Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 48 (1985)

Rubrik: Session B: Codes and Eurocodes - an advanced platform for structural design

Nutzungsbedingungen

Die ETH-Bibliothek ist die Anbieterin der digitalisierten Zeitschriften auf E-Periodica. Sie besitzt keine Urheberrechte an den Zeitschriften und ist nicht verantwortlich für deren Inhalte. Die Rechte liegen in der Regel bei den Herausgebern beziehungsweise den externen Rechteinhabern. Das Veröffentlichen von Bildern in Print- und Online-Publikationen sowie auf Social Media-Kanälen oder Webseiten ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. Mehr erfahren

Conditions d'utilisation

L'ETH Library est le fournisseur des revues numérisées. Elle ne détient aucun droit d'auteur sur les revues et n'est pas responsable de leur contenu. En règle générale, les droits sont détenus par les éditeurs ou les détenteurs de droits externes. La reproduction d'images dans des publications imprimées ou en ligne ainsi que sur des canaux de médias sociaux ou des sites web n'est autorisée qu'avec l'accord préalable des détenteurs des droits. En savoir plus

Terms of use

The ETH Library is the provider of the digitised journals. It does not own any copyrights to the journals and is not responsible for their content. The rights usually lie with the publishers or the external rights holders. Publishing images in print and online publications, as well as on social media channels or websites, is only permitted with the prior consent of the rights holders. Find out more

Download PDF: 15.03.2026

ETH-Bibliothek Zürich, E-Periodica, https://www.e-periodica.ch

SESSION B

Codes and Eurocodes – an Advanced Platform for Structural Design

Codes et Eurocodes – une base avancée pour le projet de structures

Normen und Eurocodes – Fortschritte für Entwurf und Berechnung

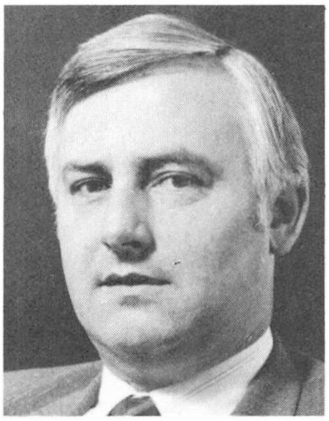
Leere Seite
Blank page
Page vide

Eurocode 3: Design Code for Steel Structures

Eurocode 3: Règlement de dimensionnement et de planification des constructions métalliques

Eurocode 3: Bemessungsnorm für Stahlbauten

Patrick J. DOWLING
Prof. of Steel Struct.
Imperial College
London, England



Patrick Dowling is Chairman of the Drafting Panel of EC 3. He is Editor of the Journal of Constructional Steel Research and Director of the consulting firm Chapman & Dowling Associates Ltd which drafted the Brazilian Steel Bridge Code.

SUMMARY
This paper outlines the technical contents of the draft EC 3 and suggests how it can be improved to match more closely the perceived needs of the member states of the European Community.

RÉSUMÉ
Cet article donne un aperçu du contenu technique de la version préliminaire de l'Eurocode 3 et suggère les améliorations à lui apporter pour mieux répondre aux besoins des Etats membres de la Communauté européenne.

ZUSAMMENFASSUNG
Dieser Vortrag umreisst den technischen Inhalt des Entwurfs des EC 3 und schlägt Verbesserungen vor, um die spürbaren Bedürfnisse der Mitglieder-Staaten der Europäischen Gemeinschaft besser befriedigen zu können.

1. INTRODUCTION

The draft European code for steel structures, Eurocode 3(1) has now been officially available for comment for some 12 months and it seems timely to review its scope, strength and shortcomings so as to ensure an informed debate which will lead to an even better draft than currently exists. Before doing so, however, it would be fair to record that the document issued from Brussels appears to have had a more favourable general reception than many of the sceptical members of our profession expected and, indeed, the members of the drafting panel dared expect. However, nobody, least of all the drafting panel, anticipated that the draft would be acceptable in all its detail, and for this reason it is very satisfying to note how constructive the comments received to date appear to be. If this paper elicits further useful comments from those who have not already commented it will have achieved its purpose.

2. AIMS, SCOPE AND FORMAT OF EC3

2.1 Aims of EC3

The general objectives of Eurocodes are treated by Verdiani (2). The specific aim of EC3 is to provide rules for the design, fabrication and erection of steel structures which produce safe and economical structures for the use and life for which they are intended. It was also the aim of the drafting panel to produce a code in a format with which steel designers are familiar; that is, an element (tension members, beams, columns etc) based format. This contrasts with the so-called action (tension, bending, compression etc) format which the concrete draft code, EC2 (3), adopts. Furthermore it was envisaged by the drafting panel that the code would be a stand-alone document containing both principles and operational rules which could be used without cross reference to EC1 (4) which contains the common unified rules for different types of construction and material.

2.2 Scope of EC3

The code is concerned with buildings and civil engineering structures and the basic principles apply to all types of steel structures. Because of the limitations imposed by time and finance the drafting panel considered that the most realistic and useful course of action was to write the code initially with buildings primarily, but not exclusively, in mind. Therefore while the treatment of buildings is relatively complete, supplementary information would be needed for anything other than the most commonly encountered short span steel bridge (a composite bridge which would be designed by cross referencing EC2 and EC4 (5)). Larger span bridges of orthogonally stiffened plated construction and other special structures such as nuclear power stations and marine platforms would need a significant amount of additional information to be included to ensure their satisfactory coverage.

It must be admitted that even in the case of buildings there is a serious omission in that cold formed sections and sheeting, much used in such construction, could not be incorporated within the document within the constrictions of time and finance. A point worthy of discussion would be whether such clauses should be included within the main draft code or added in a supplementary document. Indeed special structures such as bridges, towers and masts etc. might be best dealt with in the form of documents supplementary to EC3.

2.3 Format of EC3

The format adopted was to a certain extent influenced by EC1, the general format of which must be followed by all other Eurocodes. In an attempt to coordinate the formats of EC2 and EC3 those clauses which were identified by the panel as principles were marked by a vertical line in the margin, whereas those not so marked were classified as operational rules. As the text was not prepared with such classification of clauses in mind the exercise was only partly successful. It should be stated that there exists considerable reservation about the use of principles together with alternative opeational rules. This could create a dangerous loophole in safety requirements and appears to many to run counter to the concept of harmonisation.

The general format is given below and comprises three main sections. The first embracing Chapters 1 to 3 is concerned with basic principles, the second embraces Chapters 4, 5, 6, 8 and 9 and contains design information for both static and repeated loading and the third consisting of Chapter 7 deals with aspects of construction.

Preface
1 Introduction
2 Basis of Design
3 Materials
4 Serviceability Limit States
5 Ultimate Limit States
6 Connections
7 Fabrication and Erection
8 Test Loading
9 Fatigue

CHAPTER HEADINGS OF EC3

Chapters 5 and 6 are roughly of equal length and occupy between them approximately half of the code. Design is considered in the sequence of cross-sectional strength, component design and design of assemblages in Chapter 5, although an introductory piece does focus the designer's attention on the way the overall structure carries its loading. Chapter 6 follows a sequence of classification of joints and design methods for joints, bolted joints, welded joints, hybrid connections, and connections in cold formed sections.

3. MAJOR TECHNICAL FEATURES OF INTRODUCTORY CHAPTERS

3.1 Source Documents

A major source document for EC3 was the European Convention for Constructional Steelwork (ECCS) document, Recommendations for Steel Construction (6). This admirable document was not itself in a suitable form to be used as a draft Eurocode. It may be considered as a code drafter's code and is difficult to use directly by the designer in a similar manner to an equivalent national code. Other documents consulted included the draft German code (7), the Swiss code (8), the draft British code (9) and modified ECCS Recommendations prepared in Darmstadt (10). However, among the most important source documents were the technical papers produced by various committees of the ECCS on subjects varying from column buckling to fatigue. These committees drew on the best information available on an international basis.

3.2 Basic Design Philosophy and Materials

The units and notation used are in accordance with ISO standards. The most commonly occurring notation is listed at the outset but special notation is also defined within the text of particular clauses. Definitions of technical terms presented a difficulty because of the confusion over words which are used differently in separate member states. For example, the UK still has difficulty in accepting the term 'actions'.

Chapter 2 also covers calculation models and allows linear elastic models and non-linear materials models. In addition, either first order models based on initial geometry of the structure or second order (non-linear geometric) ones based on the deformed shape of the structure, may be applied. In most cases calculations are, of course, based on simple linear first order theory.

Materials are covered in Chapter 3. Non-alloy and low-alloy steels with yield strengths less than or equal to 450 N/mm^2, ultimate to yield strength ratios greater than 1.2, minimum strain to fracture greater or equal to 15% and fracture toughnesses corresponding to at least quality B of the International Institute of Welding Classification are covered.

