Stability of Metal Structures—A World View

STRUCTURAL STABILITY RESEARCH COUNCIL
EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK
COLUMN RESEARCH COMMITTEE OF JAPAN
COUNCIL OF MUTUAL ECONOMIC ASSISTANCE

Theoretical and experimental research on the stability of metal structures and the resulting practical design methods have been developed through independent efforts, mainly in Europe, North America, and Japan.

A world literature on this type of research exists and evidently has been exchanged. However, the lack of communication between research bodies in different areas of the world, and the lack of any plan to coordinate the various research methodologies and practical design rules, has led to significant differences in philosophical approach.

Action to rectify this problem through coordination and systematic extension of existing studies was first undertaken 35 years ago in North America by the Column Research Council (CRC), now the Structural Stability Research Council (SSRC), founded in 1944. Progress in this important effort has been reflected in the successive editions (1960, 1966, 1976) of the SSRC (formerly CRC) Guide.[1]

A similar action has been carried out in Western Europe by the European Convention for Constructional Steelwork (ECCS), founded in 1952, whose Committee on Stability is now deliberately structured with a certain parallelism to the SSRC. The resulting approaches to stability problems, largely influenced by European trends in matters of structural safety and design philosophy, are shown in the ECCS Manual (1976).[2]

Coordinating Editors:
Duiliu Sfintesco, CTICM, France
Lynn S. Beedle, Lehigh University, USA
Gerald W. Schulz, University of Innsbruck, Austria
Riccardo Zandonini, Technical University of Milan, Italy
Regional Editors:
Japan: Ben Kato, University of Tokyo
North America: T. V. Galambos, Washington University in St. Louis
West Europe: Duiliu Sfintesco, CTICM
East Europe: Otto Halasz, Technical University of Budapest

In Japan, the Handbook[3] prepared by the Column Research Committee of Japan reflects trends and design approaches in that country and is a substantial counterpart to the North American and Western European references.

Systematic confrontation of the various approaches and design philosophies was initiated by H. Beer, who planned the First International Colloquium on Stability. Due to the premature demise of Mr. Beer, the project was carried out by D. Sfintesco. The Colloquium, devoted to the problem of axially compressed members, was held in Paris in November, 1972.[4]

A major effort to promote international communication on matters of stability was initiated by L. S. Beedle, who formed an SSRC task group on International Cooperation. This group organized the Second International Colloquium on Stability, which encompassed all types of metal structural members and structures. In order to involve a large number of specialists from all parts of the world, this venture was conceived as a "traveling colloquium," with four sessions held in Tokyo, Liege (Belgium), Washington, D.C., and Budapest, between September 1976 and October 1977. The organizers of these conferences were B. Kato (Tokyo), C. Massonnet (Liege), G. Winter (U.S.A.), and O. Halasz (Budapest). To provide liaison and consistency, the ECCS Manual[2] was used as a common Introductory Report at each of the four sessions. The results of the colloquia are summarized in the individual proceedings and commentaries of each session.[5-11]

The purpose of the present report, initiated by L. S. Beedle, G. Winter, and D. Sfintesco, is to summarize a "world view" of the current state of the art and to encourage further coordination and progress in the field of stability of metal structural members and frames. The report consists of three main parts, each reflecting four separate regional views (Japan, North America, West Europe, East Europe):

This comprehensive comparison of the current state of the art in research and design for structural stability, as viewed in four major regions of the world, will be published in serial form in the AISC Engineering Journal. It is expected eventually to be published in book form. Each segment of this report will contain an appendix Reference List and List of Symbols applicable to that segment. In this issue only, the appendix also includes a List of Abbreviations and a Glossary of Terms covering the complete report.

A. *Philosophical Background and Safety Concepts*, describing the background of design methods and rules, and the principles of structural safety in each region.
B. *Design Approaches and Procedures*, discussing the actual solutions for particular design problems, the relation between these solutions and the principles of safety, and the eventual interaction between the design of the structure and the design of its members.
C. *Comparisons and Conclusions*, comparing and contrasting the design approaches of the four regions and suggesting the fields in which international research coordination may be important.

Particular acknowledgement is due the four regional editors who wrote Parts A and C and who arranged for the regional contributors to Part B: B. Kato, Japan; T. V. Galambos, North America; D. Sfintesco, Western Europe; O. Halasz, Eastern Europe.

Thanks are expressed to the three successive Technical Secretaries of SSRC who have given careful attention to collecting the manuscripts and incorporating revisions: Zu-Yan Shen, M. Nuray Aydinoglu, and Sritawat Kitipornchai. Lehigh University undergraduates Randy Haist and Paul Miller were responsible for assembling the list of symbols and the references. The U.S. National Science Foundation sponsored the Washington, D.C. Colloquium and collected a portion of the manuscript material.

As a next step, plans are underway for a Third International Colloquium on Stability, which is expected to include additional regions of the world in which significant research is being conducted. This colloquium will be devoted to the discussion of differences that exist between the various approaches to stability problems and their significance.

PART A. PHILOSOPHICAL BACKGROUND AND SAFETY CONCEPTS

JAPAN

Ben Kato, Regional Editor

PRINCIPLES OF STRUCTURAL SAFETY

Predominant Load Effect—In some areas of the world, the load effect caused by earthquake is more important than the load effects caused by wind turbulence, snow loads, gravity loads, etc. The two principal zones, or belts, of earthquake activity are (a) the Circum-Pacific belt around the Pacific Ocean and (b) the Alpide belt, which extends from the Himalaya mountain range through Iran, Turkey, and the Mediterranean Sea. The Circum-Pacific belt includes the great majority of all earthquakes, both major and minor. Japan, California (U.S.A.), Mexico, the west coast of South America, New Zealand, Indonesia, and the Philippines are all located in this belt.

A Principle of Seismic Design—Earthquake loading is unique among all types of loads because a great earthquake would generally impose on a structure a horizontal acceleration of about 1 g if the structure responds elastically, and would cause greater stresses and deflections in various critical components of the structure than all other loadings combined. However, the probability of such an earthquake occurring within the expected life of that structure is very low. In order to deal effectively with this combination of extreme loading and low probability, a strategy based on dual design criteria usually is adopted:

1. A moderate earthquake which may reasonably be expected to occur at the building site during the life of the structure is taken as the basis of design. The structure should be proportioned to resist this intensity of ground motion without significant damage. That is, a serviceability limit state design should be adopted against this level of earthquake in order to keep the structural skeleton elastic and to avoid excessive deflection which may lead to damage to exterior and interior walls, ceilings, and other nonstructural elements.
2. The most severe earthquake which could possibly occur at the site is applied as a test of the structural safety. Because this earthquake is very unlikely to occur within the life of the structure, the designer is economically justified in permitting it to cause significant structural damage; however, collapse and loss of life must be avoided. Thus, an ultimate limit state should be the basis of design for this extreme earthquake.

DESIGN OF STRUCTURAL MEMBERS VULNERABLE TO INSTABILITY

Members Governed by Earthquake Loading—If a structure is allowed to undergo substantial plastic deformation against the severe earthquake, as mentioned above, the input lateral force due to dynamic inertial force will be limited by the yield level of the structure, and the hysteretic plastic work done by the structure will absorb the input earthquake energy and damp the vibration. The structure may survive the severe earthquake by absorbing all the input energy. In this context, the assessment of the deformation capacity of steel members and structures is essential in order to evaluate the energy-absorbing capacity for seismic design.

It is generally understood that steel structures are much more advantageous than concrete or masonry structures from the asiesmic point of view, because steel is a more ductile material. But this excellent ductility of steel members may be considerably impaired by the load-carrying capacity of buckling. The slenderness ratio of members and width-to-thickness ratio of plate elements of the sections should be limited within certain values. Therefore, in the earthquake-resistant design of steel structures, rather stocky members with compact sections are usually used to insure sufficient plastic deformation capacity. Since energy-absorbing capacity can be expressed as the product of strength and plastic deformation, the evaluation of these parameters is essential in the evaluation of earthquake-resisting capacity of members subject to buckling.

Most research efforts are oriented in this direction in Japan. Although not many papers in this area were contributed to the four traveling colloquia, there is a global tendency to increase research on stability emphasizing this aspect. Kato[12] discussed the relationship between the D/t ratio and the rotation capacity (deformation capacity) of circular steel tubes subject to local buckling, taking into account the post-buckling behavior. Akiyama[13,14] discussed the lateral buckling strength and deformability of girders with realistic restraints, such as floor slabs and purlins, under simulated earthquake loading. Matsui's contribution[15] was an evaluation of the effect of lateral bracing on the strength and ductility of columns. Wakabayashi[16] demonstrated that by encasing the steel in concrete, the deformability of beam-columns could be much improved.

Since these studies deal with short members with compact sections, the effect of residual stresses are not significant, but the effect of the characteristics of the stress-strain relationship (bilinear or elastic-plastic strain hardening associated with yield of material) is more influential. The amount of plastic deformation associated with instability phenomena is quite variable and inevitably there is a wide band of scatter. Much more experimental data are needed to make meaningful statistical evaluations.

Of course, it is not impossible to design structures so that they will resist the most severe earthquake elastically, using thin-walled members with large elastic moment capacity. However, their use is limited to low-rise and lightweight structures, because the stiffer the structure the larger the acceleration of earthquake vibration the structure will receive.

Members Governed by Other Loadings—Girders in highway and railway bridges are usually designed as simply supported members. Floor beams of buildings are also designed as simply supported at both ends, where gravity load is of major concern in design. For this type of loading, the general principle of structural safety is common throughout the world. The maximum strength of these members is determined by theory and/or experiments taking into account the possible imperfections involved, such as initial crookedness and residual stresses. The characteristic values for the strength of these members are then estimated through statistical analysis of the available data.

Studies on plate buckling strength by Fujita[17] and by Nishino,[18] and studies on lateral buckling strength of beams and girders by Fukumoto[19,20] were carried out along this direction. Fukumoto's study on the buckling strength of centrally loaded columns[21] also provides valuable statistical information for determination of the design column curve.

OTHER TOPICS

Shell Buckling—With respect to the buckling strength of conical shells, much valuable information was contributed from the field of aeronautical and space sciences.[22] This information may be useful for the design of civil engineering structures, such as water storage towers.[23] A point which should be kept in mind is that the test specimens used in the field of aeronautical and space sciences are fabricated to closer tolerances than those used in the field of civil engineering. If a design formula is to be considered on the basis of this information, the differences in accuracy must be duly reflected in the evaluation of factor of safety.

Column Curves for Cold-Formed Tubular Section Members—According to the ECCS Recommendations,[24] cold-formed tubes must not be designed with curve *a* of the European Column Curves (Fig. 1), but with the lower curve *c*. Several contributions were presented in the four

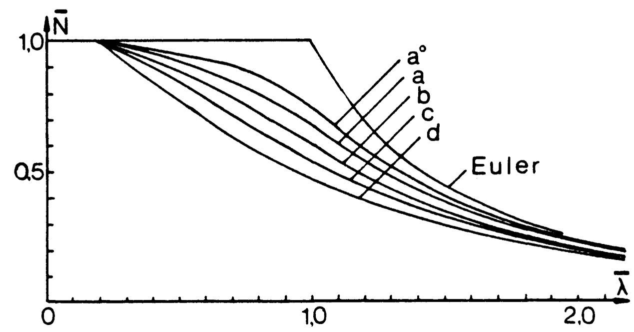

Fig. 1. European column curves for steel

"traveling" colloquia which may help the assessment of these Recommendations:

1. Ballio, Finzi, and Urbano[25] carried out an experimental investigation on cold-formed square tubes of high strength steel. Six groups of 12 specimens each were tested and characteristic values, defined as mean value minus two times the standard deviation, were determined. These characteristic values are well represented by curve *c*. The authors also tested columns with special profiles, such as rectangular tubes built-up by welding from cold-formed channels, and obtained rather controversial results.

2. Kato[26] investigated the buckling strength of cold-formed circular tubes and showed the average strength for each slenderness ratio was above ECCS curve *a*, which was established for seamless rolled tubes and welded, hot-finished tubes. The theoretical predictions based on the tangent modulus concept were compared with test results and showed satisfactory agreement. Tangent modulus was evaluated from the stress-strain relationships, which represent the mean values of the data obtained from stub column tests.

3. Schulz[27] pointed out, in his general report, that the difference between ECCS's and Kato's approach is that the former is based on ultimate strength analysis and the latter on the tangent modulus concept. He noted that there is a difference in the evaluation of the experimental data: test results in the former represent the mean values minus two times the standard deviation, while test results in the latter represent the mean value only. Thus, a direct comparison is impossible.

4. Birkemoe[28] carried out a buckling test of rectangular and square tubes. A unique heat-treatment process was applied which reduces the magnitude of residual stresses due to cold-forming, but retains the advantages of elevated yield strength resulting from the cold-forming. Comparisons of his column tests with the SSRC recommendations[1] indicate that the mean

column strength met or exceeded the predicted mean strength using the equation for Group 1 columns and the actual tensile coupon yield stress.

Generally speaking, seamless or hot-finished tubes are in more favorable condition than cold-formed tubes with respect to residual stresses, but the former are inferior to the latter in the accuracy of sectional shape and straightness of the member.

