONS

i) A numerical database system was built for material properties of steels, steel columns, steel beams and unstiffened steel plates, etc. A small database management system of the relational database was also developed.

ii) Strength variations of material properties, columns, beams and plates were made clear from the database approach. The mean value and the coefficient of variation of each member were obtained from test data and compared with existing design formulas.

iii) A structural test information-based system was developed that included database functions, test data evaluation functions, reference retrieval functions and technical terms explanation functions. The usefulness of the knowledge base approach for representing the test data was proved.

iv) An intelligent database prototype system for the ultimate loading test was developed using the knowledge engineering approach. The potential and problems of this system were clarified.

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WINTER'S BRACING APPROACH REVISITED

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Introduction

Most practical bracing formulations used in design of columns and beams are based on the principles developed by Winter (1958). He developed a simple rigid link model with fictitious hinges at the brace joints to calculate the bracing stiffness and strength requirements for columns and beams. The purpose of his development was, and I quote from his paper,

"to devise a single method which permits:
(1) To calculate a safe lower limit (rather than an exact value) of the necessary rigidity of the bracing such that the strength of the braced member will attain its maximum possible value, and
(2) For a bracing of rigidity equal to or larger than so calculated, to determine a safe, lower limit (rather than an exact value) of the strength required of such bracing."

His paper concentrated on requirements for "full bracing" which was defined or interpreted as

1. Buckling is forced to occur between braces
2. The brace is equivalent to an unyielding support

In this paper Winter's approach to design of equally spaced beams is reviewed and his methods are extended to cases when less than full bracing is provided and when braces have unequal spacing. Full bracing will also be redefined.

Basic Concepts

The exact relationship between column load and brace stiffness for the case of a uniform straight column with a brace at mid-height (Timoshenko, 1961) is shown in Fig. 1. The column buckles between braces when $\beta_i = 2P_e/L$ where β is the brace stiffness, L is the distance between braces and $P = \pi^2 EI/L^2$, the load corresponding to buckling between the braces. At brace stiffnesses less than the ideal

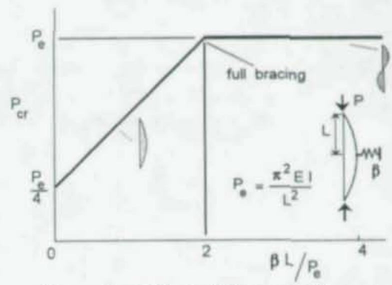

Figure 1 Effect of Brace Stiffness

value, β_i, buckling will occur with lateral movement at the brace point. However, the buckling load, P_{cr} increases almost linearly with increases in brace stiffness until full bracing is reached. For $\beta \geq \beta_i$, the column will buckle between the supports. The exact relationship between P_{cr} and β for $0 \leq \beta < \beta_i$ is a trigonometric function that is too complex for general design purposes (Timoshenko, 1961). Since the brace stiffness requirements for full bracing are small, Winter concentrated only on this particular bracing stiffness. To calculate the ideal stiffness, Winter developed a rigid-link model with a fictitious hinges at the brace points as illustrated in Fig. 2 for the case of a single brace. Multiple braces will be discussed later. Displacing the structure with the axial load P an amount Δ, cutting the structure at the hinge at mid-height and satisfying equilibrium gives $P\Delta = L\beta\Delta/2$. The Δ cancel and taking $P = P_e$ at full bracing, the $\beta = \beta_i = 2P_e/L$ which corresponds to the β_i shown in Fig. 1. Winter credits Fr. Bleich with introducing the concept of the use of fictitious hinges to calculate the ideal stiffness. This approach is applicable to any number of equally spaced braces. The number of braces does affect the value of β (Timoshenko, 1961; Winter, 1958).

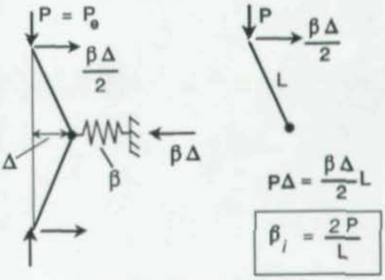

Figure 2 Winter's Model

Prior to the publication of his 1958 paper, Winter and his students conducted *extensive studies on the bracing problem, both experimental and theoretical* (Green et al., 1947; Zuk, 1956). Since the test columns were not perfectly straight, the theoretical work concentrated on members with initial out-of-straightness. With this background of experiments and exact elastic theory, Winter observed that braces with $\beta = \beta_i$ were not adequate for real columns. He noted that for perfectly straight columns with full bracing there is no force in the brace even at buckling because there is no displacement at the brace point. Tests showed that brace forces develop so he extended the *rigid link* model with fictitious hinges to cases with column out-of-straightness.

For the column shown in Fig. 3 with an initial out-of-straightness, Δ_o, there will be additional deflection as load P is applied which can be determined by taking moments about point n

$$L\,\beta\,\Delta/2 = P\left(\Delta_o + \Delta\right) \qquad (1)$$

and

$$\beta = \frac{2P}{L}\left(1 + \frac{\Delta_o}{\Delta}\right) \qquad (2)$$

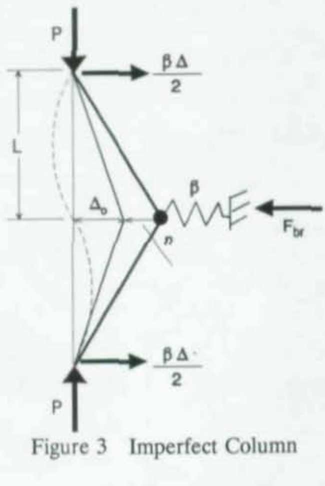

Figure 3 Imperfect Column

Substituting the total deflection $\Delta_T = \Delta_o + \Delta$ into Eq. (1) and noting that $\beta_i = 2P_e/L$ gives

$$\Delta_T = \frac{\Delta_o}{1 - \dfrac{2P}{\beta L}} = \frac{\Delta_o}{1 - \dfrac{P}{P_e}} = \frac{\Delta_o}{1 - \dfrac{\beta_i}{\beta}} \tag{3}$$