4. MAJOR TECHNICAL FEATURES OF DESIGN CHAPTERS

4.1 Serviceability Limit States

A short Chapter 4 gives some suggested limiting values for deflections for floor and roof construction. Guidance is also given in relation to the dynamic response of floors. For example, the lowest natural frequency should not be lower than 3 cycles per second for regular floors whereas the floors of a gymnasium or a dance hall should not have a natural frequency lower than 5 cycles per second. Simple criteria which can be checked under static loading may be used to satisfy these values instead of a laborious estimate of their natural frequencies and accelerations.

4.2 Ultimate Limit State

4.2.1 Classification of Structures

A distinction is drawn between skeletal structures - which are braced and unbraced against sway. Guidance on the design of both of these classes is contained within Chapter 5 of the code.

4.2.2 Section Design

Section Design Depending on the degree of slenderness of cross-sectional elements the components may be designed in different ways, e.g. elastic theory or plastic hinge theory. Four classes of cross-section, are identified and the way the design basis is altered with slenderness is indicated. Similar classifications have been defined in the draft BS 5950 (10). There is scope here for improving the basis of the information on which these clauses are founded.

Interaction equations in terms of stress resultants are provided for cross-sections designed on an elastic or a plastic basis. For example, expressions are given for the reduction in plastic moment capacity of I-sections in the presence of both axial load and shear.

In the case of slender cross-sections the full cross-sectional area may be used provided stresses are within the limits of the buckling stress. This latter value is given by expressions which make due allowance for all parameters affecting the true inelastic buckling strength of the elements of the cross-section. Alternatively slender cross-sections can be checked using a reduced cross-section subjected to the full yield stress.

The only form of stiffened plate construction specifically covered is transversely stiffened webs in plate girders. The web buckling strength may be calculated using tension field theory. The load carrying capacity of the web is calculated by a simple expression which does not involve trial and error. Guidance on end panel and end post design is given as well as on the design of intermediate stiffeners. Preliminary independent studies suggest that the design methods given agree well with all known data.

4.2.3 Component Design

Components under compressive axial loading are designed using the so-called European Column Curves which were introduced in the ECCS Recommendations (7). Columns are assumed to have geometrical imperfections of 0.1% of their length and idealised residual stress distributions in different rolled and welded sections which are illustrated in the code. There are five buckling curves and the appropriate one is arrived at by means of a selection table depending on cross-section shape and axis of buckling. It is gratifying to note that this is one area where European harmonisation has already occurred in advance of EC3.

Members in bending which have unbraced compression flanges must be checked for lateral torsional buckling. (Strength and stiffness criteria for bracing design are codified). A simple approximate method treating the compression flange as a column consisting of flange and 1/6 of the area of the web simply supported and buckling between bracing points may be used to produce speedy conservative designs. Alternatively a more complex check depending on bending moment distributions over the length of the beam may be used. Guidance is, of course, given for situations where no such check is necessary. Preliminary studies indicate that the formulae contained in EC3 produce a mean fit to test data rather than a lower bound.

Beam-columns buckling in the load plane are designed using a simple interaction formula. Other interaction formulae covering beam-column buckling involving lateral torsional buckling and biaxial bending and compression are given. Considerable simplifications are possible in all cases by reducing the formulae to linear interaction ones. This is achieved at the expense of accuracy and inevitably involves some conservatism. The resulting formulae are a considerable improvement on those previously suggested in the ECCS Recommendations (7).

Chapter 2 is a key chapter as it covers the basic limit state design philosophy, the method of partial safety factors and the rules for their application. Ultimate, serviceability and fatigue limit states are identified.

The drafting panel attempted to simplify the application of the general procedures laid down in EC1 particularly with respect to the number of different partial safety factors and load combinations to be considered in design (11). To do this it was necessary to quantify the partial safety factors and so the simplified approach could not be contained within the main body of the text. Instead it is to be found in the Preface to the code where it is put forward not as an official CEC proposal but by the drafting panel to stimulate discussion.

4.2.4 System Design

The draft code, having dealt with components, goes on to consider assemblages of components, or systems. This section deals with braced and sway frames and includes particular systems such as truss girders and built-up columns. Braced frames are defined as ones having a bracing shear stiffness of at least 5 times the shear stiffness of the frame itself, in which case all horizontal actions can be considered to be transmitted by the bracing elements.

In the case of sway frames a simple criterion is given to show when first order (linear) analysis may be used to calculate stress resultants. This criterion will be satisfied by the majority of practical single storey rigid portal frame type buildings and indeed in most multistorey buildings also.

There may be a case for a fuller treatment of methods of design for assemblages, to include, for example, the "simple" design approach to braced frames used extensively in many countries.

4.3 Connection Design

4.3.1 Bolted Connections. At the outset of Chapter 6 joints are classified according to the requirements of the design approach, e.g. partial or full strength joints in a plastically designed frame.

The most up to date guidance on edge distances and pitches etc is given, followed by similar information on the strength of individual fasteners. These latter include dowel bolts, rivets, pin connections and high-strength bolts in slip-resistant connections.

Bolted connections loaded in shear are categorised as follows
Category A: Bearing type connections with black or non-preloaded high strength bolts.
Category B: Connections with preloaded high strength bolts with no slip at serviceability limit state.
Category C: As in B except no slip at ultimate limit state.

Joints loaded in tension may be in either of two categories
Category D: Connections with non-preloaded bolts.
Category E: Connections with preloaded high strength bolts.

Special attention is paid to the problems of long connections, splices and beam to column connections. The quantification of prying forces is made possible by a simple formula based on a collapse mechanism approach.

4.3.2 Welded Connections. As in the case of bolted connections, welded connections are classified and then guidance given on the strength of butt, fillet and plug welds. The strength of welded connections is covered including the special problems of long connections, splices, beam to column connections, and welded joints in hollow section lattice girders. This latter must represent the most comprehensive set of such rules available in any code. It might be argued indeed that too much has been provided in this case.

4.3.3 Other Connections. A detailed treatment is given for the design of column base plates, including holding-down bolts. There is also a section dealing with connections in thin walled elements, covering blind rivets, bolts, screws and powder actuated fasteners. This treatment of thin walled element connections is slightly anomalous as the design of thin walled elements and sheeting is not included in this first draft as mentioned earlier.

4.4 Test Loading

An important section of EC3 is that contained in Chapter 8 relating to test loading as this encourages innovation in design when a structure does not comply with the requirements of the other sections of the code by allowing the engineer to resort to experimental verification. This is, of course, only one type of testing envisaged. The four types referred to are

(i) acceptance tests
(ii) quality tests

(iii) prototype tests, and
(iv) destructive tests.

The first two are envisaged as non-destructive tests. The second two are ultimate load tests of which test type (iii) would be used when it is desired to substitute experimental tests for other design verifications for members or structures whose serial production is under consideration. The final set are of the type carried out in research laboratories to define computational verification methods for structures similar to those tested. Statistical methods suitable for analysing prototype tests are outlined within the code. To date few comments have been received on this section and additional observations would be very welcome.

4.5 Fatigue

The fatigue rules were based on work by the ECCS committee TC6. Fatigue checks are not usually necessary for buildings although parts such as crane gantry girders may have to be designed for fatigue. No fatigue check is necessary if the number of stress cycles, n, is such that

$$n \leq 2 \times 10^6 \left[\frac{36}{\Delta\sigma}\right]^3 \quad (\Delta\sigma \text{ in N/mm}^2)$$

where $\Delta\sigma$ is the stress range, or where $\Delta\sigma \leq 26$ N/mm².

The theoretical life is assumed to depend primarily on the applied stress range and the detail class which is applicable to the particular structural component or joint. Classifications of details with pictorial representations are given for four basic groups, i.e.
Group 1: non-welded details,
Group 2: welded details,
Group 3: bolted connections and other details,
Group 4: welded details in hollow sections.

The influence of mean stress level in non-welded details can be taken into account by modifying the stress ranges either by dividing the stress ranges by a "bonus factor" or by reducing the compression component of stress ranges by 0.6. The stress range-number of cycles ($\Delta\sigma_R$ - N_R) curves used for design are based on mean minus two standard deviations in relation to test results. The different details all have relationships with the same slope on a log-log plot and are identified by the stress range tolerable at 2×10^6 cycles.

5. MAJOR TECHNICAL FEATURES OF CHAPTER ON FABRICATION AND ERECTION

Chapter 7 represents a minimum specification for the standard of workmanship to ensure that the assumptions made in the design clauses are valid. The specification relates to predominantly statically loaded structures. When fatigue predominates more rigorous standards may be required.

Items covered include preparation of materials, clearance of holes for bolted connections, washers and nuts, tightening procedures for non-preloaded and preloaded bolts, fit of contact surfaces and inspection and checks.