But a more important point may be the definition of the yield strength which is used in the evaluation of the non-dimensional slenderness ratio $\bar{\lambda}$. The design yield strength of cold-formed tubes is specified on the basis of the yield strength of steel sheets before forming in Japan, Hungary, and perhaps in other countries where cold-formed sections are extensively used. The yield strength of cold-formed sections is higher than those of virgin sheets due to strain-hardening. In cold-formed circular tubes, the relation between the compressive yield strength of circular tube, σ_y, and the tensile yield strength of the corresponding virgin sheet σ_0 is given

$$\sigma_y = \left[1.38 - 0.009\left(\frac{D}{t}\right)\right]\sigma_0$$

where σ_y is obtained from the stub-column test and σ_0 is obtained from the tensile coupon test. In cold-formed rectangular tubes, Szilassy[29] reports that the increase of yield stress is about 50% at the most severely strained corners. This increase of yield strength obviously increases the column strength, but if the non-dimensional slenderness ratio $\bar{\lambda}$ is evaluated on the basis of the increased yield strength, the advantage must be cancelled. On the contrary, the use of the non-dimensional slenderness $\bar{\lambda}$ on the basis of the virgin sheet's yield value does mean the shift of column curve to the righthand side along the horizontal axis, and the advantage of the increased yield strength can be reasonably reflected, thus meeting the higher class of column curve. There are no reasons to adopt the actual tensile coupon yield stress taken from the finished tube for the evaluation of the non-dimensional slenderness. The tensile coupon yield stress and the average compressive yield stress obtained from stub-column tests are basically different, both in magnitude and in physical meaning. Furthermore, the tensile coupon yield stress of the cold-formed square and rectangular sections varies considerably from the corner to the flat part, and one cannot decide what is the representative yield stress. On the other hand, the virgin sheet's tensile properties are always checked at the mill as a part of quality control and they form the basis of the minimum specified values of a given grade of steel. Because of these divergent views and approaches, it appears that further investigations, on a common basis, of cold-formed tubular columns are needed.

NORTH AMERICA

T. V. Galambos, Regional Editor

The legal criteria which regulate structural design on the North American continent are very diverse and they differ in their origin, depending on whether they apply to building design, highway bridge design, railway bridge design, industrial structures design, etc. For highway bridges, for example, the legal status is promulgated by the individual states and provinces through the Department of Transportation, while for buildings the individual governmental units, such as cities and counties, decide on the particular set of design criteria appropriate for them. Differences also exist between the practices and basic philosophies in the U.S.A., Canada, and Mexico. For example, for metal structures the basic concept in the U.S.A. is still Allowable Stress Design, while in Canada valid regulations are in force which are based on Limit States Design criteria. Due to this great diversity, it is not surprising that differences in design philosophy exist across the continent for the various types of structures which are subject to instability investigations.

A degree of unity is provided, however, in that the "end-users" of design criteria, namely, the groups to which the designer is legally responsible, either adopt directly, or modify slightly to conform to special local needs, the codes and specifications which are developed and maintained by a group of model standard-writing organizations. In the U.S.A. such groups as the American Institute of Steel Construction, the Aluminum Association, the Steel Joist Institute, the American Iron and Steel Institute, the American Association of State Highway and Transportation Officials, etc. each have generated and continually maintain specifications appropriate for the design of structures falling under their sphere of interest. In Canada the various committees within the National Research Council are providing model specifications which are based on fairly uniform criteria for design.

In the area of design against structural instability, the majority of the stability design criteria originate with, or are influenced by, the Structural Stability Research Council (SSRC, formerly Column Research Council) through the three successive editions of the *Guide to Stability Design Criteria for Metal Structures.*[1] It is for this reason (to remove confusion and lack of harmony in design formulas) that the Council was formed in 1944. Its efforts have met with success.

Design against instability in metal structures, as practiced by the great diversity of end-users throughout the

North American continent, is thus essentially based on the criteria and philosophies developed by the SSRC. Within the SSRC, the design philosophy has gradually evolved and changes are evident with time. This evolution is best exemplified by the developments in the philosophy of the design of axially loaded columns.

When the SSRC (then CRC) was founded in 1944, the situation regarding the design of steel columns in the U.S.A. was reminiscent of the days when construction of the tower of Babel was indefinitely suspended. Due to the early efforts of the CRC, a unified philosophy was enunciated (in Technical Memorandum No. 1, "The Basic Column Formula," May 19, 1952) by affirming: "It is the considered opinion of the Column Research Council that the tangent modulus formula for the buckling strength affords a proper basis for the establishment of working-load formulas." As a result of this firm statement, research provided the tests, the analyses, and finally, the design formulas which were adopted and are now used in all of the model design specifications.

Thus, the basic idea which is reflected in all of the current (1979) design provisions, as detailed in Part B of this report, is the tangent modulus concept. That is, a geometrically perfect metal structural element which, however, can be characterized by material non-linearities (e.g., residual stress, non-linear stress-strain curve, etc.) will reach its structural limit at the tangent modulus load. This idea was extended from columns to beams, beam-columns, and frames. It was, of course, realized that the tangent modulus concept is an idealization which often cannot be, and sometimes need not be, realized, but these effects were taken care of by suitable factors of safety. The virtue of the adoption of the tangent modulus concept by the CRC was that it brought unity out of chaos, and it has served very well over the past quarter-century.

Research has since demonstrated that geometric effects of various kinds (initial imperfections, post-buckling strength, etc.) often mask or obliterate the tangent modulus strength, and the third edition of the SSRC Guide[1] reflects the results of this thinking. For example, in the third chapter of the Guide curves are given for centrally loaded steel columns which were developed on the basis of initial imperfections. Furthermore, statistical methods were used in deriving the final curves. This philosophy is going to be reflected in future North American design specifications, and has, in fact, resulted already in some adopted or proposed changes (see Part B of this report). Thus, the emerging philosophy is to determine the strength of an element or a structure in the most rational way, taking into account all of the significant effects, be they residual stress, initial imperfection, non-linear stress-strain curve, post-buckling strength, or whatever.

In conclusion, then it can be stated that the basic design philosophies for instability design in North America emanate in one form or another from the SSRC, and that current (1979) state-of-the-art design is based principally on the tangent modulus concept. It can also be stated that this philosophy is changing toward more sophisticated concepts which encompass the best means of determining strength, including any significant effect, and consider statistical and probabilistic ideas. Design specifications at present (1979) are in transit toward this changeover.

This discussion on design philosophy would be incomplete if mention would not be made of one serious problem which is under current discussion, namely, the present lack of a fully interpreted, unified, and commonly accepted philosophy of frame design. The third edition of the SSRC Guide is equivocal on this subject, giving a number of methods without fully endorsing any one of them. This, of course, reflects the ambivalence within the profession, and the resolution of this philosophical problem is one of the greatest needs in the area of metal structures instability in the North American design community. Specifically, it will be necessary to define whether or not the overall stability of the frame must be assured up to the formation of the first plastic hinge or the formation of a plastic mechanism.

Finally, it is appropriate to say that present North American design rules on stability, while they tend to follow the basic philosophies defined by the SSRC with regard to the appropriate limit states, are not completely in agreement about the safety concepts. The majority of the design specifications use the Allowable Stress Design concept, using factors of safety on limiting stresses derived from nominally elastic analyses. However, many design formulas are based on models which incorporate indirectly or directly the inelastic behavior and the plastic or post-buckling reserve capacities. This makes it difficult to clearly see the basis for many design formulas unless one is familiar with their particular background, especially since sometimes the behavioral model and sometimes the factor of safety is manipulated to achieve the desired result. Fortunately, there exist for most of the major design specifications alternate sets of design criteria, adopted or proposed, which are based on explicit and appropriate design models and on Load and Resistance Factor (Limit States) Design. At first, these latter criteria were based on calibrated load and resistance factors, but later versions include the concepts of first-order probability theory.

The framework for progress exists, resting on the philosophies of using the ultimate strength model for behavior, and the probability model for the safety concepts. It is to be hoped that the diverse design criteria in North America will, with time, assimilate these philosophies for every day use.

WEST EUROPE

Duiliu Sfintesco, Regional Editor

EARLY DEVELOPMENTS

Western Europe is the part of the world where stability studies were born.

A rational approach to stability and to other aspects of structural mechanics seems to have been contemplated by Galileo in his treatise printed in 1638, but the first precise formulation was given by Euler in a study published in Lausanne, Switzerland, in 1744. In this study, Euler demonstrated that a straight, centrally loaded compression member in indefinitely elastic material could not remain straight when the load reached a critical value for which he gave the mathematical expression.

Until the end of the 19th century, it was accepted that iron structural members in compression could fail in two ways: compact members by squashing of the metal, and slender members by elastic instability.

As a consequence, it was required in the design of such members that the average compression stress in the cross section should not exceed:

1. For compact members, an allowable stress defined as the quotient of the ultimate stress in tension of the material, divided by a so-called safety factor, or
2. For slender members, the critical Euler-stress divided by the same factor.

This arrangement seemed to be satisfactory for a while. However, in spite of the high value adopted for the safety factor (of the order of 6!), there were some incidents with members of medium slenderness ratios (80 to 100), and, about 1900, there occurred a spectacular accident during the construction of a gas holder in Hamburg, Germany, which gave rise to further research in later years.

The first systematic experimental investigation on this matter was made in Switzerland by Tetmajer, who carried out a large series of tests on specimens of various shapes in mild steel. On the basis of his results, he proposed in 1903 to keep the critical Euler stress as the criterion for slenderness ratios above 105, and to adopt below that level a linear function of the slenderness ratio, starting with the allowable stress for slenderness zero.

Several investigators then tried to find empirical formulas, valid for the whole range of slenderness ratios occurring in practical design. The most successful one was Rankine, who proposed a formula of the type

$$\sigma \leq \frac{\sigma_{adm}}{1 + c\lambda^2}$$

where σ_{adm} was the allowable stress, and c was a factor depending on the material. This type of formula was in use until recently.

About 1919, Johnson proposed, for small and medium slenderness ratios, a parabolic transition formula of the form

$$a - b\lambda^2$$

where the constant values were chosen so that the parabola was tangent to the Euler hyperbola. This type of formula is still in use in certain specifications.

On the basis of such research findings and proposed formulas, design rules were developed in Western European countries more or less independently from each other.

For example, the French specifications went through the following stages:

- 1891: The designer was authorized to use the method of analysis of his choice.
- 1915: The use of the Rankine formula became compulsory.
- 1927: The compression stress for compact members was limited (as in tension) to one-third of the ultimate stress and to one-half of the yield stress.

POST-WAR DEVELOPMENTS

During the last three decades, a profusion of research on various aspects of structural stability, both theoretical and experimental, has been undertaken in several Western European countries. At the same time, the fundamental treatment of structural safety began to be reconsidered.

These developments were reflected in quite different ways in the design specifications of the single countries, depending on the structure of the specification-writing bodies, on their openness to national or foreign research findings and, last but not least, on the influence of certain individual scholars.

The result, as seen for the Western European area as a whole, was a considerable diversity of national specifications, and variations within the same country for various fields of application (e.g., buildings, highway bridges, railway bridges, government-owned projects, privately owned projects, etc.).

It is worthy of note that for a quarter of a century the most prestigious European specification for stability of steel structures has been the German DIN 4114, presented in 1952 by Klöppel. It was the most complete document of that type ever produced until that date. It dealt with practically all stability cases to be considered in steel structural design, thus giving a precise response to questions likely to be put by the designer to himself.

The stability of centrally loaded compression members was treated under the assumption of a certain parasitic bending effect, due to possible uncontrolled deviation of the load from the rigorously centered position.

This specification has been used as a guide or reference in several other countries.

A number of other documents making explicit use of formulas proposed by various researchers appeared in other countries. These formulas became more or less sacrosanct, intangible references in the official position (and the national ambition) of the country concerned.

A remarkable development occurred in the 1950's in France, with a new approach proposed by Dutheil. He put forward a concept taking into account the sum of all types of so-called "inevitable imperfections" of actual structural members (thus including geometric imperfections, inhomogeneity of material and residual stresses). He referred to the behavior of members "of industrial type" directly, rather than to that of an idealized "perfect" model, subsequently to be adjusted by means of certain corrections. Thus, the principle of divergence of equilibrium was used instead of the universally accepted concept of bifurcation.

This now recognized basic position gave rise at the time to considerable criticism, but the resulting controversy led to a sound, realistic evolution in the treatment of stability problems.

NEW CONCEPTS IN MATTER OF STRUCTURAL STABILITY

The classical concept of "safety factor" as a basis of the universally admitted and practiced allowable stress design procedure, was first denounced by Streletsky, as early as 1928, as having "no real meaning."

But the formal challenge against that concept was issued at the IABSE Congress of 1948 in Liege, by a group of French participants who advocated a probabilistic approach. This thesis was then supported and considerably strengthened by extensive fundamental studies, especially by American scientists (Freudenthal).

However this new approach, for which a more realistic significance was claimed and which, as a consequence, led to limit state design instead of classical allowable stress design, met very different degrees of success in the various countries.

The first countries to introduce this new philosophy into their design specifications were several Eastern European countries and—in Western Europe—France.

In this respect, the evolution of the French specifications for steel structures is quite remarkable:

- 1946: Allowable stress design, with σ limited to $3/5$ of the yield stress.
- 1956: σ limited to $2/3$ of the yield stress under all types of load except wind; $3/4$ when wind was also considered.
- 1966: Semi-probabilistic design with load and resistance factors by referring to limit states.