If $\Delta_o = 0$ and $P = P_e$, Eq. (1) gives $\beta = \beta_i$ the ideal stiffness. If $\beta = \beta_i$ and Δ_o is not zero, the heavy solid line in Fig. 4(a) shows the relationship between Δ_T and P given by Eq. (3). *For* $P = 0$, $\Delta_T = \Delta_o$. *When P increases and approaches the buckling load,* π^2EI/L^2, *the total* deflection Δ_T becomes very large. For example, when the applied load is within 5% of the buckling load, $\Delta_T = 20\Delta_o$. If a brace stiffness twice the value of the ideal stiffness is used, Eq. (3) gives much smaller deflections, as shown in Figure 4(a). When the load just reaches the buckling load, the $\Delta_T = 2\Delta_o$. For $\beta_L = 3\beta_i$ and $P = P_e$, $\Delta_T = 1.5\Delta_o$. The larger the brace stiffness, the smaller Δ_T.

For Fig. 3 and Eqs. (2) and (3), the force in the brace, F_{br}, is

$$F_{br} = \beta\Delta = \frac{2P}{L}\left(\Delta_o + \Delta\right) = \frac{2P}{L}\,\frac{\Delta_o}{1 - \dfrac{P}{P_e}} = \frac{2P}{L}\,\frac{\Delta_o}{1 - \dfrac{\beta_i}{\beta}} \tag{4}$$

which shows that the brace force is directly related to the magnitude of the initial imperfection. If a member is fairly straight, the brace forces will be small. Conversely, members with large initial out-of-straightness will require larger braces. A plot of Eq. (4) for an initial imperfection $\Delta_o = L_b / 500$ is shown in Fig. 4(b). If the brace stiffness is equal to the ideal stiffness, then the brace force gets very large as the buckling load is approached

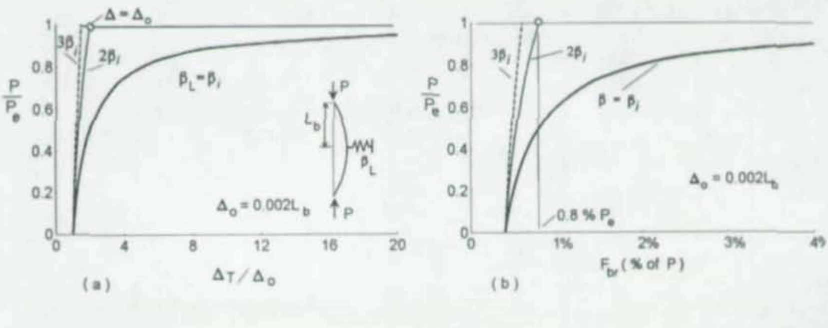

Figure 4

because Δ_T gets very large, as shown in Fig. 4(a). For example, at $P = 0.95P_e$, Eq. (4) gives a brace force of 7.6% of P_e. Winter noted that a brace system will not be satisfactory if the theoretical ideal required stiffness is provided because the brace forces get too large. If the brace stiffness is "over designed," as presented by the $ßL = 2ß_i$ and $3ß_i$ curves in Fig. 4(b), then the brace forces will be more reasonable. For a brace stiffness twice the ideal value and a $\Delta_o = L_b / 500$, Eq. (4) gives a brace force of only 0.8% P_e at $P = P_e$, not infinity as in the ideal brace stiffness case.

Winter's major contribution was his development of the interrelationship between brace stiffness and brace force. His emphasis on the importance of both properties is still widely ignored. Practicing engineers generally use strength alone, like a 2% rule, and researchers tend to examine perfect systems which give only stiffness criteria.

Expanded Winter Model

Winter developed the rigid-link fictitious-hinge model to calculate the ideal brace stiffness for full bracing, i.e. the bracing necessary to force the column to buckle between brace points. However, the model can also be used to determine the critical load on the column if the brace stiffness is less than the full bracing value. As an example, the P-ß relationship will be derived for a column with three intermediate braces.

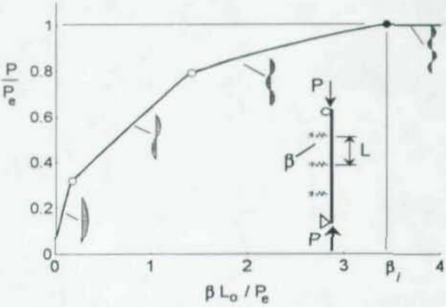

Figure 5 Column with Three Supports

The exact solution given in Fig. 5 was developed using a finite element program. It checks with the energy solution developed by Green (1947). With no bracing, $P_{cr} = \pi^2 EI/(4L)^2$. At low brace stiffness, the column buckles into a single (1st mode) wave. As the brace stiffness is increased, the buckled shape changes and additional brace stiffness becomes less effective. Full bracing occurs at $P_{cr} = \pi^2 EI/L^2$ when $ßL/P_e = 3.41$ which compares exactly with the value given by Timoshenko (1961).

The rigid-link model shown in Fig. 6 has three unknown displacement, Δ_B, Δ_C and Δ_D at each of the brace locations where fictitious hinges are shown. Taking summation of moments about point E gives $F_A = (ß/4)(3\Delta_B + 2\Delta_C + \Delta_D)$; similarly, $F_D = (ß/4)(\Delta_B + 2\Delta_C + 3\Delta_D)$. Cutting the structure at ß and summing moments gives

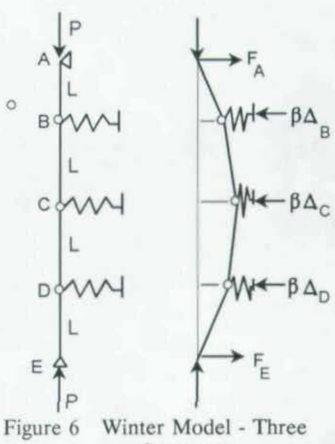

Figure 6 Winter Model - Three Supports

$$P_e\Delta_B = \frac{L\,\beta}{4}\left(3\Delta_B + 2\Delta_C + \Delta_D\right) \tag{5}$$