In the case of welding reference is made to materials, welding procedures, and preheating. For welded structures subject to predominantly static loading two sets of requirements for weld tolerances are distinguished, i.e. "quality control level" and "fitness for purpose level". The first can justifiably be required of the manufacturer. The second is based mainly on strength considerations for statically loaded structures. For fatigue prone structures the former could be regarded as the fitness for purpose level. These levels are quantified with respect to cracks, lack of fusion, slag inclusions etc.

Acceptable non-destructive testing methods are listed, as are the tolerances on end preparations and root openings.

A set of tolerance levels based on recommendations of the appropriate ECCS committees is also included. (National standards or structural calculations based on maximum tolerances or indeed special requirements of a compulsory nature may take precedence over these rules.) None of these tolerances would be difficult to achieve by the normal competent European fabricator.

6 FUTURE DEVELOPMENTS

At the time of writing the CEC is organising the Editorial Groups to deal with the various comments on the draft EC3 which are received in Brussels. In the member states committees have been coordinating activities at a national level during the period for comment. Of particular interest are the many valuable in-depth appraisals and design studies which have been carried out on the draft. The results of these studies should be available in time for the Symposium. No doubt these will be invaluable in helping to improve EC3, and they are eagerly awaited.

In the meantime the writer offers his own reflections on some aspects of the work which remains to be done. These are as follows:

(1) It is essential to provide a commentary to the Code if it is to be applied correctly. Such a commentary should contain background information to the clauses, including references, and should also give guidance to the user on the intended application of the rules.

(2) Careful thought needs to be given to the way in which supplementary clauses needed for the design of particular structures should be handled. A series of supplementary documents, dealing with specialised applications, seems to merit serious consideration. Priority should be given to the provision of clauses on cold formed steel sections and sheeting to complete the information needed for the design of building structures. Such information has alrady been prepared by the ECCS and could be made available with little extra effort.

(3) It is necessary to keep the load factor format as simple as possible in the interests of safety. Some modification to the general format contained within the body of the Code is needed for the final draft. Several suggestions, including that in the preface, are available, so it should be possible to achieve a satisfactory solution.

(4) There is a certain unevenness in the depth of coverage given to the various topics within the Code. The Editorial Groups will need to address this problem so as to produce a more consistent coverage.

(5) Decisions are needed within the Commission on the status of the principles and operational rules contained within the Code. There is a strong body of opinion within the steel industry which is firmly opposed to the concept of alternative operational rules being substituted for those in the Code. Furthermore it is essential to clarify the type of user at which the Eurocode has been aimed. Does it include architects and builders with no formal qualifications?

(6) The editorial errors and inconsistencies identified within the draft will of course need to be corrected, as indeed will any error of substance, of which, hopefully, there are few. All comments received must be given careful

consideration.

(7) Conclusions: The members of the drafting panel are pleased that, on balance, the Code has been received favourably. Those involved in steering it to its final conclusion are determined that it will be the best Structural Steel Code available, so that designers will want to use it and not merely be forced to do so.

ACKNOWLEDGEMENTS The writer wishes to thank members of the drafting panel, Professors L Finzi, A Pousset, G Sedlacek and Messrs J Janss and J Stark who put so much effort into producing the draft whilst giving their time freely. He also wishes to thank Dr R E Hobbs, secretary to the panel, for his help in the final stages of the drafting and Mr N Wilhelm for his help at the earlier stages. Thanks are also due to Mrs S Wright for typing the manuscript. On behalf of all concerned he would finally like to pay tribute to the memory of Augusto Carpena, former Technical Secretary of the ECCS, who unfortunately did not live to see the fruits of our labours.

REFERENCES

1. EUROCODE 3 "Common Unified Code of Practice for Steel Structures", Report EUR 8849, Commission of the European Communities Directorate-General for Internal Market and Industrial Affairs, Brussels, Belgium, July 1983.

2. Verdiani, D. "Common European Code: Political and Economic Implications", Introductory Report, Symposium on Steel in Buildings, IABSE/ECCS, Luxembourg, 1985.

3. EUROCODE 2, "Common Unified Rules for Concrete Structures", Report EUR 8848, Directorate General for Internal Market and Industrial Affairs, CEC, Brussels, 1984.

4. EUROCODE 1, "Common Unified Rules for Different Types of Construction and Material", Report EUR 8847, Directorate General for Internal Market and Industrial Affairs, CEC, Brussels, 1984. Brussels, 1984 .

5. EUROCODE 4, "Common Unified Rules for Composite Steel and Concrete Structures", Draft Report Directorate General for Internal Market and Industrial Affairs, CEC, Brussels, 1984.

6. "Recommendations for Steel Construction", European Convention for Constructional Steelwork (ECCS), Brussels, Belgium, 1978.

7. Deutsche Normen, DIN 18800, Part 2 (Draft) Stability of Steel Construction, Buckling of Members and Systems of Members", 1979.

8. Swiss Norm, SiA 161, "Construction Metalliques", Zurich, 1979.

9. "The Use of Structural Steel in Buildings", Draft BS 5950, British Standards Institution, London, 1981.

10. Jungbluth, O. and Wilhelm, N. "Draft Proposal for EC3 (based on ECCS Recommendations)", Technische Hochschule, Darmstadt, May 1980.

11. Pousset, A.G. and Sedlacek, G. "The Application of Safety Concepts to Steel Structures". Paper 3, Proceedings of 3rd International ECCS Symposium, Structural Steelwork in the 80's, London, 1981.

Leere Seite
Blank page
Page vide

Eurocode – Ein Beitrag zur Verbesserung der Sicherheit, Wirtschaftlichkeit und des Wettbewerbs

Eurocode – a Tool for Safety, Economy and Competition

Eurocode – une contribution à la sécurité, l'économie et la compétitivité

Gerhard SEDLACEK
Professor Dr.-Ing.
RWTH Aachen
Aachen, BR Deutschland

Gerhard Sedlacek, geboren 1939, Studium TH Karlsruhe, Promotion TU Berlin, 8 Jahre Tätigkeit in der Stahlbauindustrie, zuletzt als Abteilungsleiter für Brückenbau, seit 1976 Professor für Stahlbau an der RWTH Aachen.

ZUSAMMENFASSUNG

In der Harmonisierung der technischen Regeln im Rahmen der Eurocode-Bearbeitung liegt die Chance, nicht nur eine Vereinheitlichung über die Ländergrenzen, sondern auch weitgehend über die Baustoffgrenzen zu erreichen, eine Entwicklung, die den Wettbewerb zwischen den Baustoffen und die Möglichkeiten des Stahlbaus und des Verbundbaus erweitern kann. Für die Wettbewerbsfähigkeit der Stahlkonstruktionen ist nicht allein entscheidend, Gewichte zu sparen, wichtig ist auch die Reduktion des Entwurfsaufwands durch einfache Regeln und die Herabsetzung der Herstellungs- und Montagekosten durch Entfeinerung der Konstruktion, die der Eurocode unterstützen will. Der Einfluss von Bemessungsnormen auf die Wirtschaftlichkeit von Bauweisen darf natürlich nicht überschätzt werden. Doch gehen die Anstrengungen bei der Überarbeitung des Eurocodes weiterhin dahin, zur Verbesserung der Sicherheit, Wirtschaftlichkeit und Wettbewerbsfähigkeit von Stahlkonstruktionen beizutragen.

SUMMARY

The drafting of a Eurocode provides the opportunity of harmonizing the technical rules not only across national barriers but also between different materials and ways of construction. This development may promote the competition between different materials and enlarge the market for steel and composite structures. For the competitiveness of steel structures the saving of steel weight is not the only answer; others are the reduction of the expenditure for design by simple comprehensible rules and of the manufacturing and assembling costs by moving away from too much refinement in the structural detailing, which will be supported by the Eurocode rules. Of course the influence of design codes to the economy of structures should not be overestimated; nevertheless the efforts for the redraft of the Eurocode aim at contributing to an improvement of this tool for safety, economy and competitiveness.

RÉSUMÉ

L'harmonisation des règles techniques dans le cadre de l'Eurocode offre non seulement la possibilité d'ouvrir les frontières, mais permet aussi de mettre en concurrence les différents matériaux et les différentes méthodes de construction. Ceci élargira le marché des structures métalliques et mixtes. Pour augmenter la compétitivité de l'acier, il ne faut pas seulement diminuer le tonnage des structures, mais aussi et surtout simplifier la conception, en utilisant des méthodes de dimensionnement simples, la fabrication et l'assemblage. Il ne faut cependant pas surestimer les avantages d'un règlement sur le coût de construction. Néanmoins, la nouvelle version de l'Eurocode aura plus que jamais pour but d'augmenter la sécurité, l'économie et la compétitivité des structures métalliques.