This last approach, originally introduced tentatively as an alternative to the preceding one, was adopted almost immediately by designers and since 1966 has been the only one in use.

Other countries have long hesitated before accepting this concept, but this process is currently in rapid progress under the influence of major international bodies.

INTERNATIONAL DEVELOPMENTS IN WESTERN EUROPE

The major source of innovation and progress in Western Europe in matters of structural steel design in the last quarter of a century has been the international activity of the European Convention for Constructional Steelwork (ECCS).

The initial purpose of this activity in the field of stability was to eliminate the enormous discrepancy that existed between national specifications, particularly between the column curves of individual countries, and to attain a rational common solution on an international level.

The highly divergent theoretical positions (and national or personal ambitions) having revealed themselves as unassailable, the basic case of centrally compressed members was first treated experimentally.

To this end, a series of over 1000 column tests carried out on samples of "industrial type" members taken at random in several countries in order to be representative of the region, was performed in seven countries. Common testing procedures were employed to reflect as closely as possible the conditions existing in actual structures.[2] Statistical interpretation of the results was used as the basis for an experimental column curve, with a constant safety factor applied to mean values minus two standard deviations, thus ensuring the same degree of safety for all slenderness ratios.

In this way the new concepts in matters of stability and structural safety were linked together, resulting in a treatment of the problem which finally was, if not unanimously, at least officially admitted and adopted.

A complete theoretical investigation of all types of shapes, consistent with the test conditions and the interpretation of their results, was then carried out on computer in Graz, Austria (Beer, Schulz). The concept of multiple column curves was adopted, in order to allow specific consideration of the differing behavior of compression members depending on their cross-sectional shape and size and their fabrication process. The results are the "European Column Curves" of the ECCS (see Fig. 1).

It was hereby understood that the extent to which these multiple (5) column curves would be adopted for practical design (i.e., as given or in reduced number) would be left to the judgment of the code-makers, for consideration of both technical and economic criteria.

The other stability cases have been treated by international task groups of the ECCS, similar to those of the SSRC. The results of their studies are given in the *European Recommendations for Steel Structures* of the ECCS,[24] and described more in detail in the ECCS *Manual on the Stability of Steel Structures.*[2]

Systematic contacts and cooperation have been established with the SSRC and to some extent with parallel groups in Japan and Eastern Europe. As a result, the Second International Colloquium on Stability was organized and held in 1976–77 with sessions in Tokyo, Liege, Washington, and Budapest ("Traveling Colloquium"), with the ECCS Manual as the Introductory Report. This event was a major step toward worldwide cooperation on stability studies.

At the present stage, the specifications of all Western European countries almost completely reflect the Recommendations of the ECCS, thus achieving a remarkable international consistency. Another most important development is the preparation of a building code (the so-called Euro-Code) for the countries of the Common Market, largely based on the ECCS Recommendations.

EAST EUROPE

Otto Halasz, Regional Editor

HISTORICAL REMARKS

The Safety Concept, as adopted by most East European countries and by recommendations (RS-242/72) and specifications (SEV 384/76) issued by the Council of Mutual Economic Aid (CMEA), was originally developed and formulated mostly by Soviet experts in the 30's and 40's of this century. It was officially adopted first in the Soviet Building Code in 1946 and afterwards in all Soviet specifications. Similar concepts were adopted in Hungary and Poland around 1950. These initiatives had great influence on the work of other international organizations (ISO, CEB) in their standardizing activity.

A governing idea of the CMEA specifications was to codify the main features of design philosophy common for structures of different materials (reinforced and prestressed concrete, steel, aluminum, brick, timber), thus ensuring a uniform attitude in formulating their design rules.

The background, development and details of this process are summarized in several monographs.[30-36]

The Safety Concept—identified by various names such as "limit states approach," "method of divided safety factors," "semi-probabilistic method," and lately as "level I method in a probabilistic approach," indicating the different stages of its development—was originally offered as an alternative to the previously generally adopted "allowable stress approach" and was motivated by its criticism. Based on contemporary comments,[30] this development can be summarized as follows.

LIMIT STATES

With broadening knowledge of the performance of steel structures in the inelastic range, and reinforced concrete elements in their different working stages, it seemed necessary to interpret the original and newly accumulated ideas hidden behind the design formulas of the allowable stress approach:

$$\sigma(F_K) \le \sigma_w\ ; \quad e(F_K) \le e_w \tag{1}$$

$$\sigma_w = \sigma_y/n_1 \tag{2a}$$

$$\sigma_w = \sigma_{cr}/n_2 \tag{2b}$$

$$\sigma_w = \sigma_f/n_3 \tag{2c}$$

expressing comparison of stresses (and deflections) due to code-specified loads F_K to their allowable values σ_w (and e_w), the former derived from yield stress σ_y , critical stress σ_{cr} , and fatigue strength σ_f by safety factors n_1, n_2 and n_3, respectively. First of all Eq. (2b) proved to be problematic, in some cases representing the requirement of elastic behavior under normative loads (giving indirect and often irrelevant information about margin of safety[37] and bearing little relevance to actual behavior of reinforced concrete elements)[31] and in other cases (bearing in mind the design practice of trusses, connections, etc.) including, at least quantitatively, the consequences of ductile behavior and referring directly to failure. To avoid ambiguities and unreasonable differences in actual safety, the following suggestions were made:

1. Systematically taking into account all known phenomena rendering the structure unsuitable to anticipated design requirements, thus bringing it to a "limit state."
2. Selecting parameters Φ^i (loads, load-effects, stresses, deflections, etc.) suitable for quantitative description of the limit states by their values Φ^i_L depending on geometrical, strength, and stiffness characteristics of the structure.
3. Adopting adequate computational methods to monitor the changing values of Φ^i_F of the parameters in the loading process.
4. Judging safety by comparison of the most unfavorable value of Φ^i_F during erection and lifetime to Φ^i_L, con-

sidering 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 such 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, etc.) making further functioning impossible.

B. Serviceability limit states (restricting regular use of the structure), caused by phenomena such 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 parameters Φ^i_F and Φ^i_L, respectively.

DIVIDED SAFETY FACTORS

Progress in understanding structural behavior and in reliability of material was manifested by a successive increase of allowable stresses (e.g., increasing σ_w for mild steel from 140 MPa to 160 Mpa during the Second World War in the Soviet Union). It was commonly believed that further general increase would jeopardize safety and further progress could be achieved by differentiating among structures and loading cases only. As the main viewpoint of differentiation (within a certain class of structures and materials), the differences in variability of loads were obvious, reflecting the general experience that structures with a high proportion of (unchanging) dead load had better performance records of long life than light structures with a high proportion of strongly varying live or climatic loads. This motivated the early Soviet specifications for reinforced concrete (OST 90003/38) to apply different safety factors[38] depending on the ratio of dead load to live loads, and subsequently splitting up the general safety factors in Eqs. (2) into two parts:

$$n = \gamma_F \cdot \gamma_m \tag{3}$$

where γ_F expressed uncertainties due to loads, and γ_m those due to resistance.

SEMI-PROBABILISTIC APPROACH

In this relation it was necessary to clear up the actual meaning of traditional terms such as "characteristic" (normative, code-specified, nominal, guaranteed) values of load (F_K) and resistance (R_K). This brought about the concept that they can be interpreted by regarding load and

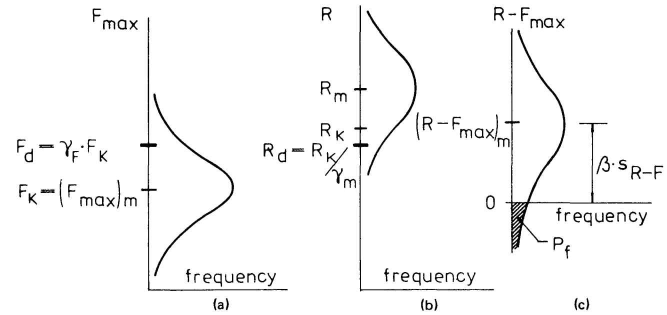

Figure 2

resistance as random variables,[39-43] characterized 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. 2).

Following the interpretation of Rzanicin,[44-46] the risk of failure (P_f) can be expressed by the probability

$$P(R - S < 0) = P_f \tag{4}$$

(shadowed area in Fig. 2), which, in case of Gaussian distribution, depends on the safety index β:

$$\beta = \frac{(R - F_{max})_m}{s_{R-F}} = \frac{R_m - (F_{max})_m}{\sqrt{[R_m \cdot \delta_R]^2 + [(F_{max})_m \cdot \delta_F]^2}} \tag{5}$$

allowing one to establish correlation between risk of failure and the "central" safety factor n_C:

$$n_C = \frac{R_m}{(F_{max})_m} = \frac{1 + \sqrt{\beta^2(\delta_R^2 + \delta_F^2) + \beta^4 \delta_R^2 \delta_F^2}}{1 - \beta^2 \delta_R^2} \tag{6}$$

This can be (somewhat arbitrarily and approximately) split up:

$$n_C - n_F \cdot n_R \; ; \quad n_F = 1 + \bar{\beta}\delta_F \; ; \quad n_R = \frac{1}{1 - \bar{\beta}\delta_R} \tag{7}$$

and thus, in case of arbitrarily chosen characteristic values, by:

$$\gamma_F = n_F \frac{(F_{max})_m}{F_K} \; ; \quad \gamma_m = n_R \frac{R_K}{R_m} \tag{8}$$

The basic design formula can be established:

$$\gamma_F F_K \leq \frac{R_K}{\gamma_m} = F_d \; ; \quad \frac{R_K}{\gamma_m} = R_d \; ; \quad F_d \leq R_d \tag{9}$$

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

$$\Phi^i_F(\gamma_{Fj} F_{Kj}) \leq m \Phi^i_L(\sigma_y/A, \gamma_m) \tag{11}$$

where m represents a correction factor for special circumstances in fabrication, tolerances, and structural behavior, and A represents geometrical quantities.

RESISTANCE FACTORS

Originally, $\overline{\beta} = 3$ was suggested.[30] For characteristic values 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,

$$\gamma_m = \frac{(\sigma_y)_m - 2s_\sigma}{(\sigma_y)_m - 3s_\sigma} \approx 1.1\text{-}1.2 \tag{13}$$

For ultimate tensile stress, $\gamma_m \approx 1.45\text{-}1.6$.

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 a certain characteristic time interval (for instance, one year for snow load). Having very few data about σ_F,[47] load factors were mainly estimated based on 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 of γ_F values for limit states A:

Dead load: $\gamma_F = 1.1$; exceptionally 1.2.

Live load, if intensity $q\ N/m^2$:

$$q \leq 2000: \quad \gamma_F = 1.4$$
$$2000 < q \leq 5000: \quad \gamma_F = 1.3$$
$$q > 5000: \quad \gamma_F = 1.2$$

Snow load: $\gamma_F = 1.4$

Wind load: $\gamma_F = 1.2$

For limit states B: $\gamma_F = 1$

LOAD COMBINATIONS

To express the reduced probability of coincidence of maxima of more components in a loading process, loads are categorized as:

1. Permanent loads (self-weight, earth pressure)
2. Variable loads and actions (live load, climatic loads, imposed deformations), subdivided wholly are partially into:
   a. Sustained (long-term) loads or load-parts
   b. Instantaneous (short-term) loads or load-parts
3. Catastrophic loads (seismic loads, etc.)

Two load combinations are suggested:

1. 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-time loads) are to be applied.
2. Irregular combination, including one catastrophic load. Combination factor $\gamma_c = 0.8$ for all short-term loads can be applied.

TRENDS IN RESEARCH

The above specifications allow, without giving detailed rules, use of probabilistic design procedures as well. This means a return to the basic idea outlined in connection with Eqs. (4), (5), and (6)—to check safety directly by the risk of failure P_f, safety index β, using limiting values for central safety factor n_c,[48] or prescribing limiting values Φ_L^i depending on coefficients of variation both of resistance and loads. Actual application needs further research. For example:

- To find theoretically well-founded probability density functions for different steel grades[49] and for loads.
- To find appropriate probabilistic procedures for non-linear (instability) problems[50,51] 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 its separated parts (which can differ substantially in both the safe and unsafe sense).[35]
- To find a compromise between requirements of safety and economy in the choice of risk of failure or safety index.[52–54]

RELIABILITY CONSIDERATIONS

The limit states approach, in its traditional form, is concerned mainly with failure due to exceptionally high loads. For this reason it suffices to "condense" the loading history into a probability density function (Fig. 3a) containing no information about the actual number of load-cycles or time-effect (except that longer lifetime involves more unfavorable F_{max} distribution).