Similarly, cuts at C and D give

$$P_e\Delta_C = \frac{L\,\beta}{4}\left(2\Delta_B + 4\Delta_C + 2\Delta_D\right) \tag{6}$$

$$P_e\Delta_D = \frac{L\,\beta}{4}\left(\Delta_B + 2\Delta_C + 3\Delta_D\right) \tag{7}$$

Defining $X_1 \equiv \Delta_C / \Delta_B$ and $X_2 \equiv \Delta_D / \Delta_B$ and solving Eqs. (5), (6) and (7) simultaneously gives three solutions:

1. $X_1 = \sqrt{2}\ ;\ X_2 = 1$ and $\beta L / P_e = 0.586$
2. $X_1 = 0\ ;\ X_2 = -1$ and $\beta L / P_e = 2.000$
3. $X_1 = -\sqrt{2}\ ;\ X_2 = 1$ and $\beta L / P_e = 3.414$

Case 3 controls since it requires the largest brace stiffness.

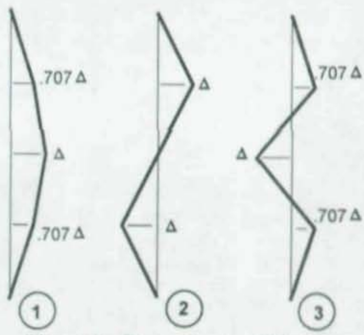

Figure 7 Three Roots - Winter Model

The buckled shapes corresponding to the three solutions are shown in Fig. 7. They are similar to the three modes shown in Fig. 5 before full bracing is reached. Winter only utilized the root that gave the required brace stiffness for full bracing. However, all the roots can be utilized to approximate the complete solution for any brace stiffness. The approach is illustrated in Fig. 8 in which the exact solution from Fig. 5 is shown as the solid line. The solid line can be approximated as follows with no bracing the column can support a load of P_e / 16 so $P_{cr} / P_e = 0.625$ (point 1). The brace stiffness required for the first mode shape is obtained from Case 1, i.e. $\beta L/P_e = 0.586$ at $P_{cr} / P_e = 1.0$ (point 2). A straight line is constructed, shown dashed, between points and 1 and 2. At zero brace stiffness the second mode starts at $P_{cr} = \pi^2 EI/(2L)^2$ or $P_{cr} / P_e = 0.25$ (point 3) and the Case 2 stiffness is $\beta L/P_e = 2.000$ (point 4). A

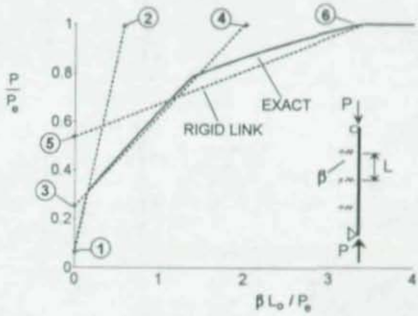

Figure 8 Winter Model Approximation

straight line connects points 3 and 4. For the third mode and no bracing, $P_{cr} = \pi^2 EI / (4L/3)^2$ and $P_{cr} / P_e = 0.5625$ (point 5) is connected to the Case 3 stiffness, $\beta L/P_e = 3.414$. The lower bound of the dashed lines defines the response which is a good approximation of the exact theory.

The rigid link model which has been used only to determine the full bracing stiffness requirement can also be used to construct the complete P-β relationship for any value of brace stiffness.

Unequal Bracing Spacing

The examples and illustrations used by Winter (1958) were all associated with members that had equally spaced braces. But this was done for simplicity. Lesser known are the following two sentences in his paper.

> *"By the same simple means of writing moment equations about the fictitious hinges at the braces it is also possible to analyze the required bracing for columns with unequally spaced supports and/or with cross-sections which differ in the individual spans. In this case it is merely necessary to introduce for each portion between braces its appropriate P_e depending on length and cross-section of that portion."*

Plaut (1993) has solved the problem of a column with a single brace at any location, as shown in Fig. 9a. The lengths are chosen to be consistent with the Plaut presentation. The column is divided into two segments, a_1 and a_2, with L defined as the total length. For a typical brace location, $a_1 / L = 0.4$, the Winter model is shown in Fig. 9(b). External equilibrium gives the external reaction at A and C as $0.6\beta\Delta$ and $0.4\beta\Delta$, respectively. Cutting the structure at the hinge gives

$$P\Delta = 0.6\beta\Delta(.4L) \tag{8}$$

or

$$\beta = 4.16\ P/L \tag{9}$$

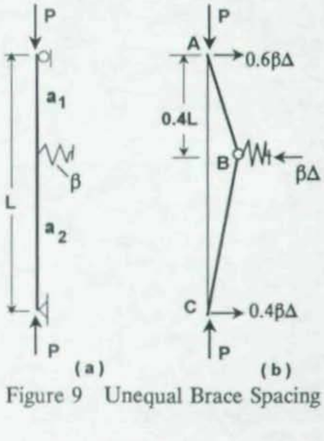

Figure 9 Unequal Brace Spacing

For this case $P_e = \pi^2 EI/0.6L^2$ which would cause buckling in the longest segment. Substituting P_e for P in Eq. (9) gives the ideal $\beta_i = 4.167(\pi^2 EI/(0.6L)^2) = 114.2\ EI/L^2$. Results for other values of a_1 / L are given in Table 1. The Winter results are exactly the same as those presented by Plaut for the brace stiffness required for column load to reach a level corresponding to Euler buckling of the longest unbraced segment. It may be surprising that Winter's solution is exact for this complex problem so some further comments are needed.