1. Allgemeines

Es ist erklärtes Ziel der Eurocodes /1/,

- "die Funktionsfähigkeit des gemeinsamen Marktes durch die Beseitigung von Hemmnissen als Folge unterschiedlicher Regelwerte zu verbessern,
- einheitliche technische Regeln für die wirksame Anwendung der Richtlinie des Rates Nr. 71/305 für die Koordination des Vergabeverfahrens bei öffentlichen Aufträgen bereitzustellen, die neben den nationalen Vorschriften angewandt werden können,
- die Wettbewerbsfähigkeit der europäischen Bauindustrie und der mit ihr verbundenen Industrien und Berufsgruppen in Ländern außerhalb der Gemeinschaft zu stärken,
- eine abgestimmte Grundlage für die geplanten einheitlichen Regeln für Baubedarfsartikel zu schaffen."

Wie kann dieses Ziel erreicht werden? Was sind die Erfordernisse, damit die Eurocodes für die Praxis attraktiv werden? Was sind die dringenden ersten Schritte auf dem Wege zur Harmonisierung?

Dazu sollen im folgenden einige Entwicklungen und Überlegungen im Hinblick auf den Stahlbau mitgeteilt werden.

2. Mehrdimensionale Harmonisierung

In der ersten Phase der Eurocode-Bearbeitung wurden die ersten Entwürfe für die Eurocodes 1, 2, 3, 4 und 8, die besonders den Stahlbau und den Betonbau betreffen, bearbeitet und in 1984 und 1985 veröffentlicht, Bild 1.

Die Entwürfe zeigen bereits die Tendenz, mit der sich die Harmonisierung der Regeln vollzieht. Diese erfolgt in zwei Dimensionen:

1. Die Harmonisierung der Regeln über die Ländergrenzen hinweg auf der Basis anerkannter, gleicher Sicherheitsprinzipien.

   Diese Harmonisierung ist weitgehend durch die Arbeit der internationalen technischen Verbände und Vereinigungen wie CEB, EKS, JCSS. etc., die sich in Model Codes und Empfehlungen niedergeschlagen hat, vorbereitet worden. Auf der Grundlage dieser Model Codes und Empfehlungen wurde versucht, in den jeweiligen Eurocodes praktische Bemessungsregeln zu schaffen, die die neuen Sicherheitsprinzipien /2/ erfüllen und die von den Mitgliedländern akzeptiert werden können. Die Bemessungsregeln betreffen z.Zt. hauptsächlich Fragen der Tragwerkswiderstände für vorgegebene Einwirkungen, Bild 2, die Einwirkungen selbst sind noch nicht definiert, so daß bei der Überprüfung und versuchsmäßigen Anwendung dieser Regeln in den Mitgliedsländern noch von den jeweils gültigen nationalen Lastvorschriften ausgegangen werden muß.

2. Die Harmonisierung der Regeln über die Baustoffe und Bauarten hinweg auf der Basis gleicher oder ähnlicher Berechnungs- und Bemessungsmodelle, Definitionen etc.

   Dieser Harmonisierungsschritt ist das Ergebnis der Einsicht, daß es sinnlos ist, gleiche oder ähnliche Sachverhalte im Betonbau, Stahlbau oder Verbundbau in verschiedener Weise zu regeln, daß es vielmehr für die Verbesserung der Anwendbarkeit der Regeln und der

Eurocode No 1:	Einheitliche Regeln für verschiedene Bauarten und Baustoffe
Eurocode No 2:	Betonbauwerke
Eurocode No 3:	Stahlbauwerke
Eurocode No 4:	Verbundkonstruktionen aus Stahl und Beton
Eurocode No 5:	Bauwerke aus Holz
Eurocode No 6:	Mauerwerksbauten
Eurocode No 7:	Gründungen
Eurocode No 8:	Bauwerke in Erdbebengebieten

Bild 1: Übersicht über die Eurocodes
bereits veröffentlicht (1984) /1/

Baustoffübergreifende Regeln und Bezeichnungen:

1. Einleitung: Zweck, Geltungsbereich, Bezeichnungen
2. Grundlagen für Entwurf und Bemessung:
   - Grundsätzliche Anforderungen
   - Bemessung nach Grenzzuständen
   - Einwirkungen und Kombinationen
   - Materialeigenschaften
   - Dauerhaftigkeit
   - Berechnungsmodelle und Versuchsmodelle
   - Gütesicherung

Baustoffspezifische Regeln und Bezeichnungen:

3. Werkstoffe: Baustähle, Verbindungsmittel
4. Grenzzustand der Gebrauchsfähigkeit: Durchbiegungen, dyn. Wirkungen
5. Grenzzustand der Tragfähigkeit:
   - Querschnittseigenschaften und Berechnungsmodelle
   - Querschnittsnachweise: Zug, Druck, Biegung, Beulen
   - Bauteilnachweise: Imperfektionen, Druckstab, Biegeträger
   - Systemnachweise: Imperfektionen, verschiebliche und unverschiebliche Systeme
6. Verbindungen:
   - Schrauben, Nieten, Bolzen
   - geschweißte Verbindungen
   - gemischte Verbindungen

   Stützenfüße

   Verbindung dünnwandiger Bauteile
7. Herstellung und Montage
   - Materialbehandlung: Verformung, Oberflächenbehandlung, Schneiden
   - Geschraubte Verbindungen: Lagern, Anziehen, Passung, Kontrollen
   - Geschweißte Verbindungen: Verfahren, Qualitätssicherung
   - Abmessungstoleranzen
8. Belastungsversuche: bei Abnahme, Prototypen, Qualitätsprüfungen
9. Grenzzustand der Betriebsfestigkeit (Ermüdung)
   - Anwendungsbereich und Grundlagen
   - Spektrum der Spannungswechsel
   - Betriebsfestigkeit
   - Sicherheitsnachweis

Bild 2: Inhalt des Eurocode 3 (Stahlbau) /1/
[- - -] für Harmonisierung zwischen verschiedenen Baustoffen vorgesehen

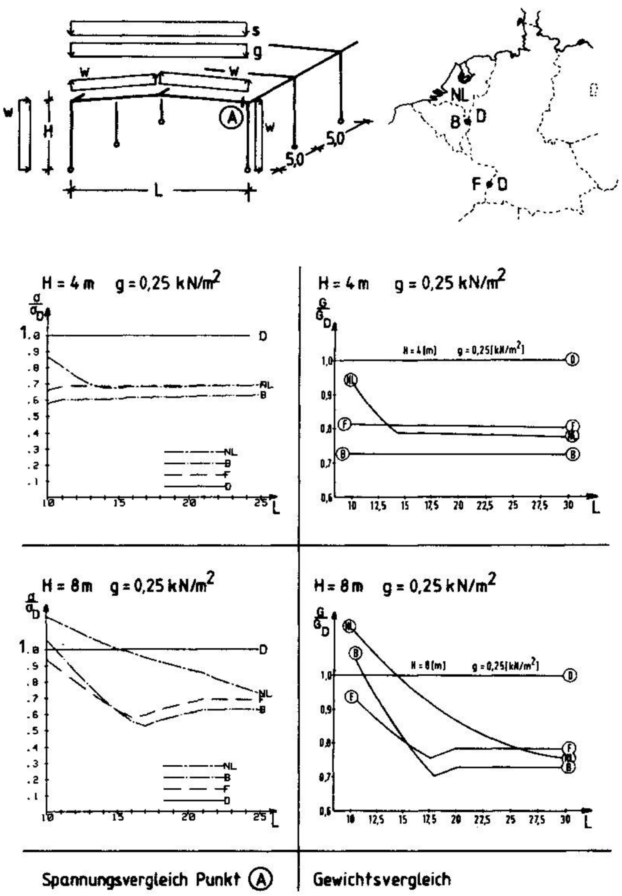


Bild 3: Auswirkung unterschiedlicher nationaler Lastnormen (B, D, F, NL) auf die Spannungen σ/σ_D und Gewichte G/G_D bezogen auf deutsche Vorschriften bei Hallenrahmen /3/.

besseren Öffnung des Wettbewerbs zwischen den Baustoffen darauf ankommt, ein Maximum an gleichartigen Regeln und ein Minimum an bauartspezifischen Regeln zu erzeugen. So wäre es für die Anwender einfacher, bei Projekten Alternativen mit verschiedenen Baustofflösungen zu bearbeiten und die wirtschaftlichsten Lösungen zu finden.