Some limit states, such as those caused by fatigue, accumulation of damage, and residual deformations, are closely connected to actual loading history. Therefore, the term "safety," rather than "reliability," should be used, interpreted as probability of survival of the structure $P(t)$, depending on required time of operation (Fig. 3b).[35]

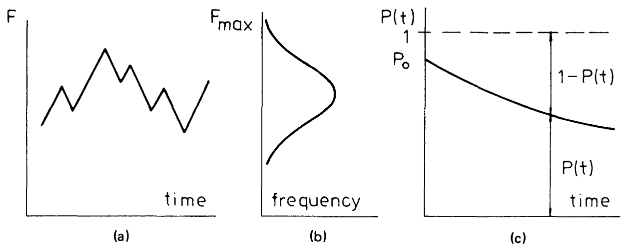

Figure 3

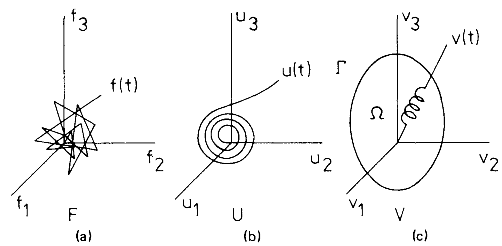

Figure 4 (Ref. 51)

The way of thinking can be characterized, following Bolotin's[51] interpretation, by Fig. 4: from the (stochastic) loading history, described by $f(t)$, to the structural response $u(t)$, and further to quality parameters $v(t)$; 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 for 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.

Economic considerations, such as the choice of safety index β, can be based on reliability as well, containing time-dependent elements (such as amortization). In this sense optimization using cost-function of the form[35]

$$C = C_0 + C_1(P)(1 - P) \qquad (16)$$

provide a solution, establishing an important correlation between operational life and required safety.

FINAL REMARKS

It seems to be a common attitude of modern specifications to set growing requirements for the designer: to look more closely at the real complexity of the structure starting with the method of fabrication and unavoidable tolerances and defects, 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 board as one of a multitude of similar structures differing slightly in form, strength, loading,—not identified at the design stage. Thus, the definition of safety involves the use of statistics and the laws of the theory of probability and reliability.

Additionally, analysis of structural response is becoming more sophisticated. Specifications must incorporate an increasing number of design rules, sometimes in the form of tables and diagrams (often derived directly from experiments), making them voluminous and increasing the number of appendices.

Although we possess all the facilities required 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 safe and economical everyday design practice and the best way for feeding back accumulating information gained by research.

PART B. APPROACHES AND DESIGN PROCEDURES

CHAPTER		CONTRIBUTORS			
No.	Title	Japan	North America	West Europe	East Europe
1	Compression Members	M. Wakabayashi	R. Zandonini, L. Tall	G. W. Schulz	O. Halasz, M. Ivanyi
2	Built-up Members	T. Suzuki	B. G. Johnston	L. Finzi	O. Halasz, M. Ivanyi
3	Beams	Y. Fukumoto	T. V. Galambos	J. Lindner	O. Halasz, M. Ivanyi
4	Plate and Box Girders	F. Nishino	A. Ostapenko	C. Massonnet	O. Halasz, M. Ivanyi
5	Beams-Columns	H. Akiyama	W. F. Chen	J. Strating	O. Halasz, M. Ivanyi
6	Frames	S. Morino	L. W. Lu	U. Vogel, C. Massonnet	O. Halasz, M. Ivanyi
7	Triangulated Structures	T. Nakamura	T. V. Galambos	P. Dubas, C. Urbano	O. Halasz, M. Ivanyi
8	Shells	S. Kobayashi	A. Chajes	D. Vandepitte	O. Halasz, M. Ivanyi
9	Composite Members	M. Wakabayashi	G. Winter	P. J. Dowling	—
10	Cold-formed steel	—	S. J. Errera	J. W. Stark	O. Halasz, M. Ivanyi

Chapter 1. Compression Members

a. REGIONAL RECOMMENDATIONS

JAPAN

(There are no regional recommendations in Japan.)

NORTH AMERICA

(Key Document: SSRC Guide[1])

FLEXURAL BUCKLING

Steel—The flexural buckling of a centrally compressed member is treated extensively in the Guide (Chap. 3). The effect of geometrical imperfections, such as out-of-straightness and end eccentricities of the applied loads, as well as the residual stress influence, are discussed for the principal types of sections studied so far.

With reference to design considerations, the Basic Column Strength Formula, set out by the Column Research Council in 1960 and still adopted in many North American codes, is discussed.

Finally, there is discussion of the results of recent research studies, carried out with the purpose of reducing the uncertainties affecting the column strength evaluation in order to achieve a more economic design consistent with needed safety.

On the basis of the results of these studies, a multiple-column-curves approach based on the maximum load is recommended. This approach uses the three curves shown in non-dimensional form in Fig. NA1.1, for which a set of equations is presented (Guide, p. 68).

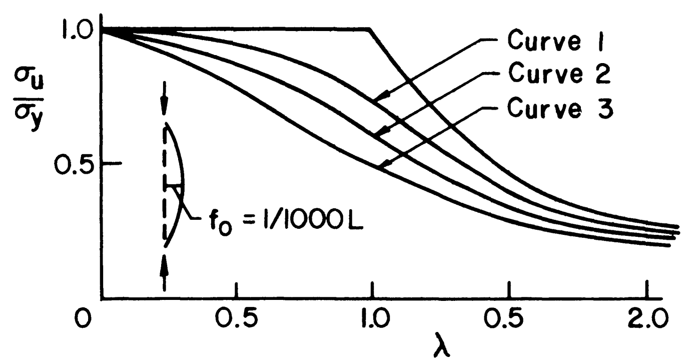

Figure NA1.1

Commentary—The multiple curves-maximum strength approach assigns column strength according to various categories of column manufacture, fabrication, and geometry.

This approach is based on a theoretical study of column curves using the actual measured residual stress distributions and an assumed initial out-of-straightness at mid-height of $\delta_0 = 0.001L$.

A total of 112 column curves has been computed. The column shapes are further grouped in three categories, each of which is represented by a single "average" curve.

Aluminum—The evaluation of maximum strength of both straight and initially crooked aluminum columns is treated, with reference to the more significant parameters affecting column strength. As to residual stresses, in the discussion of the theoretical analysis their role is pointed out to be of relatively lower importance than for steel members. This is because of the reduced residual stress levels from straightening operations applied to extruded and heat-treated alloys in the current mill practice.

No specific recommendations are given for design purposes. However, the use of a tangent modulus approach, as done in the North American codes, is partially justified on the basis of the work by Batterman and Johnston.[55] The results of this study pointed out that a small amount of end-restraint can offset the effect of initial crookedness on the column strength.

EFFECTIVE LENGTH

Effective length factors K are given for the usual end conditions. An extensive treatment of the effective length of columns in continuous and multistory frames is also provided for braced and unbraced frames.

Alignment charts for symmetrical frames, symmetrically loaded, are provided, based on an elastic bifurcation theory.

BUCKLING OF TUBULAR COLUMNS

The present status of knowledge, research, and specification practice is reviewed. The buckling characteristics of axially compressed tubes are discussed in terms of four types of behavior: (1) elastic column buckling; (2) inelastic column buckling; (3) elastic local buckling; and (4) inelastic local buckling.

For overall buckling the use of the CRC parabolic curve, rewritten in terms of the normalized slenderness ratio

$$\lambda = 0.9(KL/D)\sqrt{\sigma_y/E}$$

is suggested for hot-formed, manufactured tubes. For fabricated cylinders, a more conservative straight line equation is given.

For local buckling purposes, the Guide provides several formulas and procedures. One of these is the Plantema equations used by AISI. No recommendations are given. A fictitious reduction of the yield strength is suggested to take into account the interaction between general and local buckling.

LOCAL BUCKLING OF PLATES

General and simplified formulas are reviewed for the computation of the critical buckling stress in the elastic and inelastic ranges. Plate thickness requirements, as determined by the critical buckling stress as well as by the use of the effective width concept based on post-buckling strength, are treated with special reference to specification practice.

The interaction between local and overall buckling for rolled structural shapes and box sections is also treated; it is concluded that, for the small deflection theory of plate buckling, this interaction is negligible when torsional instability is not an important factor.

However, this interaction should be considered when columns are designed to fail in the post-buckling range.

WEST EUROPE

(Key Document: ECCS Recommendations[24])

STEEL

The design criteria is

$$N \leq N_K \qquad \text{(WE1.1)}$$

where N is the design axial load and N_K is the limit axial load. N_K is obtained by entering the non-dimensional column curves (Fig. WE1.1) with the appropriate yield stress, together with the column curve selection tables (Tables WE1.1 and WE1.2).

For certain shapes a fictitious reduced yield stress has to be used, as indicated in the Selection Table WE1.1. This assures the adjustment of the column curves to the actual load carrying behavior of these particular shapes.

Commentary—The column curves are based on the ultimate strength concept of ECCS. The theoretical model is a hinged strut with an initial out-of-straightness at midspan of $l/1000$ and, depending on shape of cross section and manufacturing procedures, with residual stresses.[2]

The characteristic strength, as determined experimentally, is defined as mean value minus two times the standard deviation of a series of test results. The parameters for the theoretical analysis were chosen in such a way that the same safety level was obtained as defined for the experimental investigation.

The values of the column curves are given in the form of tables in the ECCS Recommendations.[24] Approximate equations were developed by Maquoi and Rondal[56] and by Lindner.[57]

The column curves are used as design curves for single, hinged members (simple members, built-up members, composite members) and for members in trusses. They are used as reference curves for the design of beam-columns. See Chapters 2, 5, 7, and 9.

For members which may fail by flexural-torsional buckling, the appropriate column curve for weak axis buckling has to be entered with a value of $\bar{\lambda}$ given by

$$\bar{\lambda} = \sqrt{\sigma_r/\sigma_{cr}} \qquad \text{(WE1.2)}$$

where σ_{cr} is the elastic critical stress for flexural-torsional buckling.

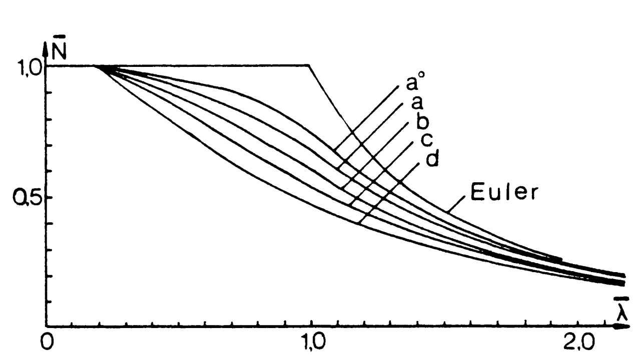

Fig. WE1.1. European column curves for steel

Table WE1.1. Column Curve Selection Table for Steel

Shape of Section		Steel: Fe 360, Fe 430, Fe 510 — Column Curve	Steel: Fe 360, Fe 430, Fe 510 — Yield Stress Reduction	Steel: $\sigma_r \geq 430$ N/mm² — Column Curve	Steel: $\sigma_r \geq 430$ N/mm² — Yield Stress Reduction
	Tubes (hot fin.):				
	Rolled	a	1.0	a	1.0
	Welded	a	0.94	a	0.94
	Box shape:				
	Welded	b	0.94	a	0.90
	Heat-treated	a	1.0	a°	1.0
	Welded H-shape:				
	About x-x:				
	Flame-cut fl.	b	0.94	a	0.90
	Rolled fl.	b	0.94	a	0.90
	About y-y:				
	Flame-cut fl.	b	0.94	a	0.90
	Rolled fl.	c	0.94	b	0.90
	Heat-treated H-shape:				
	About x-x	a	1.0	a°	1.0
	About y-y	b	1.0	a	1.0
	Rolled H-shape:				
	About x-x:				
	$h/b > 1.2$	a	1.0	a	1.0
	$h/b \leq 1.2$	b	1.0	a	1.0
	About y-y:				
	$h/b > 1.2$	b	1.0	b	1.0
	$h/b \leq 1.2$	c	1.0	b	1.0
	Coverplated rolled H-shape:				
	About x-x	b	0.94	a	0.90
	About y-y	a	0.94	a	0.90
	Tee	c	1.0	—	—
	Channel	c	1.0	—	—

Table WE1.2. Thick-Walled Sections ($t > 40$ mm)

Shape of Section		Column Curve
	Rolled H-shape:	
	About x-x	d
	About y-y	d
	Welded H-shape (rolled plates):	
	About x-x	c
	About y-y	d
	Welded H-shape (flame-cut pl.):	
	About x-x	c
	About y-y	c

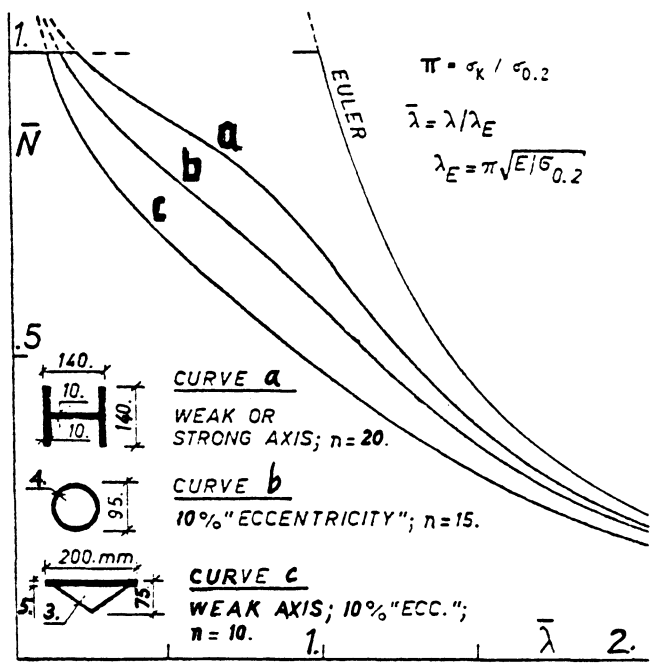

Fig. WE1.2. Column curves for aluminum alloy

Aluminum—The column curves shown in Fig. WE1.2 were developed following the same ultimate strength principles as established by ECCS for steel members. The geometrical imperfection in longitudinal direction is for all sections an initial curvature with $l/1000$. For hollow sections, an additional eccentricity of the applied load is considered due to not negligible departures of the wall thickness from the nominal values (10% "eccentricity"). The effect of residual stresses on column strength could be neglected for extruded members, but had to be considered for welded members, together with the influence of the heat-affected zones.