Table 1 Comparison of Winter and Plaut

a_1 /L	Winter $\beta L^3/EI$	Plaut k_e
0.1	135.4	135.4
0.2	96.4	96.4
0.3	95.9	95.9
0.4	114.2	114.2
0.5	157.9	157.9

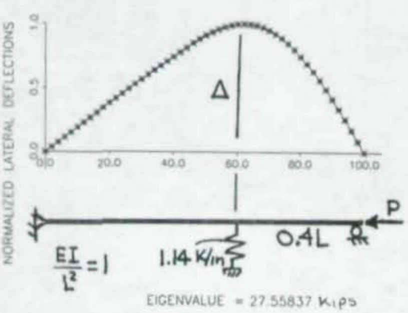

Figure 10 Buckled Shape

The Winter solution is exact only for the load level corresponding to an assumed Euler buckling load controlled by the longest segment. Of course this is exactly the maximum load that would usually be assumed in design. The column can support loads higher than this segmental Euler load because the shorter segment can provide some rotational restraint to the longer segment but this is usually ignored in design. If load levels higher than the Euler segment load are desired, greater bracing will be required as derived by Plaut (1993).

If the β_i as derived above is used, there will be movement at the brace point at the Euler load, as shown by the buckled shape in Fig. 10. The Winter model does not give results that correspond to a column load with an immovable support in this case. Winter's "full bracing" comments should more correctly be defined as the bracing to reach a load corresponding to an effective length factor of 1.0. If the support was immovable, the column could support a load 20% higher than the Euler load (Plaut, 1993) but the bracing requirement would be substantially increased.

Concluding Remarks

Winter's method was shown to provide a very general solution. It is important to note that the bracing requirements derived by the Winter approach are only equivalent to immovable supports when the problem is symmetric and the unbraced segments want to buckle simultaneously. However, Winter's solution is valid for all cases in which K = 1.0 is used in design,even when the bracing spacing is not uniform. Use of the Winter solution will enable critical column segments to reach the Euler load.

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EUROCODE 3 - THE EUROPEAN CODE FOR STEEL STRUCTURES

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INTRODUCTION

Perhaps the most important aspect of Eurocode 3 'Design of Steel Structures'[(1)] is that it is but one of the set of nine 'Structural Eurocodes' that between them cover Steel, Concrete, Composite Steel-Concrete, Timber, Masonry and Aluminium Alloy structures, Foundations, Loading and Earthquakes. This serves to illustrate the underlying vision of harmonizing structural design not only between different countries but equally between different materials and hopefully between different types of structures. Traditions and customs vary in all these three dimensions, so harmonization offers benefits in ease of understanding, cross-fertilization of ideas and reduction of errors due to misinterpretation.

Reviewing the many papers written on EC 3 over the last 15 years, it is salutary to find that whereas the administrative, political and legal framework within which it has been portrayed has gone through several metamorphoses, the philosophical and technical aspects have meanwhile progressed calmly from aspiration to implementation and the underlying vision, whilst sometimes bruised by events, has remained unchanged.

Eurocode 3 is undoubtedly the offspring of the ECCS 'Recommendations for Steel Construction'[(2)], the midwife being the Commission of the European Communities (CEC) assisted in the later stages by the General Secretariat of the European Free Trade Association (EFTA). The principal difference is that the ECCS document is a "code for code writers" whereas EC 3 has to be capable of superseding existing national design standards.

The whole Eurocode family has now been adopted by CEN, the European Committee for Standardization, the members of which comprise the 18 national standards bodies for all the countries in EFTA and the European Union (EU). Initially, the Eurocodes take the form of European Pre-standards (ENV), which means they can be used as optional alternatives to existing national standards. Feedback is encouraged so that they can later be issued as full European Standards (EN) to supersede existing national standards after a suitable overlap period, details of which are yet to be decided.

The influence of the Structural Eurocodes extends further, several other countries in central and eastern Europe, including some from the former Soviet Union and in South America, etc. having expressed interest in one form or another, and a number of those countries have joined the relevant CEN sub-committees as associate members. There has also been an interaction between EC 3 and the draft international standard for steel structures[(3)], recently released by ISO for discussion.

AIMS AND SCOPE

The main aim of EC 3 is to provide rules for the design of structural steelwork that produce safe and economical structures for the purpose and lifespan for which they are intended. This aim has remained from the beginning, but originally also encompassed rules for fabrication and erection, that have now been transferred to a separate standard for "execution".

The scope of EC 3 is buildings and civil engineering structures, the basic principles for which apply to steel structures of all types. At the time the work started there was much debate about the form of codes, including whether they should be general and all-embracing or whether there should be separate (though compatible) specialist codes, for example, for buildings and bridges. Whilst the intention remained to cover all but the most specialised structures in due course, drafting concentrated initially on a document that would fully cover the needs of building design. Its provisions would also be of general application for other types of structure, but not necessarily sufficient, and would need to be supplemented by auxiliary documents specific for bridges, masts and towers, etc.

The intention was to permit and encourage the use of all methods of design likely to lead to economies of material or of effort, whist retaining a form familiar to steel designers. Insofar as this itself varied between the traditions of the countries concerned, this required the synthesis of a new common European approach, that draws on the experiences of several countries but in some cases differs from all of them.

The decision to base all Structural Eurocodes on the principles of Limit States Design (which was new to designers in most of the countries concerned) provided both an imperative and an opportunity to introduce a harmonized approach.

It was also necessary to accommodate the needs of the drafters of EC 4 for composite steel-concrete design which had to draw upon, and link into, both EC 2 for concrete and EC 3 for steelwork, a process demanding strong technical co-ordination when the three documents were being drafted concurrently by three independent teams of experts with differing backgrounds and interests.

FORMAT

By the time the original EC 3 document(1) reached publication as ENV 1993-1-1 it had become Part 1.1 of a Eurocode 3 planned to have up to 7 parts and several sub-parts, but for convenience it is still generally called EC 3.

The contents as published are listed in table 1 and include 9 chapters plus 9 annexes. Table 2 lists 7 new or revised annexes currently in preparation for Part 1.1 and table 3 lists the 3 further sub-parts of Part 1 also at various stages of preparation. The further parts of EC 3 that are either under preparation or are planned for the future appear in table 4.

The safety format is that of limit states design using partial safety factors. It differs from American "Load and Resistance Factor Design" (LRFD) only in that, whereas LRFD multiplies resistances by factors less than unity, Eurocodes divide resistances by factors greater than unity, in accordance with ISO 2394(4).