Die Harmonisierung der Regeln über die Baustoffe und Bauarten hinweg ist in den bisher veröffentlichten Eurocodes 2, 3, 4 und 8 zunächst nur in ersten Ansätzen gelungen, hauptsächlich im Inhaltsverzeichnis und bei den Prinzipien des Kapitels 2, Bild 2. Die Weiterarbeit auf diesem Gebiet wird die Schaffung harmonisierter praktischer Anwendungsregeln bedeuten und für die Verbesserung der Eurocodes besondere Bedeutung haben, vor allem im Interesse des Verbundbaus und der Mischbauweisen.

3. Notwendige Harmonisierungen zwischen den Baustoffen

3.1 Einwirkungen und Dauerhaftigkeit

Um die Eurocodes praktisch anwendbar zu machen, sind Lastdefinitionen erforderlich, da die Anwendung der neuen Regeln für die Tragwerkswiderstände eine angepaßte Definition der Einwirkungen verlangt.

Die sehr unterschiedlichen Auswirkung der derzeitig in einigen Mitgliedsländern gültigen nationalen Lastregelungen für Wind und Schnee auf die Spannungen und Gewichte von Stahlhallen geht aus Bild 3 hervor /3/.

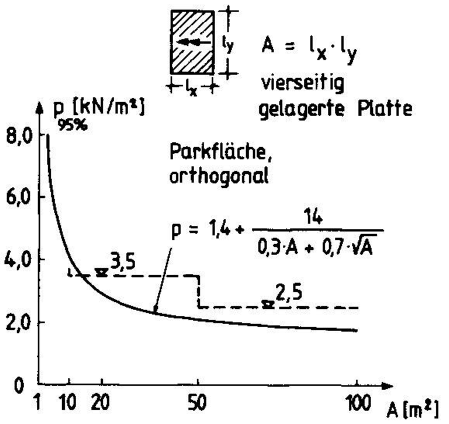


Bild 4: Vorschlag für die Lastmodellierung von Verkehrslasten auf Parkdecks /5/.

Bild 5: Vorschlag für eine Normzahlenreihe /6/

Normzahl $k_n = 10^{\frac{n-1}{40}}$	Rundungszahl Fehler ≤ 2,8 %
1	1
1,059 253	1,05
1,122 018	1,1
1,188 502	1,2
1,258 925	1,25
1,333 521	1,35
1,412 537	1,4
1,496 235	1,5
1,584 893	1,6
1,678 804	1,7

Eine Harmonisierung der Lasten beginnt bei der Modellierung der Lastenverteilung und -anordnung, erst danach kommt die Frage der Lastintensitäten. Ein gelungenes Vorbild für eine solche internationale Harmonisierung der Lastanordnung ist das UIC-Lastbild für Eisenbahnlasten /4/, ein Vorschlag für Parkhauslasten geht aus Bild 4 hervor /5/.

Für die Lösung der Harmonisierung der Lastintensitäten ist eine Methode erforderlich, die es gestattet, Vorschläge für ähnlich große Lastintensitäten als identisch zu erkennen und für unterschiedliche Lastintensitäten Grenzen für eine Einstufung festzulegen. Dazu wurde ein multiplikatives System in Form einer Normzahlreihe mit einer Regel für Auf- und Abrundung vorgeschlagen, /6/, zu der eine Übereinkunft über eine zweckmäßige Schrittweite getroffen werden muß, ein Beispiel ist in Bild 5 angegeben.

Mit einer solchen Normzahlreihe könnte ein Klassifizierungssystem geschaffen werden, das außer auf Lastintensitäten auch auf Widerstände und Sicherheitsfaktoren angewendet werden kann. Ein Beispiel für die vorteilhaf-

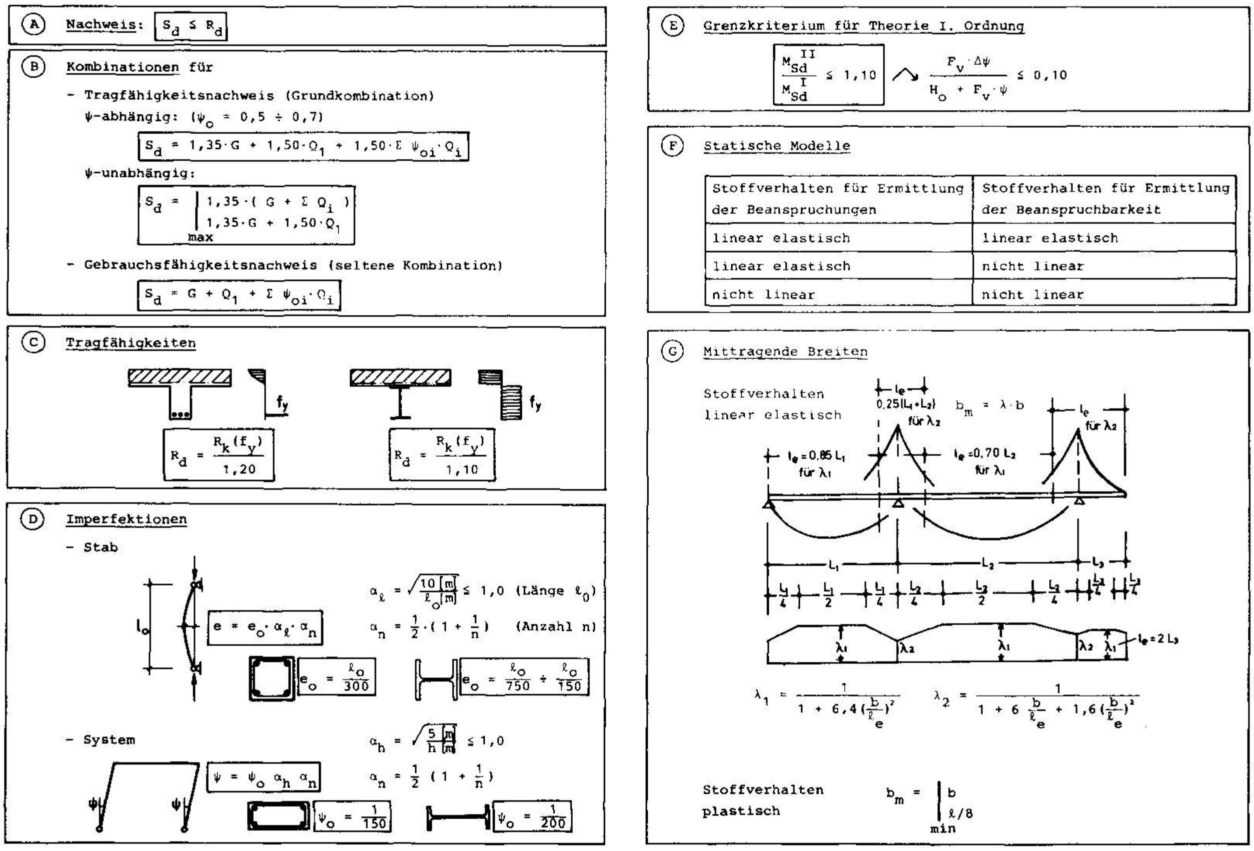
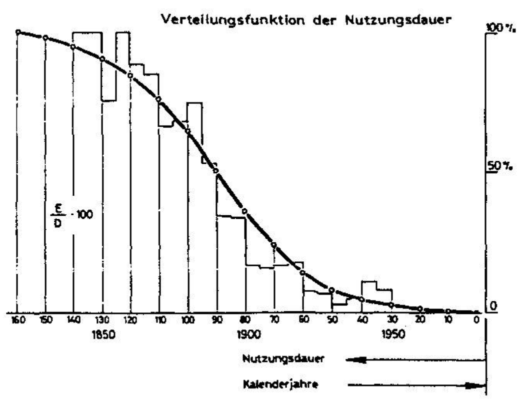


Bild 6: Vorschläge für die bauwesenübergreifende Harmonisierung von Bemessungsregeln (aus dem Arbeitspapier Jan. 1985 der CEB-EKS-Arbeitsgruppe)



te Anwendung im Eurocode 3 ist die Kerbfallklassifizierung für Ermüdungsnachweise in einem Normzahlraster.

Die Schrittweite des Klassifizierungssystems kann auch als ein Maßstab für Genauigkeitsanforderungen, z.B. den notwendigen Aufwand für Sicherheitsnachweise, interpretiert werden. Ein Beispiel hierfür ist die im Eurocode 3 angegebene Schranke für die Anwendung der Theorie 1. Ordnung, nämlich die Genauigkeitsschranke von 10%, die einen großen Anwendungsbereich für einfache lineare Nachweise zur Folge hat, Bild 6.