The stress-strain relationship adopted is the Ramberg-Osgood law

$$\epsilon = \frac{\sigma}{E} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^n \qquad \text{(WE1.3)}$$

with $E = 7000$ kN/cm². The hardening parameter n and $\sigma_{0.2}$ (kN/cm²) are assumed to be numerically equal.

Table WE1.3. Aluminum Alloy Column Curve Selection (Yield stress $\sigma_{0.2} \geq 100$ N/mm^2)

Cross Section			Aluminum Alloy		Elastic Limit Reduction Factor
Shape		Symmetry*	Heat-treated	Not Heat-treated	
Extruded	Open and Solid	Symmetrical	a	b	1
		Unsymmetrical	b	c	1
	Hollow	Symmetrical	b	c	1
		Unsymmetrical	c	c	1
Welded		Symmetrical	a	b	n (see Table WE1.4)

* About an axis perpendicular to the plane of buckling.

Table WE1.4. Reduction Factors for Column Curve Evaluation—Longitudinally Welded Aluminum Alloy Sections

Aluminum Alloy	Slenderness Range	$\sigma_K = n\ \sigma_{0.2}\overline{N}$
Heat-Treated	$0 < \bar{\lambda} \leq 1$	$n = \frac{A_r}{A} - \left(\frac{A_r}{A} - 0.85\bar{\lambda}\right)$
	$1 < \bar{\lambda} < 3$	$n = 0.775 + 0.075\bar{\lambda}$
	$\bar{\lambda} \geq 3$	$n = 1$
Not Heat-Treated	$0 < \bar{\lambda} \leq 1$	$n = 1 - 0.2\bar{\lambda}$
	$1 < \bar{\lambda} < 3$	$n = 0.7 + 0.1\bar{\lambda}$
	$\bar{\lambda} \geq 3$	$n = 1$

The values of the column curves are given in the form of tables and as equations in the ECCS Recommendations/Aluminium.[58] (See Tables WE1.3 and WE1.4.)

The theoretical background for the development of the design rules given above are contained in the papers by Mazzolani and Frey[59] and by Faella and Mazzolani.[60]

EAST EUROPE

[Key Documents: INCERC; CMEA Specifications for Steel Structures (see Ref. 61); CMEA Recommendations for Aluminium Structures]

STEEL

Flexural Buckling—Limit states design approach. Multiple column curves and selection tables corresponding to ECCS Recommendations are proposed.

In preparation of a Regional Recommendation for Steel Structures a complete probabilistic approach is being considered, resulting in column curves depending on the value of tolerable risk and coefficient of variation of loads as well.

Flexural Torsional Buckling—Analysis is reduced to that of flexural buckling by introducing a fictitious slenderness ratio:

$$\lambda = \pi \sqrt{\frac{E \cdot A}{N'_{cr}}} \qquad \text{(EE1.1)}$$

where N'_{cr} is the theoretical critical axial force causing flexural-torsional buckling according to the elastic theory.

Effective Length—Effective length factors are given for axially loaded columns with uniform cross sections or cross sections varying continuously or stepwise, with usual end conditions.

Local Buckling of Plates—Plate thickness requirements are based on rules of Soviet specifications (see "Specifications and Codes," Fig. EE1.3).

ALUMINUM

Limit states design approach. Alloys are grouped according to their conventional yield stress $\sigma_{0.2}$:

$$\text{Group I:} \quad \sigma_{0.2} \leq 120 \text{ MPa}$$
$$\text{Group II:} \quad \sigma_{0.2} \geq 120 \text{ MPa}$$

One column curve for Group I and two column curves for closed and open cross sections, respectively, in Group II, are given. Limit loads are defined as follows:

a. At small and medium values of slenderness ratio (λ), by the peak-load of a column with Weinhold's[62] stress-strain diagram and initial eccentricity, $e = e_o W/A$, where e_o is:

$$\text{Group I:} \quad e_o = 0.0075\ \lambda$$
$$\text{Group II:} \quad e_o = 0.0030\ \lambda$$

b. At large values of λ, by ~70% of the Euler load.

b. SPECIFICATIONS AND CODES

JAPAN

AIJ STANDARD FOR STEEL STRUCTURES (see Fig. J1.1)

Equations giving the allowable stress in the allowable stress design approach are:

For $\lambda \leq \Lambda$:

$$\sigma_w = \sigma_y \frac{\left[1 - 0.4\left(\frac{\lambda}{\Lambda}\right)^2\right]}{\left[\frac{3}{2} + \frac{2}{3}\left(\frac{\lambda}{\Lambda}\right)^2\right]} \tag{J1.1}$$

For $\lambda > \Lambda$:

$$\sigma_w = \frac{0.277\sigma_y}{(\lambda/\Lambda)^2} \tag{J1.2}$$

where $\Lambda = \sqrt{\pi^2 E/0.6\sigma_y}$ and $\lambda = l/r$, l and r are effective slenderness ratio, effective length and radius of gyration of the member, respectively. Maximum permitted value of λ is 250 for compression members, in general, and 200 for columns.

Commentary—This formula is based on a tangent-modulus theory. The proportional limit is assumed to be $0.6\sigma_y$ to take account of geometrical imperfections and material inhomogeneities. The factor of safety takes into consideration the fact that the experimental results scatter in the range where $\lambda \simeq \Lambda$. The factors of safety are 1.5 for a column of zero length and 2.17 for a long column.

AIJ Standard presents provisions for members having variable cross sections and lateral supports as follows. For the purpose of design computation, a member having variable cross section may be converted into a prismatic member which is capable of carrying an equivalent elastic buckling load.

For the lateral supports of compression members:

1. The intermediate supports of a continuous compression member shall be given sufficient strength and stiffness to enable the member to maintain full load-carrying capacity, even if it has initial deflection.
2. Unless the exact computation is made, laterally supporting members or frames shall be assumed to be subject to concentrated lateral force equal to or not less than 2% of the axial compression. Where a compression member has a large initial deflection, or where rigidity of laterally supporting members or frames is small, the foregoing concentrated lateral force and effective length of the compression member shall be increased.

AIJ GUIDE FOR PLASTIC DESIGN OF STEEL STRUCTURES (see Fig. J1.2)

Equations giving the ultimate strength in the plastic design approach are:

$$\left.\begin{aligned}
&\text{For } 0 \leq \bar{\lambda} \leq 0.3: \\
&\qquad \sigma_u = \sigma_y \\
&\text{For } 0.3 < \bar{\lambda} \leq 1.3: \\
&\qquad \sigma_u = \sigma_y[1 - 0.545(\bar{\lambda} - 0.3)] \\
&\text{For } \bar{\lambda} > 1.3: \\
&\qquad \sigma_u = \sigma_y/(1.3 \cdot \bar{\lambda}^2)
\end{aligned}\right] \tag{J1.3}$$

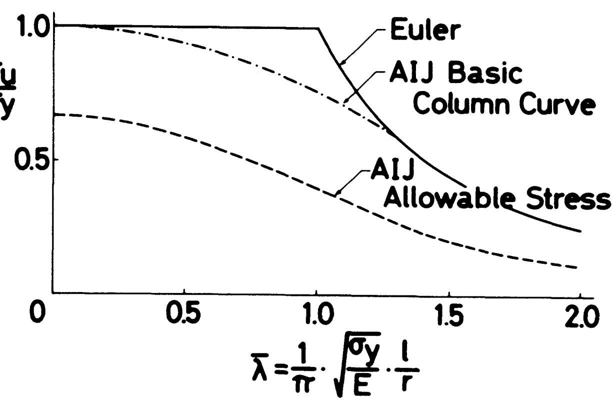

Figure J1.1

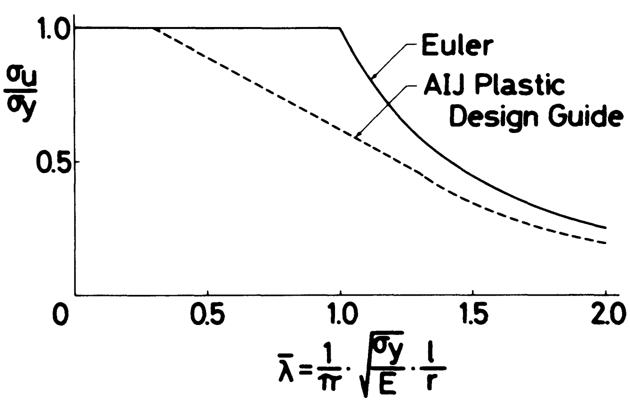

Figure J1.2

where $\bar{\lambda} = (1/\pi)\sqrt{\sigma_y/E} \cdot \lambda$, and $\Lambda = l/r$, l and r are effective slenderness ratio, effective length and radius of gyration of the member, respectively.

Maximum permitted value of λ for columns is 200.

Commentary—The denominator 1.3 of the last equation in Eq. (J1.3) is nearly equal to 2.17/1.65, which is the ratio of the factor of safety in the allowable stress design of AIJ Standard for Steel Structures to the load factor used in this plastic design AIJ Guide.

JSCE SPECIFICATIONS FOR STEEL RAILWAY BRIDGES (see Fig. J1.3)

Equations giving the allowable stress in N/mm^2 in the allowable stress design approach are:

For SS41, SM41 and SMA41:

$$\left.\begin{array}{ll} 0 < \lambda \le 28: & \sigma_w = 123 \\ 28 < \lambda \le 130: & \sigma_w = 123 - 0.785(\lambda - 28) \\ 130 < \lambda: & \sigma_w = 726000(1/\lambda)^2 \end{array}\right] \quad (J1.4)$$

For SM50:

$$\left.\begin{array}{ll} 0 < \lambda \le 24: & \sigma_w = 167 \\ 24 < \lambda \le 115: & \sigma_w = 167 - 1.23(\lambda - 24) \\ 115 < \lambda: & \sigma_w = 726000(1/\lambda)^2 \end{array}\right] \quad (J1.5)$$

For SM50Y, SM53, SMA50:

$$\left.\begin{array}{ll} 0 < \lambda \le 22: & \sigma_w = 186 \\ 22 < \lambda \le 105: & \sigma_w = 186 - 1.45(\lambda - 22) \\ 105 < \lambda: & \sigma_w = 726000(1/\lambda)^2 \end{array}\right] \quad (J1.6)$$

For SM58, SMA58:

$$\left.\begin{array}{ll} 0 < \lambda \le 20: & \sigma_w = 235 \\ 20 < \lambda \le 95: & \sigma_w = 235 - 2.07(\lambda - 20) \\ 95 < \lambda: & \sigma_w = 726000(1/\lambda)^2 \end{array}\right] \quad (J1.7)$$

where λ is the effective slenderness ratio.

Maximum permitted effective slenderness ratios λ are 100 and 120 for main and secondary compression members, respectively.

Commentary—The JSCE column-strength curve is a straight line which is cut off at $\bar{\lambda} = 0.3$ and meets the Euler curve at the stress $\sigma_E = 0.5\ \sigma_y$. This curve corresponds to the strength of a column with large residual stresses such as in built-up members by welding. The values of the factor of safety increase from 1.9 for $\bar{\lambda} \le 0.3$ to 2.8 for $\bar{\lambda} \ge \sqrt{2}$.

Guaranteed minimum yield stresses (N/mm^2) for plates of $16 < t < 40$ by Japan Industrial Standard are 235 for SS41, SM41, and SMA41, 314 for SM50, 353 for SM50Y, SM53, and SMA50, and 451 for SM58 where t (mm) is the thickness of the plate.