Table 1: Contents of Eurocode 3: Part 1.1

Chapter 1	Introduction;
Chapter 2	Basis of design;
Chapter 3	Materials;
Chapter 4	Serviceability limit states;
Chapter 5	Ultimate limit states;
Chapter 6	Connections subject to static loading;
Chapter 7	Fabrication and erection;
Chapter 8	Design assisted by testing;
Chapter 9	Fatigue.
Annex B	Reference standards;
Annex C	Design against brittle fracture;
Annex E	Buckling length of a compression member;
Annex F	Lateral-torsional buckling;
Annex J	Beam-to-column connections;
Annex K	Hollow section lattice girder connections;
Annex L	Column bases;
Annex M	Alternative method for fillet welds;
Annex Y	Guidelines for loading tests.

Table 2: New or revised annexes for EC 3: Part 1.1

Annex D	The use of steel grades S 460 and S 420;
Annex G	Design for torsion resistance;
Annex H	Modelling of building structures for analysis;
Annex J	Beam-to-column connections - extended version;
Annex K	Hollow section lattice girder connections - revised version including multi-planar joints;
Annex N	Openings in webs;
Annex Z	Determination of design resistance from tests.

Table 3: Further sub-parts for EC 3: Part 1

Part 1.2	Structural fire design;
Part 1.3	Cold formed thin gauge members and sheeting;
Part 1.4	Supplementary rules for stainless steels.

Table 4: Further parts of EC 3

Part 2	Bridges and plated structures;
Part 3	Towers, masts and chimneys;
Part 4	Tanks, silos and pipelines;
Part 5	Piling;
Part 6	Crane structures;
Part 7	Marine and maritime structures;
Part 8	Agricultural structures.

Arising from debates on the nature and format of codes, such as the IABSE colloquium[(5)], all the Structural Eurocodes distinguish between "Principles" and "Application Rules".

Due to the need to establish technically equivalent versions in each relevant language, attention had to be paid to expressing provisions in "clear, unambiguous, translatable English" (CUTE). Producing the definitive text in English has obvious advantages for native English-speakers, but also some drawbacks. So that the code can be understood by designers not involved in its preparation, the sort of "International English" (otherwise known as "Euro-speak") used in committee has to be eliminated. At the same time the document has to remain clear to non-native English readers and this can lead to apparent wordiness. More particularly, clear and precise definitions are demanded for technical terms commonly used in a loose and imprecise manner. A whole new meaning has consequently had to be added to the English language for "Action" (load or imposed deformation).

Most controversially, in order to harmonize member axes for all structural materials, the convention commonly found in computerised structural analysis has been adopted for all Structural Eurocodes, with x along the member and y and z for cross-section axes, despite the long tradition of using x and y for the cross-section axes of steel sections.

CONTENT

General

Chapter 1 concerns the scope, definitions, symbols and axes. The use of symbols will be discussed later.

Basis of design

Chapter 2 is intended to be basically similar in all Structural Eurocodes and concerns the safety format, load cases, load combinations, safety factors, etc. including some simplifications applicable only for buildings. The design philosophy and limit state principles, including the derivation of partial safety factors, is discussed by Dowling and Chryssanthopoulos[(6)].

Materials

Chapter 3 covers materials. Steel grades S 235, S 275 and S 355 are covered (the numerical reference is the yield strength in MPa) and steel grades S 420 and S 460 will be added in Annex D. This Chapter also refers to Annex C for design against brittle fracture.

Serviceability limit states

Chapter 4 considers vertical and horizontal deflection limits, which are treated as advisory rather than mandatory, and also gives rules intended to limit the effects of rainwater ponding on roofs and the effects of vibrations on floors.

Ultimate limit states

Chapter 5 covers both the design of complete frames and the design of individual members for the ultimate limit states of static strength and structural stability.

Methods of analysis

Elastic or plastic global analysis may be used to calculate the internal forces and moments in a statically indeterminate structure. A distinction is made between first-order methods using the initial geometry of the structure and second-order methods that account for changes in shape of the structure under load. First-order methods may be used for braced frames and non-sway frames, and also when methods are used that make indirect allowances for second-order effects, such as amplified sway moment or Merchant-Rankine methods. Full second-order methods may be used in all cases, but of course, are not normally required except for sway frames.

Methods for plastic global analysis range from the commonly adopted rigid-plastic method to advanced computer-based elastic-plastic methods. Two forms of elastic-plastic analysis are distinguished. In the elastic/perfectly plastic method the members remain elastic until a plastic hinge has fully formed, whereas in the elasto-plastic method the spread of plasticity through the depth and along the length of a member is followed in an incremental computer-based analysis.

A special feature of the code is the treatment of semi-continuous frames as well as the more usual simple and continuous frames. In addition to the more widely known elastic global analysis of semi-contiuous frames with semi-rigid connections, Eurocode 3 covers the use of plastic global analysis for semi-continuous frames in building structures. This involves the consideration of partial-strength joints that can develop plastic hinges with a smaller plastic moment resistance than the members that they connect, yet with sufficient rotational stiffness to satisfy serviceability criteria an sufficient rotation capacity at the ultimate limit state to justify the use of plastic analysis.

The use of such joints can lead to worthwhile economies in construction, compared to simple connections. The joints themselves are introduced in the connections chapter and detailed design procedures are given in an annex.

Structural stability

The effects of imperfections are required to be taken into account in global analysis, beam bracing systems and members. Only geometric imperfections are given, but their values have been amplified to cover residual stresses, lack of fit and other practical imperfections.

Normally, the effects of imperfections on members are accounted for within the buckling strength formulae given in the code. In beam bracing systems, allowance is made for an initial bow imperfection, or initial lack-of-straightness, in the members to be restrained.

The effects of imperfections are dealt with in the global analysis of building frames by means of an initial sway imperfection that is a function of the number of columns and storeys. These sway deformations can be represented by equivalent horizontal forces. Allowance for their effects must be made in all load combinations including those involving wind forces.

Clear definitions of sway and non-sway frames are given in the code. A non-sway frame is one that is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes. The simple criterion "elastic critical load factor at least 10" is used to define "non-sway" frames.