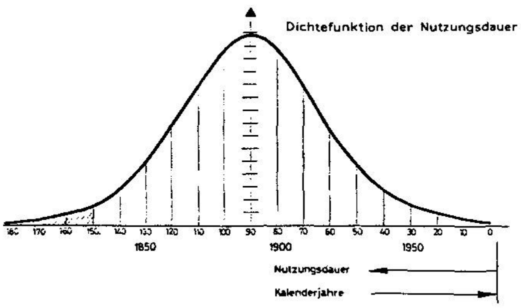


Bild 7: Dichte- und Verteilungsfunktion der Nutzungsdauer stählerner Eisenbahnbrücken in der BRD /7/

Bei der zukünftigen Festlegung der Lastintensitäten, vor allem für technische Lasten, sollte auch in längeren Zeiträumen gedacht, und allzu knappe Lastansätze sollten vermieden werden.

Dies hat seinen Grund in der bisherigen überschaubaren Erfahrung mit der Dauerhaftigkeit von Stahlbauten, die bezogen auf bisher gemessene Werte von Nutzungszeiten noch eine sehr junge Bauweise ist.

Am Beispiel der Einsatzzeiträume stählerner Eisenbahnbrücken in der Bundesrepublik, Bild 7, läßt sich nämlich zeigen, daß die Nutzungszeit der bisher ersetzten Konstruktionen weniger von Unterhaltungsmängeln oder mangelnder Betriebsfestigkeit, als vielmehr durch ungenügende Lastansätze bei der Bemessung bedingt war. Immerhin beträgt der bisher ermittelte Erwartungswert der Nutzungszeit der Eisenbahnbrücken über 90 Jahre /7/.

Die in den Eurocodes geforderte Dauerhaftigkeit der Konstruktionen wird also auch durch ausreichend vorausschauende Lastansätze für Tragfähigkeits- und Gebrauchsfähigkeitsnachweise und durch gleichwertige konstruktive Maßnahmen erzeugt, die den ständig sich ändernden Nutzungsbedingungen gerecht werden. Das sind Maßnahmen, die eine Veränderung der Nutzungsflächen, der Raumaufteilung, und Installation ermöglichen, sowie Maßnahmen zur Erhaltung und Verbesserung der Ansehnlichkeit, für die der Stahlbau hervorragende Voraussetzungen mitbringt.

Zu den ausreichend vorausschauenden Lastansätzen gehört auch eine vorausschauende Lastkombination, die Nutzungsänderungen zuläßt. Gerade im Wirtschaftsbau ist bei bestehenden Bauwerken mit einer laufenden Änderung und Verbesserung der Prozesse und damit der Lasten zu rechnen, was außer Lastvoraussagen auch Voraussagen für die Gleichzeitigkeit von Lastextremen, z.B. für Kranbahnen, die man für die Festlegung der Kombinationsfaktoren ψ benötigt, sehr erschwert.

Wo es also nicht einfach möglich ist, Veränderung von Lasten durch Bauteilverstärkungen aufzufangen, muß zum Zwecke der Dauerhaftigkeit von einfachen, konservativen ψ-unabhängigen Kombinationen ausgegangen werden, die in den Eurocodes, Bild 6, bereits vorgeschlagen sind.

3.2 Gebrauchsbedingungen

Daß die Konstruktionen für die gegebenen Nutzungsbedingungen tragsicher sind, wird im Wettbewerb zwischen verschiedenen Bauweisen als selbstverständlich vorausgesetzt, ein Mehr an Tragfähigkeit als gefordert wird in der Regel auch nicht honoriert. Anders sieht es mit den Gebrauchsbedingungen aus, sie sind in der Regel neben den Kosten Bewertungskriterium.

Erstaunlicherweise wird bis heute von den Möglichkeiten des zahlenmäßigen Nachweises der Gebrauchsfähigkeit, wie sie im Eurocode angegeben wird, kaum Gebrauch gemacht.

Ein Beispiel für die Durchführung eines Gebrauchsfähigkeitsnachweises ist in Bild 8 angegeben /8/.

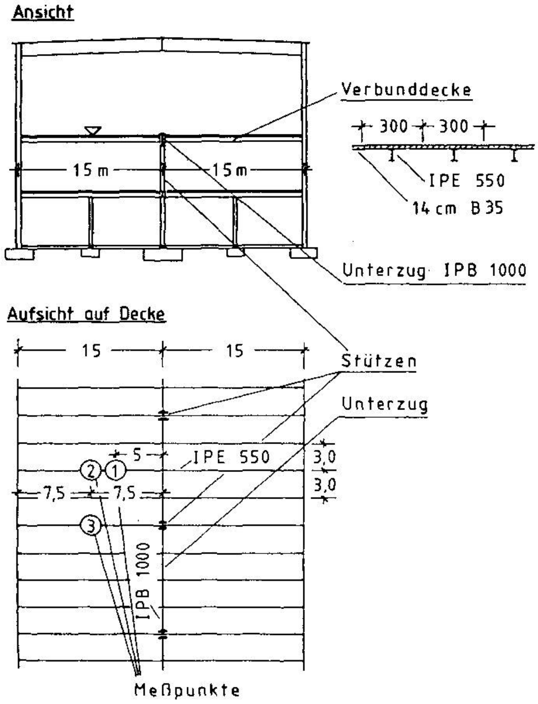


Bild 8: Abmessungen der Deckenkonstruktion einer Ausstellungshalle in Berlin /8/

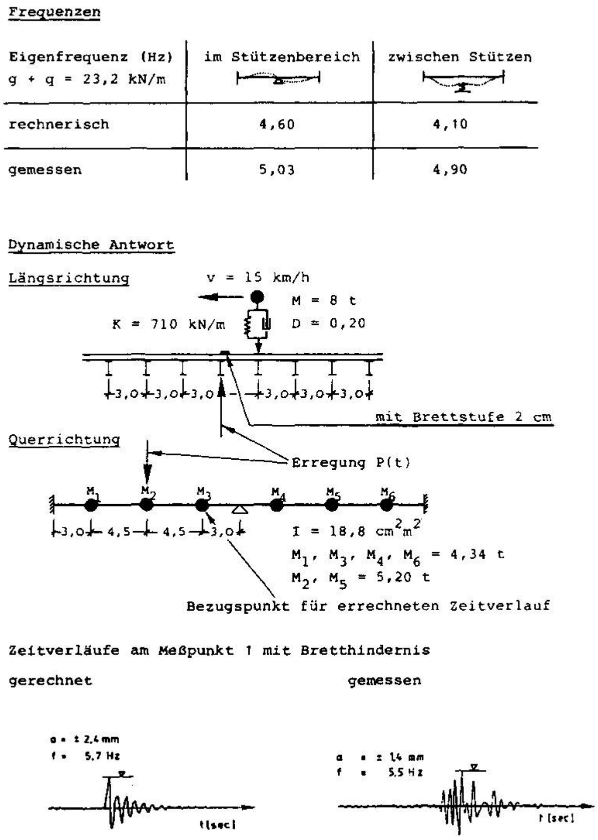


Bild 9: Eigenfrequenzen und Antworten der Deckenkonstruktion auf dyn. Einwirkungen /8/

Für eine Ausstellungshalle in Berlin war nachzuweisen, daß die Verbunddeckenkonstruktion schwingungsunempfindlicher als eine Konkurrenzkonstruktion ist. In Bild 9 sind die Bauwerksfrequenzen und Bauwerksantworten bei der Überfahrt von Gabelstaplern über Bretterhindernisse dargestellt, die die gewünschten Bedingungen sowohl rechnerisch vor der Ausführung und gemäß Messungen nach der Ausführung erfüllten. Im Eurocode 3 sind Belastungs- und Durchbiegungsbedingungen genannt, die es auch ohne solche komplizierte dynamische Simulation des Bauwerksverhaltens erlauben, die erforderlichen Gebrauchsbedingungen einzuhalten.

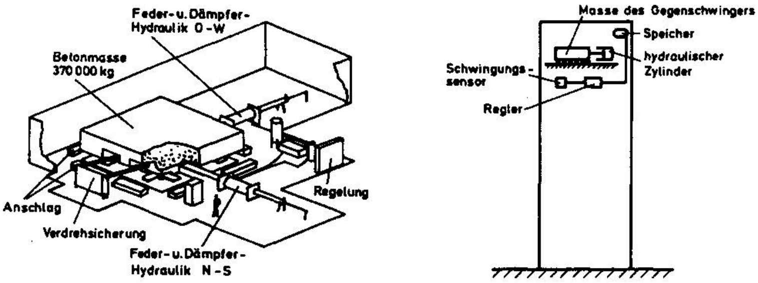


Bild 10: Ausrüstung eines Hochhauses mit passiven oder aktiven Schwingungsdämpfern /9/, /10/

Wenn die Gebrauchsbedingungen nicht von der für Tragsicherheit ausgelegten Konstruktion von Hause aus erfüllt werden, steht die Frage an, ob mit Konstruktionsänderung oder mit Regelung durch passive oder aktive Elemente wirtschaftlich verbessert werden kann. Bild 10 zeigt das Prinzip der Ausrüstung eines Hochhauses mit passiven oder aktiven Schwingungsdämpfern zur Verhinderung störender Schwingungen /9/, /10/.