JRA SPECIFICATIONS FOR HIGHWAY BRIDGES (see Fig. J1.4)

Equations giving the allowable stress in N/mm^2 in the allowable stress design approach are:

For SS41, SM41, SMA41:

$$\left.\begin{array}{ll} \lambda \le 20: & \sigma_w = 137 \\ 20 < \lambda < 93: & \sigma_w = 137 - 0.824(\lambda - 20) \\ 93 \le \lambda: & \sigma_w = 1180000/(6700 + \lambda^2) \end{array}\right] \quad (J1.8)$$

For SS50:

$$\left.\begin{array}{ll} \lambda \le 17: & \sigma_w = 167 \\ 17 < \lambda < 86: & \sigma_w = 167 - 1.11(\lambda - 17) \\ 86 \le \lambda: & \sigma_w = 1180000/(5700 + \lambda^2) \end{array}\right] \quad (J1.9)$$

For SM50:

$$\left.\begin{array}{ll} \lambda \le 15: & \sigma_w = 186 \\ 15 < \lambda < 80: & \sigma_w = 186 - 1.27(\lambda - 15) \\ 80 \le \lambda: & \sigma_w = 1180000/(5000 + \lambda^2) \end{array}\right] \quad (J1.10)$$

For SM50Y, SM53, SMA50:

$$\left.\begin{array}{ll} \lambda \le 14: & \sigma_w = 206 \\ 14 < \lambda < 76: & \sigma_w = 206 - 1.47(\lambda - 14) \\ 76 \le \lambda: & \sigma_w = 1180000/(4500 + \lambda^2) \end{array}\right] \quad (J1.11)$$

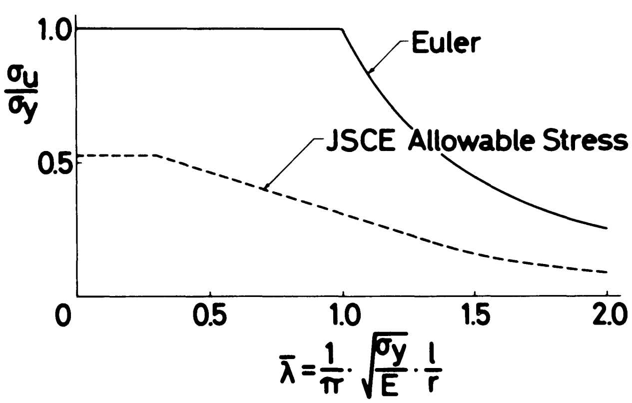

Figure J1.3

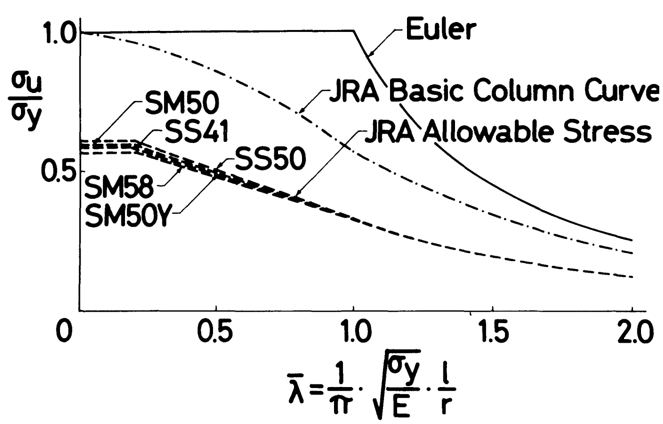

Figure J1.4

For SM58, SMA58:

$$\left.\begin{array}{lll} \lambda \le 14: & \sigma_w = 255 \\ 14 < \lambda < 67: & \sigma_w = 255 - 2.06(\lambda - 14) \\ 67 \le \lambda: & \sigma_w = 1180000/(3600 + \lambda^2) \end{array}\right] \quad \text{(J1.12)}$$

where λ is the effective slenderness ratio.

Maximum permitted effective slenderness ratios in truss members are 120 and 150 for main and secondary compression members, respectively.

Commentary—The basic column curve is obtained from the theoretical weak axis buckling strength of an I-profile which has a maximum compressive residual stress of $0.4\sigma_y$ and out-of-straightness of $l/1000$.

It is approximately expressed by the following equations:

$$\left.\begin{array}{l} \text{For } \bar{\lambda} \le \bar{\lambda}_1(=1): \\ \qquad \sigma_u/\sigma_y = 1 - 0.136\bar{\lambda} - 0.300\bar{\lambda}^2 \\ \text{For } \bar{\lambda} > \bar{\lambda}_1: \\ \qquad \sigma_u/\sigma_y = 1.276 - 0.888\bar{\lambda} + 0.176\bar{\lambda}^2 \end{array}\right] \quad (1.13)$$

where

$$\bar{\lambda} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{l}{r}$$

The first equation of Eq. (J1.13) is replaced by two straight lines for covering larger residual stresses than assumed above and the second equation is replaced by a simpler equation, that is,

$$\sigma_u/\sigma_y = 1/(0.773 + \bar{\lambda}^2) \qquad \text{(J1.14)}$$

Equations (J1.8) through (J1.12) are obtained from this column curve with a uniform safety factor of approximately 1.7.

Guaranteed minimum yield stresses for SS41, etc., are shown in the commentary to the preceding specification.

AIJ STANDARD FOR ALUMINIUM STRUCTURES (Draft form)

Design formula is similar in form to that for steel structures. The mechanical properties of aluminum alloys are duly reflected in the formula:

$$\left.\begin{array}{l} \text{For } 20 < \lambda \le \Lambda: \\ \qquad \sigma_w = \sigma_y \dfrac{\left[1 - \alpha\left(\dfrac{\lambda}{\Lambda}\right)^2\right]}{\left[1.5\left(0.9 + 0.6\dfrac{\lambda}{\Lambda}\right)\right]} \\ \text{For } \lambda > \Lambda: \\ \qquad \sigma_w = \dfrac{1-\alpha}{2.25\left(\dfrac{\lambda}{\Lambda}\right)^2}\sigma_y \end{array}\right] \quad \text{(J1.15)}$$

where α and Λ are specified according to the grades of aluminum alloys.

NORTH AMERICA

STEEL

Flexural Buckling—An allowable stress design usually is used in most of the specifications, even where in certain codes a limit state design has recently been allowed. In any event, whatever the approach, the strength of the column is computed with reference to only one column curve.

Allowable Stress Design—Figure NA1.2 shows some of the curves given for the allowable stress σ_a for steel grade ASTM A36.

AISC (Buildings, 1978); AASHTO (Bridges, 1978):

The following formulas define σ_a:

$$\left.\begin{array}{l} \text{When } KL/r < C_c \text{ (inelastic buckling):} \\ \qquad \sigma_a = \sigma_y[1 - (KL/r)^2/2C_c^2]/\text{F.S.} \\ \text{When } KL/r > C_c \text{ (elastic buckling):} \\ \qquad \sigma_a = \dfrac{\pi^2 E}{\text{F.S.}(KL/r)^2} \end{array}\right] \quad \text{(NA1.1)}$$

where

$$C_c = \frac{\sqrt{2\pi^2 E}}{\sigma_y}$$

K = effective length factor

Commentary:

Formulas (NA1.1) are a design approximation that use the CRC basic column curve divided by a factor of safety, F.S.

The CRC curve, presented in the first edition of the Guide, provides an estimation of the maximum capacity of columns of light and medium weight shapes based on the tangent modulus theory, modified to take into account yield stress variation as well as the presence of residual stresses. It uses one curve for both axes of bending and assumes the value of σ_u of half the yield stress as the upper limit of the elastic buckling range.

The AISC specification uses a value of F.S. increasing with KL/r to compensate for the difficulty in determining the value of K and for the initial crookedness; AASHTO uses a constant, but higher, value of F.S. Limiting values of L/r are specified in both codes.

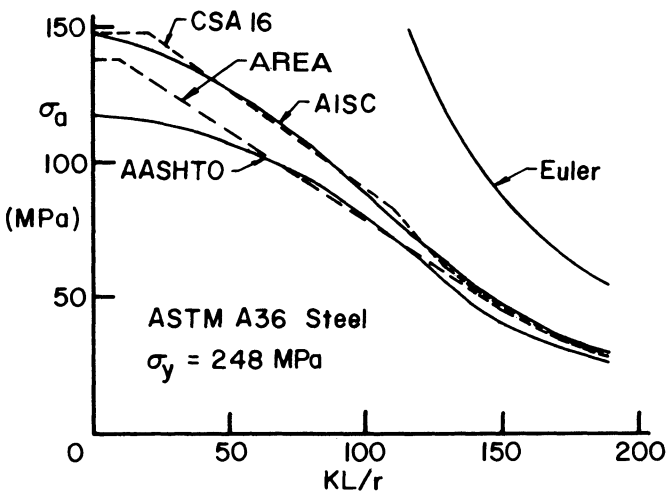

Figure NA1.2

AISI (Cold-Formed Members, 1968):

The CRC curve basically is used with the introduction of a factor Q which allows for the local buckling. More details will be given in Chapter 11.

AREA (Bridges, 1978); CSA 16 (Buildings, 1969); CSA 56 (Bridges, 1978):

A straight line is used in these specifications to approximate the theoretical curve in the inelastic range.

Limit State Design

AASHTO (Bridges, 1978):

A limit state design is allowed; the column strength is computed as $P_u = 0.85A\sigma_u$ where σ_u is determined by Eq. (NA1.1) with a factor of safety equal to one.

CSA 16.1 (Buildings, 1978); CSA S6 (Bridges, 1978):

The SSRC Curve 2 (see Fig. NA1.1) is used to compute the factored compressive resistance of the member. Limitations are given to the geometry of the sections in order to guarantee that the flexural buckling is the actual limit state.

AISC:

A study is under way to evaluate the possibility of introducing the Load Resistance Factor Design (LRFD) approach in future editions of the AISC specification.

Effective Length—Values of the effective length factors dependent on the end restraints are usually given, often identical to the values proposed by the SSRC.

The alignment charts provided by the SSRC for framed columns are suggested as a rational method to compute the value of the K factor for that case in many codes; AISC and AASHTO specifications allow for a correction to K to take into account buckling in the inelastic range.

Local Buckling of Plates—AISC, AREA, AASHTO, and CSA specify limiting values for the width-to-thickness ratio, b/t, in order to ensure that local buckling does not reduce the overall capacity of the member. These values, although somewhat different, are ultimately based on the critical buckling stress of stiffened or unstiffened plates. CSA limit state design provides a further reduction of the b/t ratio if moment redistribution without local buckling has to be considered.

AISC specifications allow the above values to be exceeded, provided that the allowable compressive stress is reduced. An effective width must then be computed for stiffened elements; for flanges of square and rectangular sections of uniform thickness, the following formula is applicable:

$$b_e/b = \frac{664t}{\sqrt{\sigma_c}}\left(1 - \frac{132}{b/t\sqrt{\sigma_c}}\right) \leq 1 \qquad \text{(NA1.2)}$$

where b is the actual width, t is the thickness, and σ_c is the compressive stress in the element expressed in MPa.

The interaction between local and general buckling is considered by introducing appropriate multiplying factors into Eq. (NA1.1).

Buckling of Axially Loaded Tubular Columns

AWWA (Waterworks, 1973):

An allowable stress design is specified based essentially on experimental results with a factor of safety equal to two applied against local buckling. General and local buckling are checked simultaneously by computing the allowable stress as a product of two parameters X and Y to account respectively for the two types of buckling.

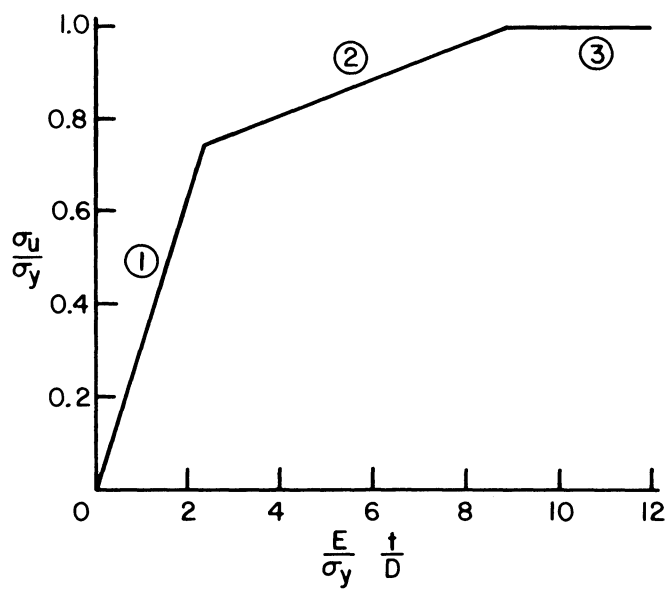

Figure NA1.3

AISI (1968); AISC (1978):

Tubular columns with ratio $D/t < 22753/\sigma_y$ must be designed according to the equations for flexural buckling of axially loaded columns with $Q = 1$.

For D/t between $22753/\sigma_y$ and $89635/\sigma_y$, a linear decrease in the compressive stress σ_c is given by the use of the formula

$$\sigma_c/\sigma_y = 0.0368\alpha + 0.665 \qquad \text{(NA1.3)}$$

where $\alpha = Et/\sigma_y D$.

D/t ratios larger than $89635/\sigma_y$ are governed by the inclined line 1 in Fig. NA1.3.

Commentary:

The AISI and AISC provisions are similar to the Plantema equations, except that they are somewhat more conservative.

ALUMINUM

Aluminum Association (1971):

Flexural Buckling—The Design Specifications of the Aluminum Association as well as the earlier ones published by ASCE are based on the tangent modulus theoretical approach, with a straight line approximation to the tangent modulus curve in the inelastic range. Distinction is made between artificially and non-artificially aged alloys. A constant factor of safety of 1.95 is used.

Local Buckling of Plates—Formulas are given to compute the critical stress of the component plates, whatever the b/t ratio. A straight line approximation is used in the inelastic range. No allowance is made to consider the post-buckling strength of the plate.

Buckling of Axially Loaded Tubular Columns—Allowable stresses are given for local buckling in the inelastic and elastic ranges. The formulas take into account the effects of imperfections.

WEST EUROPE

STEEL

Most Western European codes are still based on allowable stress design (with column curves derived from bifurcation theory, Perry–Robertson model, ultimate strength theory). However, in most cases revisions are underway to introduce principles of limit states design.