A distinction is also made between braced and unbraced frames. A frame is said to be braced by another part of the structure, referred to as the "bracing system", if the bracing system reduces its horizontal displacements by at least 80%. This usage of the term "braced frame" means that a frame with cross-bracing has to be referred to as "triangulated".

All braced frames can be treated as non-sway frames, but some unbraced frames may turn out to be non-sway frames using the foregoing criterion, whereas a frame with a relatively narrow bay of cross-bracing may turn out to be so flexible that it has to be treated as a sway frame.

The code allows the use of simple rigid-plastic analysis for braced frames and for unbraced frames of up to two storeys in height, with Merchant-Rankine amplification if they are sway frames. Sway frames may be designed using rigid-plastic analysis, provided that a specific model (a more conservative variant of that proposed by R H Wood) is used and the related conditions are satisfied. Otherwise a second-order elastic-plastic sway analysis must be used.

Design of members

In the design of structural members, consideration needs to be given to cross-section resistance, buckling resistance of the member and where appropriate, shear buckling resistance and web crushing, buckling or crippling.

Cross-sections are divided into four classes. Class 1 cross-sections are ones for which it is possible to take advantage of the full plastic cross-sectional resistance and use rigid-plastic analysis with full moment redistribution. Class 2 cross-sections are ones that can develop their plastic resistance but have limited rotation capacity. Class 3 cross-sections are ones that can reach yield in their extreme compression fibre, but in which local buckling prevents the attainment of the full plastic resistance. Class 4 cross-sections are ones where appropriate allowances must be made for the effects of local buckling when determining their resistance.

The limitations on slenderness of the various elements of the cross-section have been selected from the best available data for which sufficient support could be found, but may be rather conservative in some cases. The concept of effective width is used to reduce the section properties of class 4 sections whilst using the full yield strength to calculate the elastic resistance of the effective cross-section. When applying interaction formulae for buckling to members with class 4 cross-sections, the effective cross-section can be determined separately for resistance to axial force and for resistance to bending about each of the principal axes, provided that the effective widths of the elements in compression in each case are based on the attainment of the yield strength in the effective cross-section. In addition the possible extra moments due to eccentricity of the axial force arising from any shift of the neutral axis of the effective cross-sectional area, relative to the neutral axis of the gross cross-section, must be taken into account.

Fastener holes in compression zones of the cross-section are neglected whereas the tension resistance is taken as the smaller of the plastic resistance of the gross cross-section and about 80% of the ultimate resistance of the net area. The shear resistance of a web need not be reduced due to bolt holes unless the ratio of net to gross area is less than the ratio of yield to ultimate strength. It is, however, necessary to check for block shear at the ends of a beam.

The effects of shear on the plastic moment resistance must be accounted for when the shear exceeds 50% of the plastic shear resistance; an appropriate expression is given for the reduced plastic moment of resistance.

Similarly, the effects of axial force on the plastic moment resistance must be accounted for when the axial force exceeds 25% of the axial resistance of the full cross-section or 50% of that of the web; again appropriate expressions are given for the reduced plastic moment of resistance.

Buckling resistance of members

The basis for the checks on buckling resistance of columns is the European Column Buckling Curves contained in the ECCS Recommendations(2) which have been derived from the statistical evaluation of test results of a large number of experiments on columns with different sections, production methods and steel grades. Four column curves *a, b, c* and *d* are given in Part 1.1 and a fifth a_0 curve is introduced in Annex D for grade S 460 steel only. The selection of the appropriate curve is based on the type of cross-section and axis of bending.

A generalised slenderness is calculated using the "buckling length" otherwise known as the effective length. For rigid-jointed frames this can be obtained from Annex E. From the generalised slenderness, the appropriate column curve gives a reduction factor which is applied to the compression resistance of the cross-section to get the buckling resistance of the member.

Lateral-torsional buckling resistance is calculated by a method that also refers to the same buckling curves, using an appropriate equivalent slenderness to determine the buckling resistance moment. Annex F may be used to arrive at this equivalent slenderness, which is a function of the elastic critical moment for lateral-torsional buckling of the beam, taking account of the type and level of the loading and the degree of end fixity and warping restraint, as well as the cross-sectional parameters. If the generalised equivalent slenderness is less than 0.4 no reduction in the resistance need be made to account for lateral-torsional buckling.

Shear buckling resistance

Unstiffened webs with depth-to-thickness ratios less than given values (e.g. 69 for grade S 235 steel) need not be checked for shear buckling. Where these limits are exceeded two methods may be used to check the shear buckling resistance. The first is a simple post-critical buckling approach and the second is a form of the "Cardiff method" which allows explicitly for tension field action. Both may be used for transversely stiffened webs and rules are also given for the design of the stiffeners, including checks on their compressive buckling strength and their flexural stiffness.

Built-up members

Built-up compression members such as laced or battened struts are treated in some detail. Rules are given for the design of the chords, lacing members and battens, based upon an analogous model of a member subjected to finite shear deformations, allowing for the effects of an overall initial bow imperfection in addition to the usual imperfections in the individual components.

Design of connections

Chapter 6 contains an extensive treatment of connection design for statically loaded structures because of the importance of this topic in relation to economical design. These provision draw heavily on the work of ECCS committee TC 10. Further details are given by Stark[7].

Bolted connections are divided into five categories that distinguish between connections loaded in shear and tension, and connections with preloaded bolts that are designed to resist slip. Advantage is taken of the larger deformations that can be allowed to occur in connections where rotation is required at the ends of beams. In the case of welded connections, advantage has been taken of the best information available for the design of fillet welds, both side and end, long lap joints and intermittent welds.

Beam-to-column joints, both with welded connections and with bolted end-plate connections, are treated in Annex J. This also contains information for the calculation of prying forces. A special feature of the code is the treatment given to semi-rigid and partial strength connections[7]. These methods have the potential to produce more economic design of frames by avoiding expensive stiffening of joints. Annex J provides methods for designing such connections.

Fabrication and erection

Chapter 7 now covers fabrication and erection only to the extent that such information is needed by the designer. This includes such practical matters as sizes of bolt holes, the tolerances assumed in developing the design rules and the points on which the designer may need to provide specific information in the Project Specification, such as areas where plastic hinges are expected to form and so special fabrication restrictions apply, for example to punched holes.