3.3 Entfeinerung der rechnerischen Nachweise

Eine Entfeinerung der Stahlkonstruktionen ist durch Regeln für steifenlose Verbindungen, höhere Ausnutzung der Verbindungsmittel und Ausnutzung des überkritischen Beulverhaltens im Eurocode 3 eingeleitet. Damit werden wesentliche Herstellungs- und Montagekosten gespart.

Ein ebenso wichtiges Thema ist die Entfeinerung der rechnerischen Nachweise und die weitestmögliche Vereinheitlichung der Bemessungsregeln mit denen des Stahlbetonbaus, so daß auch bei solchen Ingenieuren, die im wesentlichen mit Betonkonstruktionen arbeiten, die Schwellenangst vor Alternativen in Stahl abgebaut wird. Die Vereinheitlichung ist Gegenstand der Arbeit einer Koordinierungsgruppe für die Eurocodes und einer Arbeitsgruppe CEB-EKS, und betrifft zunächst die Regeln, Definitionen und Bezeichungen für folgende Punkte:

- die Einwirkungen
- Die Kombinationsregeln und Sicherheitsbeiwerte, für die Einwirkungen,
- die statischen Tragwerksmodelle, die Imperfektions modelle für Theorie 1. und 2. Ordnung, und die Kriterien für deren Anwendung
- Festigkeitsmodelle für Tragfähigkeit, Gebrauchsfähigkeit und Betriebsfestigkeit,
- gemeinsame Regeln für die Auswertung von Versuchen.

Einige Vorschläge aus dieser Arbeit sind in Bild 6 dargestellt.

4. Zusammenfassung

In der Harmonisierung der technischen Regeln im Rahmen der Eurocode-Bearbeitung liegt die Chance, nicht nur eine Vereinheitlichung über die Ländergrenzen, sondern auch weitgehend über die Baustoffgrenzen zu erreichen, eine Entwicklung, die den Wettbewerb zwischen den Baustoffen und die Möglichkeiten des Stahlbaus und des Verbundbaus erweitern kann.

Für die Wettbewerbsfähigkeit der Stahlkonstruktionen ist nicht allein entscheidend, Gewichte zu sparen, wichtig ist auch die Reduktion des Entwurfsaufwands durch einfache Regeln und die Herabsetzung der Herstellungs- und Montagekosten durch Entfeinerung der Konstruktion, die der Eurocode unterstützen will.

Die Nachweismöglichkeit der Gebrauchstüchtigekeit liefert ein zusätzliches Sicherheitselement und Gütemerkmal und sollte zusätzlich zum Tragsicherheitsnachweis beachtet werden.

Der Einfluß von Bemessungsnormen auf die Wirtschaftlichkeit von Bauweisen darf natürlich nicht überschätzt werden. Doch gehen die Anstrengungen bei der Überarbeitung des Eurocodes weiterhin dahin, zur Verbesserung der Sicherheit, Wirtschaftlichkeit und Wettbewerbsfähigkeit von Stahlkonstruktionen beizutragen.

5. Literatur

/1/: Eurocode 3, Entwurf Nov. 1983, Kommission der Europäischen Gemeinschaften.

/2/: Eurocode 1, Entwurf 1984, Kommission der Europäischen Gemeinschaften.

/3/: Hemmert-Halswick, A.: Untersuchungen von Lastfallkombinationen für die Bemessungen von Stahlbauten im Hinblick auf Vereinfachung, Forschungsbericht IfBt.

/4/: Prommersberger, G. und Siebke, H.: Das Belastungsbild UIC-71, die neue Bemessungsgrundlage für den Eisenbahnbrückenbau. Eisenbahntechnische Rundschau 25 (1976) Heft 1/2 S. 33-40

/5/: König, G., Hosser, D., Maurer, R.: Superimposed loads in car parkings. Note January 1985, CIB Commission W 81 - Actions on Structures.

/6/: Siebke, H.: Runden und Ordnen mit definierter Genauigkeit. Veröffentlichung in Vorbereitung.

/7/: Siebke, H.: Monetäre Bewertung langlebiger Investionsgüter, abgeleitet für Eisenbahnbrücken. Veröffentlichung in Vorbereitung.

/8/: Sedlacek, G.: Aspekte der Gebrauchstüchtigkeit von Stahlbauten. Stahlbau 10/1984 S. 305-310

/9/: Ruscheweyh, H.: Dynamische Windwirkung an Bauwerken. Bauverlag GmbH, 1982

/10/: Petersen, N.R.: Design of large scale tuned mass dampers. Reprint 3578 ASCE 1976 (ASCE Convention & Exposition, Boston, April 2-6, 1979)

Eurocode 4: Composite Steel and Concrete Structures

Eurocode 4: constructions mixtes acier-béton

Eurocode 4: Bemessungsnorm für Stahl- und Betonverbundkonstruktionen

R.P. JOHNSON
Professor of Civil Engineering
University of Warwick
Coventry, United Kingdom

Roger Johnson graduated at Cambridge, then worked in industry before becoming a university teacher. He has been active in research on composite structures since 1960. He has been contributing since 1968 to the preparation of codes of practice, and was Chairman of the Drafting Panel for Eurocode 4.

SUMMARY

The first draft of Eurocode 4 was written in 1984, to be consistent with the available drafts of Eurocodes 2 and 3. An account is given of its scope, principal contents and main innovations, and its relationship to Eurocodes 2 and 3 and to the Model Code for composite structures that was prepared between 1972 and 1981.

RÉSUMÉ

En 1984, la version préliminaire de l'Eurocode 4 fut écrite conformément aux versions préliminaires des Eurocodes 2 et 3. L'exposé donne un aperçu du contenu, les innovations essentielles, les relations avec les Eurocodes 2 et 3, les relations avec le Code Modèle des Constructions Mixtes, mis au point entre 1972 et 1981.

ZUSAMMENFASSUNG

Der erste Entwurf des EC 4 wurde 1984, im Einklang mit den bereits vorhandenen Entwürfen EC 2 und EC 3, aufgesetzt. Er enthält eine Darstellung der Anwendungsbereiche, der Hauptinhalte und der Hauptneuerungen und zeigt die Zusammenhänge zu EC 2 und EC 3 und zu den zwischen 1972 und 1981 erarbeiteten Modellrichtlinien für Stahlverbundkonstruktionen.

1. INTRODUCTION

1.1 Background

The first draft of Eurocode 4 was completed in English in October 1984, as a 150-page document of 12 chapters. It will be published by the Commission of the European Communities (CEC) within the next few months in English, French, German, and Dutch. It is less self-contained than other Eurocodes, as it had to be consistent with the August 1983 draft of Eurocode 2 for concrete structures (EC2), and the July 1983 draft of Eurocode 3 for steel structures (EC3), and it can be used only in conjunction with them.

The work on EC4 exposed a few difficulties in using EC2 and EC3, which will be considered by their drafting panels at the same time as the comments on them now being prepared in the ten Member States.

Eurocode 4 is based on the Recommendations for Composite Structures of 1981 [1], referred to here as the Model Code; but there are substantial differences of scope and presentation, because much of the Model Code was written in the mid-1970's before EC2 and EC3. Three of the members of the drafting panel for EC4 worked on the Model Code, one of these also on EC3; and the fourth contributed to EC1 and EC2. The technical content of EC4 is thus firmly based on the expertise of the much larger groups that drafted these preceding documents.

Eurocode 4 is the first one to take full advantage of the work of the Coordinating Committee for Eurocodes, which was set up in 1982 with Mr. G. Breitschaft as Chairman. Working from draft Eurocode 1 ("Common unified rules for different types of construction and material"), this group produced in 1983 a model Preface and model Chapters 1 and 2, for use in Eurocodes 2 to 8. Appropriate versions of them were incorporated in EC2 and EC3 at a late stage of drafting. This established the mutual consistency that was essential for the preparation of EC4.

1.2 Scope of EC4

The Code is concerned with composite floor slabs, beams and columns, and with framed structures that have composite beams or columns or both. The scope is thus wider than that of the Model Code, in respect of frames. Profiled steel sheet is not included in EC3, so reference had to be made also to ECCS recommendations. Propped and unpropped construction and lightweight concrete are included.

The scope does not include encased composite beams, piles for foundations, cable-stayed or box-girder bridges, or composite plates. The range of plate-girder bridge decks for which EC4 can be used is likely to be limited by omissions from EC3 (e.g., longitudinally stiffened members), rather than from EC4.

2. THE PREFACE

This provides background information on the harmonisation process, and the presentation, calibration, and future use of EC4.