According to a survey in 1978, the ECCS column curves have been adopted in the original or in a modified version by the following countries in their existing codes or in their present code revisions:

Applied in allowable stress design:

Italy	CNR-UNI 10011	(1979)
Belgium	NBN 51-001	(1977)

Applied in limit states design:

Belgium	NBN 51-001	(Draft 1979)
France	Recommendations for Plastic Design (Curves "a" and "b" only)	(1975)
German F.R.	DIN 4114 (Simplified selection chart)	(1979)
Great Britain	B.S. 5400	(Draft 1979)
Switzerland	SIA 161 (Simplified selection chart)	(1979)
Norway	3472 A	(Draft 1979)
Yugoslavia	JUS UE 7081	(1978)

ALUMINUM

The Western European countries that have a code for the design of aluminum structures are: Austria (1977), Belgium (NBN 1-50, 1968), France (D.T.U., 1976), German F.R. (DIN 4113, 1975), Great Britain (CP 118, 1969), and Sweden (SVR, 1970).

The corresponding column curves are derived from different theories (bifurcation theory, Perry–Robertson model, Dutheil method, ultimate strength theory) and they are applied both in allowable stress design (A, B, GFR, GB, S) and in limit states design (F).

EAST EUROPE

STEEL

Soviet Union: SzNiP II-V.3-72 (see Ref. 63)

Limit states design approach; column curves for seven steel grades (σ_t = 380, 440, 460, 520, 600, 700, and 800 MPa) are given.

Limit load is defined as follows:

a. At small and medium values of slenderness ratio (λ) by the elastic-plastic peak load of a column with unified stress-strain diagram and initial eccentricity

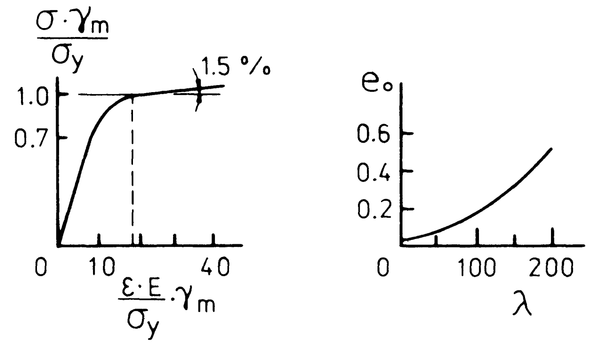

Figure EE1.1

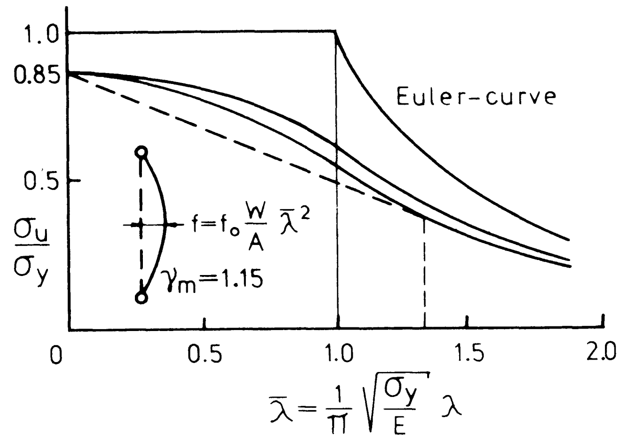

Figure EE1.2

$e = e_0 W/A$ according to Fig. EE1.1; σ_y/γ_m denoting the design value of yield stress.

b. At large values of λ by ~70% of the Euler-load.

Soviet Union: Bridges (see Ref. 64)

Initial eccentricity $e_o = 0.125 + 0.0018\lambda$ is chosen. For I shapes a second set of curves is given, taking into account residual compressive stress of the magnitude $\sigma_{res} = 0.4\sigma_y$.

Czechoslovakia: CSN 731401/1976 (see Ref. 65)

Limit states design approach; limit load is defined by the onset of yielding in case of initial curvature (expressing the effect of all imperfections) with the amplitude $f_o = 0.17$ for tubes and closed sections, and $f_o = 0.26$ for open sections, resulting in

$$\frac{\sigma_u}{\sigma_y}\gamma_m = \frac{1+f_o}{2} + \frac{1}{2\bar{\lambda}^2} - \sqrt{\left(\frac{1+f_o}{2} + \frac{1}{2\bar{\lambda}^2}\right)^2 - \frac{1}{\bar{\lambda}^2}} \qquad \text{(EE1.2)}$$

For flexural-torsional buckling of L and T shapes, the dashed line in Fig. EE1.2 can be used.

Hungary: MS$_z$ 15024/75 (see Ref. 66)

Limit states design approach; multiple column curves based on the results of a probabilistic method are given by the interaction formula

$$\frac{\sigma_u}{\sigma_y} - \frac{a\gamma_m}{b}\frac{\sigma_u}{\sigma_y}\frac{\sigma_u}{\sigma_E} + a\frac{\sigma_u}{\sigma_E} = \frac{1}{\gamma_m} \qquad \text{(EE1.3)}$$

(σ_E being the Euler-stress) and

A. $a = 1.2,\quad b = 1.500$
B. $a = 1.3,\quad b = 1.625$
C. $a = 1.3,\quad b = 3.250$

for different cross sections specified by a selection table.

Rumania: STAS 10108/0–76 (see Ref. 67)

Limit states design approach; multiple column curves and selection tables are given. Equation of column curves is the same as Eq. (EE1.2) with

$$\begin{aligned} &\text{A.}\quad f_o = 0.2 \\ &\text{B.}\quad f_o = 0.3 \\ &\text{C.}\quad f_o = 0.5 \end{aligned} \qquad \text{(EE1.4)}$$

Poland: PN-76/B-03200 (see Ref. 68)

Limit states design approach; a single column curve is given for various steel grades in form of:

For $\lambda \leq \lambda_p$:

$$\frac{\sigma_u}{\sigma_y}\gamma_m = 1 - 0.2\left(\frac{\lambda}{\lambda_p}\right) - 0.1\left(\frac{\lambda}{\lambda_p}\right)^2 - 0.2\left(\frac{\lambda}{\lambda_p}\right)^3$$

For $\lambda > \lambda_p$:

$$\frac{\sigma_u}{\sigma_y}\gamma_m = 0.5\left(\frac{\lambda_p}{\lambda}\right)^2 \qquad \text{(EE1.5)}$$

where

$$\lambda_p = \pi\sqrt{\frac{E\gamma_m}{0.75\sigma_y}}$$

German Democratic Republic: TGL 13503-1972 (see Ref. 69)

Allowable stress design approach, using multiple column curves. Allowable stress is defined as follows:

a. At small and medium values of slenderness ratio (λ) by $\sigma_w = 0.67\sigma_K$, where σ_K stands for the mean axial stress causing onset-of-yielding in case of initial eccentricity $e = e_o W/A$.

b. At large values of λ by $\sigma_w = 0.5\sigma_E$ (in case of bridges $\sigma_w = 0.4\sigma_E$).

Initial eccentricity for steel grades $\sigma_t = 380 \sim 520$ MPa:

Curve 1:

$\lambda \leq 250$: $e_o = 0.004\lambda$

$\lambda > 250$: $e_o = 0.005\lambda - 0.25$ (EE1.6)

Curve 2:

$\lambda \leq 100$: $e_o = 0.0025\lambda$

$\lambda > 100$: $e_o = 0.005\lambda - 0.25$ (EE1.7)

Curve 3.

$\lambda \leq 100$: $e_o = 0.000025\lambda^2$

$\lambda > 100$: $e_o = 0.005\lambda - 0.25$ (EE1.8)

For steel grade $\sigma_t = 600$ MPa:

Curve 1:

$\lambda \leq 40$: $e_o = 0.0025\sqrt{\lambda}$

$40 < \lambda \leq 250$: $e_o = 0.004\lambda$ (EE1.9)

$\lambda > 250$: $e_o = 0.005\lambda - 0.25$

Curve 2:

$\lambda \leq 100$: $e_o = 0.025\sqrt{\lambda}$

$\lambda > 100$: $e_o = 0.005\lambda - 0.25$ (EE1.10)

Curve 3:

$\lambda \leq 100$: $e_o = 0.0025\lambda$

$\lambda > 100$: $e_o = 0.005\lambda - 0.25$ (EE1.11)

Yugoslavia: Stability of Steel Structures, 1964 (see Ref. 70)

Allowable stress design approach, using tangent modulus theory with stress-strain relation

$$\sigma = \sigma_p + (\sigma_y - \sigma_p) \sin \frac{E\epsilon - \sigma_p}{\sigma_y - \sigma_p}, \quad \text{(EE1.12)}$$

where σ_p denotes the limit of proportionality. Safety factor $n_E = 2.6$ in the elastic range, and

$$n = 1.5 + 1.1 \left(\frac{\lambda}{\lambda_p}\right)^2$$

in the plastic range is chosen, where

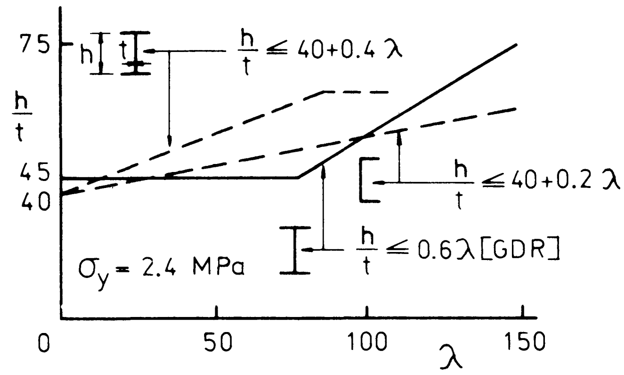

Figure EE1.3

$$\lambda_p^2 = \pi^2 E/\sigma_p$$

Draft of new specifications adopts ECCS Recommendations.

General—Dealing with interaction of column and plate buckling, most specifications (GDR, Hungary) use Bleich's concept requiring equal critical stress for local and overall buckling, resulting in design formulas represented by the solid line in Fig. EE1.3.

Soviet specifications analyze plate buckling—using Stowell's theory—taking into account a non-uniform stress distribution along the cross section due to the effect of initial imperfections resulting in design formulas represented by the dashed lines in Fig. EE1.3.

Soviet, Czechoslovakian and GDR specifications allow for utilizing post-buckled behavior, substituting width h by effective width $h_e = 30 \sim 40t$.

ALUMINUM

A similar approach is followed by most specifications. Some details, such as effective length factors, are parallel to those in steel specifications.

C. COLLOQUIUM CONTRIBUTIONS (COMMENTARIES)

JAPAN

Y. FUKUMOTO, N. KAJITA, AND T. AOKI (TOKYO)[21]

This report presents the results of an experimental and theoretical investigation into the probabilistic behavior and strength of steel columns. The test program consisted of 248 column test and supplementary tests. The random variation of the column strength parameters was detected experimentally and the effects of the variety of the column strength parameters on the random variation of the maximum column strength were analyzed on the probabilistic bases. Figure J1.5 shows the comparison of strength histograms between theory and test for welded H columns.

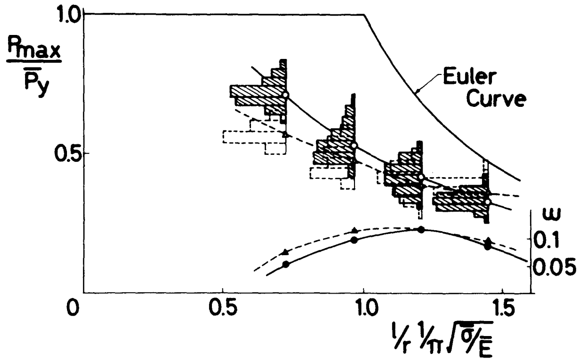

Fig. J1.5. Comparison of theoretical and test strength histograms for welded H columns

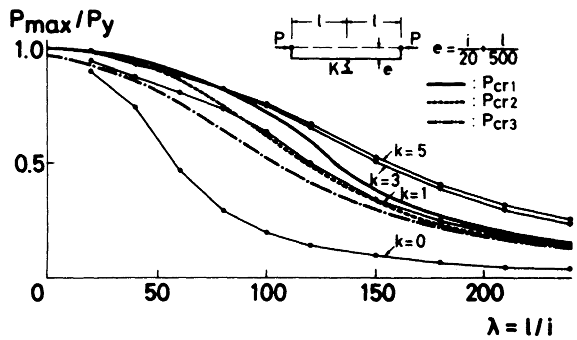

Figure J1.6

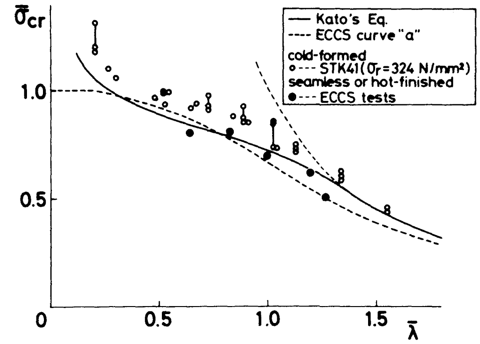

Figure J1.7

C. MATSUI AND K. YAGI (LIEGE)[15]

This paper presents the results of the elastic-plastic analysis of laterally braced compression members. The maximum compressive strength of the braced member can attain the expected strength P_{cr} under the condition that the brace stiffness $k \geq 3$ as shown in Fig. J1.6. The ratios of the force in the bracing member to that in the braced member are less than 0.02 in any case under the condition that $k \geq 3$.