The actual fabrication and erection rules applicable in the workshop and on site, which appeared in early drafts, have now been transferred to a separate draft standard pr EN 1090 on 'Execution of Steel Structures'[8].

Design assisted by testing

Chapter 8 now covers the conditions under which testing may be used to obtain information for the design process. Recommendations for testing are given in the associated Annex Y.

Design against fatigue

The inclusion in Part 1.1 of chapter 9 covering fatigue is not meant to imply that ordinary building structures need to be checked for fatigue. Rather it appears in Part 1.1 as part of the general rules for design of steel structures that can be referred to from other future parts, but, of course, can also be used for those special cases in building design where fatigue is an issue. The rules are based on those produced by ECCS Committee TC 6[9].

Joints in hollow section lattice girders

Annex K has been produced in parallel with the recent recommendations of the international organisation for tubular construction CIDECT[10] and similar provisions related to North American practice have been given by Packer and Henderson[11].

Cold formed steelwork

The use of cold formed hollow sections, both circular and rectangular, is covered in Part 1.1 by slight variations in the rules for hot rolled hollow sections. Cold formed thin gauge members and profiled sheeting are the subject of a separate Part 1.3 due for issue this year. The draft for this was produced in parallel with the corresponding draft ISO standard and makes use of the work of ECCS committee TC 7 as well as that of AISI.

Fire resistance

Provisions for structural fire design are given in Part 1.2 which is also due for issue this year. They are based on the recommendations produced by ECCS committee TC 3[12].

SOME FEATURES

Principles and application rules

In adopting a harmonised presentation for structural design standards, taking account of the wide differences between current styles in various European countries, but avoiding copying any one of them, part of the solution has been to distinguish clearly between two types of paragraphs, those stating the fundamental requirements ("Principles") and those providing an acceptable means to satisfy these requirements ("Application Rules"). Roughly half the paragraphs are of each type, in contrast to Danish codes, which also make the same distinction but consist mainly of Principles. The opposite extreme is represented by British codes (for example) which basically comprise Application Rules. The resulting clarity of expression in the Eurocodes, avoids the need for a separate "commentary" stating the reasons for the rules, though background documents indicating their derivations are still appropriate.

Symbols

Symbols have not only been harmonised around the ISO recommendations[13], the consistent use of descriptive subscripts has been developed to the extent that, once familiarity has been gained, the meaning of any particular symbol in a given expression can generally be determined by inspection, without the need for a complete key listing the symbols which it uses. These self-explanatory symbols do, in effect, approach the concept of "ideograms".

For example, all internal forces and moments are distinguished by the subscript S whilst the corresponding resistance have the subscript R and all design (factored) values also acquire the subscript d. Thus knowing that M represents a moment, the expression:

$$M_{Sd} \leq M_{Rd}$$

represents the requirement that the design value of the internal moment (due to the applied loads) must not exceed the design value of the resistance of the component intended to carry it.
In the case of a Class 1 or Class 2 ("Compact") section this can be amplified to:

$$M_{Sd} \leq M_{pl,Rd}$$

to clarify that it is the plastic resistance moment that is relevant, and to:

$$M_{y,Sd} \leq M_{pl,y,Rd}$$

to clarify that it is the moment about the y-y axis that is of concern.

This system is not completely novel, but is of sufficient interest to deserve mention here. The authors first learnt of it from proposals attached to comments from the Netherlands on the draft of EC 3, but are uncertain of its origin. After initial reluctance, familiarity seems to convert most users to enthusiasm for the system, because it largely eliminates the need for explanatory words in calculations, such as "factored", "furnished" or "required".

Beam bracing systems

By considering an initial deformation in the bracing that provides lateral restraint to beams, as well as in the beams themselves, a simple model has been given in EC 3 which allows the restraint forces to be determined taking account of the flexibility of the bracing system and of its deformation under any independent transverse forces, such as those due to wind loads.

CALIBRATION

Statistical calibration studies have been carried out for many of the rules in Eurocode 3. Details are contained in Background Documents[14]. These studies aimed to ensure that a uniform safety level, defined by a safety index $\beta = 3.8$, was achieved throughout the code. The procedure followed is defined in a joint report[15] from TNO Delft, TU Eindhoven and RWTH Aachen. Subsequently, a JCSS Working Document[16] has also proposed essentially the same procedures.

Calibrations carried out in the course of developing and editing Eurocode 3 included amongst others:

- resistance moments of unrestrained beams
- static strength of bolts
- static strength of welds
- buckling of struts
- lateral-torsional buckling
- failure of columns with bi-axial moments
- effective section properties of cold-rolled sections
- use of high strength steel
- joints between structural hollow sections
- fatigue design rules

In short, every rule for which an adequate database of test results could be assembled has been submitted to the same procedure. The programme of calibration studies was organised by Professor G Sedlacek of RWTH Aachen in what is believed to be the most extensive exercise of its type so far carried out. The studies themselves were carried out in various centres including Delft, Liege, St Remy and Aachen. Similar procedures were also applied in the development of Eurocode 4 'Design of composite steel and concrete structures'(17).

In Eurocode 3, in order to avoid having a large variety of values of the partial safety factor for resistance γ_M, two representative categories were selected as follows:

1) Cases where failure is related to yield strength, including buckling phenomena
2) Cases where ultimate tensile strength governs, including net section failure and the strengths of bolts and welds

Fixed representative values of 1.1 and 1.25, respectively, were selected for these two categories. Where necessary, the coefficients in the design expressions were then adjusted slightly to compensate for variation from the optimum values.

TRIAL CALCULATIONS

Useability aspects of EC 3 were checked by trial calculations. Some were ad-hoc, but at the public enquiry stage some countries carried out complete design examples which were also compared to the use of their national codes.