B. KATO (LIEGE)[26]

The paper presents the estimation of deformational residual stresses and locked-in stresses confined in cold-formed and welded steel tubes during the course of manufacturing, and the effect of these stresses on buckling strength of tubular columns. The column buckling curve obtained from tangent modulus evaluated on the basis of a number of stub column tests shows good accordance with available test results as shown in Fig. J1.7. It is also shown that there are no substantial differences between this curve and curve *a* of ECCS in magnitude.

NORTH AMERICA

W. F. CHEN (LIEGE)[71]

The paper presents the first results of an experimental investigation on fabricated tubular columns, statically loaded under pin-ended conditions. Longitudinal and circumferential residual stresses are measured. The results plotted in Fig. NA1.4 show a good agreement with the AISC factored column formulas.

D. H. HALL (WASHINGTON)[72]

The paper presents a statistical method to examine the importance on the column strength of the following five parameters: yield stress (σ_y), slenderness ratio (KL/r), pattern of residual stress (pat), maximum compression residual stress (σ_r), initial crookedness ($crook$). Linear equations are given for different cases.

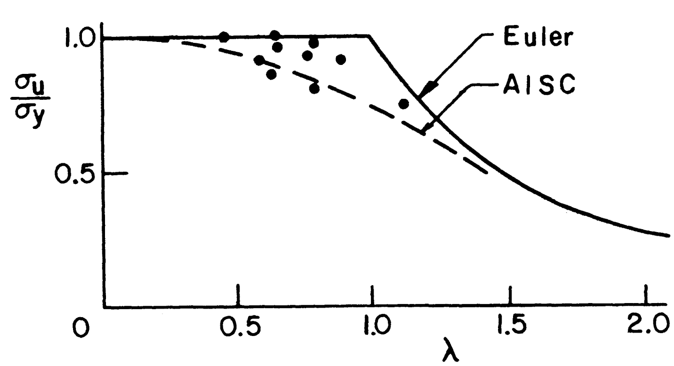

Figure NA1.4

J. W. CLARK (WASHINGTON)[73]

A study of the influence of axial eccentricities on the strength of elastically-restrained aluminum columns is carried out using simplified methods for upper and lower bounds analysis. It was shown that for design purposes the effect of the geometric imperfection on end-restrained columns can be assumed the same as in the case of pinned-end columns, provided the effective length factor K is between 0.7 and 1.0.

A. OSTAPENKO (WASHINGTON)[74]

The paper presents test results of an experimental study on the local buckling of axially loaded welded tubular columns. Based on the test results the following formula is recommended for 365 MPa Steel.

For $c < 0.07$:

$$\sigma_c/\sigma_y = 38c - 480c^2 + 2000c^3 \qquad \text{(NA1.4a)}$$

For $c \geq 0.07$:

$$\sigma_c/\sigma_y = 1.0 \qquad \text{(NA1.4b)}$$

where

$$c = \sqrt[3]{E/\sigma_y}\left(\frac{1}{D/t}\right)$$

The parameter c is introduced in place of the customary parameter

$$\propto = \left(\frac{E}{\sigma_y}\right)\left(\frac{1}{D/t}\right)$$

WEST EUROPE

N. F. YEOMANS (LIEGE)[75]

New buckling tests conducted by CIDECT with seamless tubes and welded, hot-finished tubes are described. The statistically evaluated test results all fall above ECCS curve a. The new tests show no significant differences in column strength between the two types of tubes.

G. BALLIO, L. FINZI, AND C. URBANO (LIEGE)[25]

The application of the ECCS column curves for different types of tubular members of high-strength steel is experimentally and theoretically investigated. Figure WE1.3 shows that ECCS curve a can be used as a design curve for welded rectangular tubes built-up from cold-formed channels made of Nicuage steel ($\sigma_r = 670\ \text{N/mm}^2$), when for the weak axis case a modified slenderness is introduced. Figure WE1.4 shows that cold-formed tubes of high-strength steel with a wall-thickness up to 10 mm can be designed according to ECCS curve c.

L. FINZI AND C. URBANO (WASHINGTON)[76]

For the use of the practicing engineer, the five ECCS column curves are related to one single curve (in the example shown, to curve a^0) by means of a modified slenderness ratio. Modified ratios of inertia can be tabulated for the various cross sections.

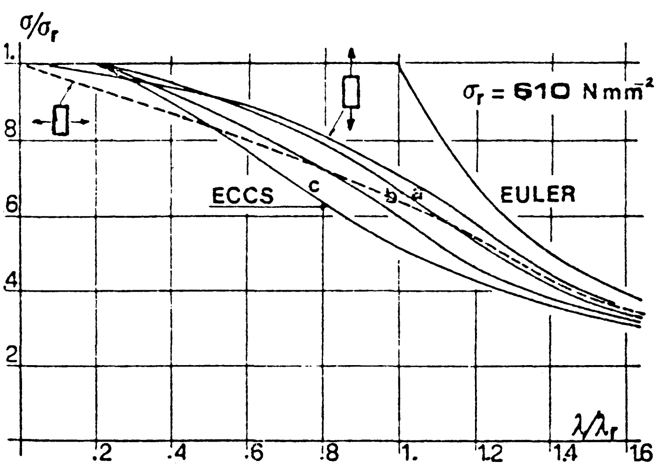

Figure WE1.3

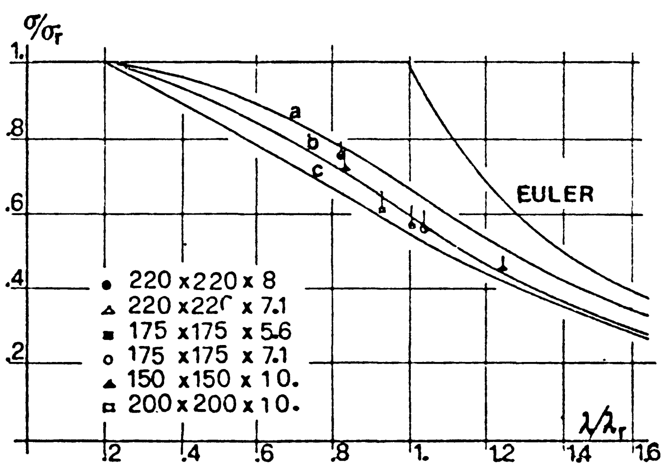

Figure WE1.4

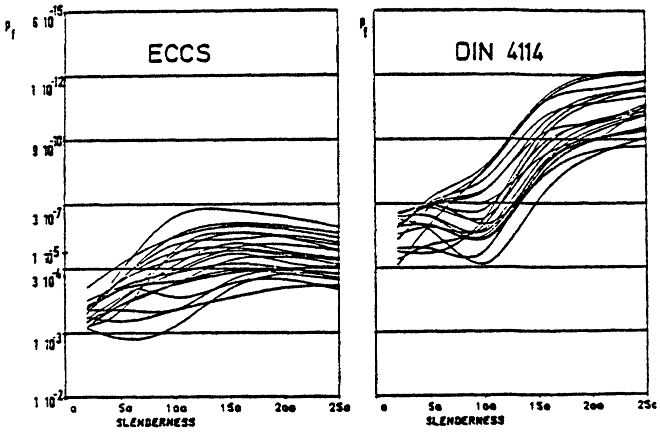

Figure WE1.5

R. HAWRANEK (LIEGE)[77]

The paper relates the reliability level of the ECCS column curves and of the column curve of the German Code DIN 4114 (1952) to a probability based study of column strength. Figure WE1.5 allows qualitative conclusions on the failure probability of I-sections designed according to ECCS curve *b* and DIN 4114 (Edition 1952, presently under revision).

F. M. MAZZOLANI AND C. FAELLA (LIEGE)[78]

The behavior of high-strength steel bars under repeated axial loadings is investigated. The influence of imperfections on the load carrying capacity during the loading history is studied. The influence of residual stresses on the load carrying capacity in compression tends to disappear after several cycles of preloadings in tension. The reduction of column strength through the Bauschinger effect was found to be most pronounced for the ideal model of the stress-strain relationship.

F. M. MAZZOLANI AND F. FREY (LIEGE)[59]

The paper presents the work conducted by ECCS Committee 2 "Aluminium-Alloy Structures" on the buckling behavior of extruded members. The theoretical and experimental background of the relevant ECCS Recommendations is described.

EAST EUROPE

J. MELCHER (BUDAPEST)[79]

The theoretical background of the Czechoslovakian column curves and their comparison to those given by other specifications is presented in the paper. The possible differences in the behavior of an isolated column and a compressed component of a real structure are emphasized.

V. FIRBASOVA (BUDAPEST)[80]

Paper gives a numerical method for the analysis of a compressed hybrid column of double-symmetric cross section in case of simplified and more complex stress-strain diagram.

K. RYKALUK (BUDAPEST)[81]

The effect of residual stresses on the critical load of centrally compressed short columns of double-symmetric cross section losing their stability by the torsional mode of buckling is dealt with. Results are summarized by means of modification of the classical elastic solution.

E. SCHLECHTE (BUDAPEST)[82]

Paper raises the possibility of giving simple criteria for cases where flexural-torsional buckling can be disregarded and check of flexural buckling suffices. Amplification factor for unequal end-eccentricities and simultaneous action of uniformly distributed load is suggested.

J. MURZEWSKI (BUDAPEST)[50]

Inelastic instability is investigated using probability considerations and defining ultimate load as the minimum of two random variables: squash load and elastic critical load. Applications for columns and cylindrical sheels are demonstrated assuming Weibull distribution and no correlation between variables.

E. MISTÉTH (BUDAPEST)[49]

Probability density function of strength of steel is given based on a mixture of two distributions expressing plastic and rigid properties of steel.

M. TOCHACEK AND P. FERJENCIK (BUDAPEST)[83]

Special problems of prestressed steel structures are analyzed, pointing out cases where:

a. Buckling resistance may be deteriorated
b. Buckling strength is increased
c. Dealing with prestressed flexible elements subjected to subsequent compression.

A. A. POTAPKIN (BUDAPEST)[64]

Up-to-date methods are worked out for stability analysis of steel bridges. Practical design rules given in related specifications are summarized.

d. CURRENT STUDIES

JAPAN

From the viewpoint of seismic design of steel structures, the study of the post-buckling behavior of columns is necessarily associated with the energy absorption capacity. An investigation of this problem has been started and is being continued by Kato, Akiyama, and Inoue (University of Tokyo, 1975).[84]

A theoretical evaluation of the column curves on the basis of probabilistic safety concept is being carried out by Fujimoto and Iwata[85] and Fujimoto, Iwata, and Nakatani.[86]

NORTH AMERICA

In addition to the colloquium contributions, the following studies are currently underway:

1. Influence of end restraint on the strength of initially crooked columns (TG 23 of SSRC)
2. Load and Resistance Factor Design Approach for centrally compressed members (Washington University in St. Louis)
3. Influence of cyclic loading on column stability (University of Michigan)
4. Experimental investigations on local buckling of tubular columns (Lehigh University, Alcoa, Battelle, University of Wisconsin and others)

The research needs can be summarized as follows:

1. Complete the study of column strength parameters:
    a. Residual stresses in heavy shapes
    b. Residual stresses in welded built-up columns
    c. Gag-straightening effect
    d. Influence of out-of-straightness
    e. Complete the column selection table
2. Influence of end restraint on strength of crooked or damaged columns.
3. Can a single formula, with corrections, be developed as a design-tool substitute for multiple column curves?
4. Flexural (and torsional) buckling of angles, channels, and tee sections.
5. Can jumbo shapes be delivered to the site without cold-straightening?
6. What are the limiting conditions for which the beam-column behavior over-shadows the column strength reduction effect?

WEST EUROPE

STEEL

The refined assessment of column strength through multiple column curves dependent on manufacturing procedures makes it necessary to continously observe technological progress and to correct and update the selection charts. At the same time the theoretical basis of the column curves, which originally was a deterministic one, has to be adjusted to probabilistic safety concepts.

Studies are being conducted by:

Grimoult (Cometube, Paris) and Braham/Rondal (CRIF, Liege): Buckling of thin-walled hollow sections (240 tests).

R. Hawranek (Techn. Univ. Munich): Structural code optimization. Determination of safety elements (I-sections).

ALUMINUM

The reliability of the three column curves *a*, *b*, and *c* needs to be further investigated through numerical simulations and tests. Especially curve *c* needs additional experimental verification.

Tests are planned for welded columns of different alloy (Al-Mg and Al-Zn-Mg) and with unsymmetrical cross sections.

Calculations and tests are presently being carried out at the Universities of Liege (Belgium), Karlsruhe (German F.R.), and Novara and Naples (Italy).

EAST EUROPE

STEEL

In preparation for future Regional Recommendations, column curves corresponding to a complete ("level II") probabilistic design methods are considered, based on actual statistics, depending on the tolerable risk and variability of external loads as well. Refined analysis of flexural-torsional buckling, including the effect of residual stresses and effect of end restraints, is dealt with.

ALUMINUM

Analysis taking into account special cross sections and technology of aluminum alloy members is needed.