Considerations of completeness and useability were the principal topics for complete design studies carried out as the final editing of EC 3 progressed, by CEDIC(18), the organisation of Consulting Engineering for the EEC countries, now superseded by EFCA which also covers the EFTA countries. A number of recently completed buildings in a variety of countries were chosen and the design was re-checked according to EC 3. These buildings were selected so as to be reasonably challenging in terms of testing the availability of appropriate code provisions, whilst also being reasonably representative of best modern practice.

The development of CEDIC's reports on these studies interacted with the development of the final editing of EC 3, so that comments made in their initial report were found to have already been acted upon, in terms of code improvement, by the time the final report was prepared. Thus the final conclusion could be reached that EC 3 is "sufficiently clear, transparent and comprehensible for practising engineers".

SIMPLIFIED VERSIONS

Several people, including the writers of the CEDIC report, have identified the need for measures to assist the introduction and acceptance of the Eurocodes into everyday design practice. Whilst the comprehensiveness and freedom of choice for the designer are welcomed, the need is seen to give simple guidance on the appropriate methods to adopt in common cases, at least until designers become more familiar with those options and features that are novel for them.

Essentials of Eurocode 3

The ECCS has published a shortened version of EC 3 "Essentials of Eurocode 3"[(19)] (E-EC 3). It is intended as a design aid to facilitate the use of EC 3: Part 1 during the ENV period, and contains only those rules "that are likely to be needed for daily practical design work". This has led to the omission of plastic analysis, second-order analysis and semi-rigid joints.

The E-EC 3 is intended to be used by designers who have studied at least the relevant portions of EC 3. It is intended to serve as an aide-memoire both for the essentials of the Eurocode provisions themselves, and for other necessary design information, including tables and figures which can be treated as "deemed to satisfy" the rules of EC 3. In all cases of doubt, or for items not covered, EC 3 and the relevant NAD must be consulted. E-EC 3 is not intended to be used independently of the Eurocode itself.

Concise Eurocode 3

"The Concise EC 3" (C-EC 3)[(20)] is a different type of simplified version published by the SCI. It has a number of features in common with E-EC 3 and the two drafting groups worked in close collaboration. The emphasis is however different, in that C-EC 3 is a shorted version of EC 3, limited in its scope to cover only those types of building structures that can currently be designed using a modern national code. It excludes frames where second-order analysis is necessary and does not cover elastic-plastic analysis for semi-rigid joints.

The C-EC 3 is a self-contained, stand-alone design code. Its purpose is to introduce designers to the provisions of EC 3 by building on familiar ground. Within its own more limited scope it can be used independently of EC 3, yet it will produce designs that also comply fully with the Eurocode itself. However, it is not intended as a complete substitute for EC 3 and direct reference to the Eurocode will be more appropriate in cases where the need for maximum economy warrants the use of the most refined available approach.

DEVELOPMENTS

The planned period for trial use of ENV 1993-1-1 is three years, starting from the date of issue by CEN in May 1992. During the ENV phase of the Structural Eurocodes, they are accompanied in each of the countries concerned by a National Application Document (NAD) for that country, which gives national values for the safety factors γ_F and γ_M for loads and resistances respectively that are given in boxes in the ENV text of EC 3. The NADs also include temporary national recommendations on loads and load combination factors. These will eventually be superseded by Eurocode 1 'Basis of design and actions on structures'.

In addition, the NADs state the current national supporting standards for materials etc to be used pending the completion of the proposed CEN standards referred to in EC 3. In some cases national authorities have also included variations to the ENV provisions which they require as a condition for use during the ENV period.

These aspects of NADs will naturally be amongst the issues to be taken into account by the maintenance group set up to monitor use of EC 3 and give preliminary consideration to any modifications that may be needed for the EN version.

Meanwhile work is in full swing on Part 2 Bridges and Plated Structures. Work has also started on Part 3 Towers, Masts and Chimneys and is expected to start soon on three other parts.

Design aids

It is a clear sign that a new code is being taken seriously when textbooks start to be based upon it. This has already started in 1993 with the publication in Ireland of an EC 3 version of the textbook "Structural Steelwork" by Joannides, Weller and Gwynn[21].

In addition, a remarkable enterprise is about to come to fruition just a week after this conference with the launch of the European Steel Design Education Programme which provides authoritative lecture material for use in Universities across Europe and uses EC 3 as its technical basis.

In France a series of detailed authoritative papers on the various topics covered in EC 3 have been published by CTICM in their journal "Construction Metallique" and comparable efforts have been made in the appropriate technical journals in various other countries.

Explanatory documentation and published worked examples to EC 3 have already come to the attention of the authors from the Czech Republic and Romania as well as the more expected sources such as the ECCS and the SCI. Design software to EC 3 is also widely advertised.

Although, as already stated, the Eurocode format has been planned to avoid the need for an integral or other directly related Commentary, explanations of the provisions from the drafters will inevitably have their own value for educational and other purposes. Reference has already been made to the Background Documents prepared to justify the code provisions as they were drafted. These have been made available to the national standards bodies who are CEN members, but it is far from clear how - or even whether - more widespread publication can be acheived. A series of Eurocode Design Handbooks has therefore been introduced by Thomas Telford to meet such needs and this will of course include one on EC 3[22].

CONCLUDING REMARKS

Papers on code developments are given at many conferences, but one might wonder why one is included in this 50th Anniversary Conference of SSRC. The authors would like to think that the answer is lies in the theme, "SSRC-Link between research and practice", for they truly believe that codes like EC 3 do form the vital final link in that chain.

Within the sections of EC 3 on structural stability are to be found many of the crucial themes of the Structural Stability Research Council, as a witness to its influence on a worldwide scale, albeit in this case with a European rather than an American flavour due to the intervening deliberations of the ECCS in its committee TC 8.

Eurocode 3 has been produced by the combined efforts of a large number of Engineers throughout the EEC and EFTA countries. It has also had a not inconsiderable input from colleagues in central and eastern Europe as well as from the United States, Canada, Japan and elsewhere around the globe. One of the most rewarding aspects of involvement in such work is finding that it not only brings together the research and the thinking of Engineers from different backgrounds, it also brings them together as people both in stimulating technical discussion and in mutual regard and friendship.

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NAME INDEX

KEYWORD INDEX