2.1 Harmonisation

As load specifications are not harmonised between Member States, no partial safety factors (γ_F) nor coefficients for representative values (ψ) are given in EC4. Each State will decide its own. The values that were envisaged during drafting are given in the Preface, for discussion only.

Harmonisation of materials specifications is more advanced, so partial safety factors for materials (γ_M) are given as a basis for trial designs. They are enclosed in boxes to indicate that that they are not an official proposal from CEC. For consistency, all the values in EC4, except those for shear connectors, are copied from EC2 or EC3, even though it is doubtful whether the values of γ_M at the ultimate limit state for structural steel (1.0) and reinforcing steel (1.15) are in harmony.

The uncertainties that are represented in Eurocode 1 by γ_d, and in the British Bridge Code (BS 5400) by γ_{f3}, are split into two parts, represented by γ_{Rd} and γ_{Sd}. These are merged with the partial coefficients γ_m and γ_f as follows:

$$\gamma_M = \gamma_m \gamma_{Rd} \quad , \quad \gamma_F = \gamma_f \gamma_{Sd} \; ,$$

to give γ_M and γ_F, which are the values given in the Preface or (in boxes) in the clauses.

2.2 Presentation

This was based on that used in EC3, except that Ultimate Limit State precedes Serviceability Limit State, as it does in EC2.

Principles are distinguished from Rules for Application by a vertical line in the margin, as in EC3, as this was thought to be simpler than the separate clause numbers used in EC2.

The division of the text into Principles and Rules gave particular difficulty, which has led to a review by the Steering Committee for Eurocodes of the definitions of Principles and of Rules given in Model Chapter 1.

Careful thought was given to the extent to which material relevant to EC4 but given in EC2 or EC3 (or, for much of the Preface and Chapter 2, in both) should appear again. It was decided to repeat data that were concise, important, and frequently used (e.g., the list of the Grades of concrete and associated values of modulus of elasticity); but otherwise to give cross references to clauses in EC2 or EC3, sometimes supplemented by a summary of their content.

3. CHAPTERS 1 TO 3 (INTRODUCTION, BASIS FOR DESIGN, AND MATERIALS)

3.1 Symbols

Reference is made mainly to ISO document IS 3898 [2]. Minor deviations are usually for consistency with EC2 or EC3 or with established practice for profiled steel sheeting. Innovation was necessary for the subscripts meaning "steel". In EC4, "a" is used for structural steel, "s" for reinforcing steel, and "ps" for prestressing steel: except that the yield strength of structural steel is f_y, not f_a.

The fact that symbols gave little difficulty is encouraging evidence of the recent progress towards a European consensus on notation.

3.2 Terminology

The writer had difficulty in breaking the habits of half a lifetime in learning to use "strength" only for materials, and "resistance" for cross sections or members; and in adopting the comprehensive word "action", with associated "effects". Having done so, he urges fellow English-thinkers to persevere with "Eurospeak", not to reject it. We owe it to our colleagues from the other Member States, who so often have to use or translate from our language, to remove some of its ambiguities; and also to allot precise and unfamiliar meanings to certain words, to match existing words in other languages.

3.3 Classification of actions

Unlike EC2, a distinction is made in EC4 between prestressing by tendons and by imposed deformation of the structure; the latter is named "precamber". Definitions are also given in EC4 for the "primary" and the "secondary" effects of shrinkage of concrete, prestressing, and non-uniform temperature.

3.4 Properties of concrete

For analysis of composite structures, it is often appropriate to use simpler (i.e., less accurate) values than are given in EC2. Two approximate values for the long-term free shrinkage of normal-density concrete are given in EC4; and three different levels of approximation for modular ratios – the simplest being to use $n = 15$ for all purposes.

As in EC2, properties of concrete are related to the 28-day cylinder strength, f_c. When considering data related to cube strengths, f_{cu}, it was assumed that for normal-density concrete, $f_c = 0.80\ f_{cu}$. The ratio can in fact be as low as 0.67 or as high as 0.85. Values related to compressive strength are given in Ref. [3].

4. ULTIMATE LIMIT STATE

4.1 Beams and frames

A definition is given of *critical cross sections*, at which design action effects in bending and shear should be compared with resistances. Design longitudinal shear between adjacent pairs of such sections has to be compared with shear resistance.

No attempt is made to define procedures appropriate for a particular type of structure, such as "buildings" or "bridges". Design methods are related to the slenderness of the steel flanges and webs at the sections considered. As in EC3, there are four classes: 1, plastic; 2, compact; 3, semi-compact; and 4, slender.

The limiting slendernesses for class 3 are as in EC3, but those for classes 1 and 2 are lower by between 10% and 25%. This is to take account of two differences from all-steel members:

- the limit to plastic curvature set by crushing of concrete, and
- the greater redistribution of moments from internal supports to midspan that often must occur before the sagging moment of resistance is developed.

For the analysis of structural systems, two methods are given in detail: elastic analysis, for all classes, and plastic hinge analysis, for members or frames where all the critical sections are in class 1. Elastic-plastic analysis is also allowed, but no details are given.

Frames with some composite members are classified according to the type of the joints between beams and columns. Methods of analysis are given, using references to EC2 or EC3 as appropriate, for frames where the joints are "simple" or "rigid", as in steel construction, or "monolithic", as in concrete construction. Semi-rigid joints are not excluded, but no methods of analysis are given.

The primary effects of shrinkage and of non-uniform temperature may be neglected, and the secondary effects have to be considered only in exposed structures, such as bridges, with cross sections in class 3 or class 4.

Deck or floor slabs have to be designed for local loads to EC2, but these loads can be ignored in the design of composite members.

In the analysis of structures, and of sections in classes 1 and 2, shear lag is allowed for by a simple but conservative rule for effective breadth of a flange. A more accurate but complex method for calculating effective breadth is provided for analysis of cross sections in classes 3 and 4.

4.1.1 Resistance of cross sections

Two idealisations are given in EC2 for the stress-strain curve for concrete in compression. They both lead to complex calculations when used for the bending resistance of a composite beam, in which the neutral axis is often within or close to the steel top flange. In EC4 a third, less conservative, method is given. This is to use a rectangular stress block at $0.85\ f_c/\gamma_M$ extending to the neutral axis, with a recommended value of 1.5 for γ_M.

Parametric studies have shown that for composite beams, the difference from the methods of EC2 is negligible; but for composite columns the bending resistance of a cross section is over-estimated. A correction factor of 0.90 is given in EC4 in the design method for columns that is based on Ref [4], to allow for this and other approximations.

In EC4, the preceding method is used for the calculation of the bending resistance M_{pc} of beam cross sections in class 1 or 2. The resistance to combined bending moment M and vertical shear V is as shown in Fig. 1, in which $R_f d_f$ is the resistance of the member when the steel web is excluded, and $V_{p\ell}$ is the resistance to shear found in accordance with EC3. For cross sections in classes 3 and 4, elastic analysis is used.

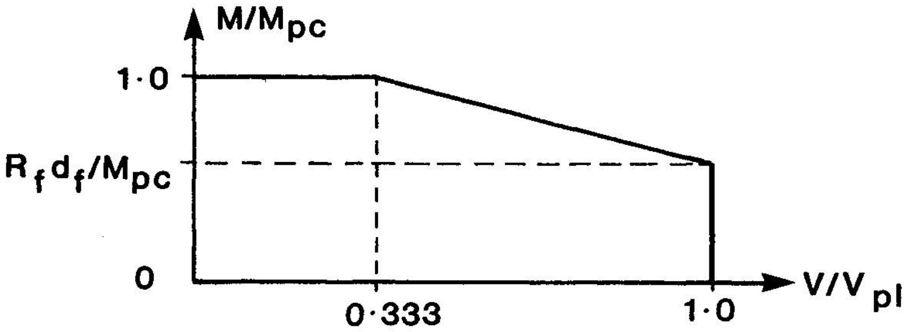


Fig. 1. Resistance of cross section in combined bending and shear

4.2 Composite columns

The cross sections considered in detail are fully or partly encased I or H sections, and concrete-filled tubes of rectangular or circular section. General recommendations are given for design of column lengths that are either in frames or loaded through bearings.

Two simplified design methods are given for column lengths of uniform doubly symmetric cross section, including those in unbraced frames or with biaxial bending, subjected to known axial load and end moments. Effective lengths are found by the methods of EC3.

The first method, developed at the University of Bochum [4], makes use of the European column curves, as given in EC3, and of an ingenious simple method of calculating for a cross section the interaction curve between axial load and bending moment, given in an Appendix to EC4.

The second method, developed at the University of Warwick [5], is quicker but of reduced scope, in that transverse loads applied within the column length and some other situations are excluded. For given slendernesses and bending moments, the axial resistance is calculated directly from simple formulae, that were deduced from parametric studies and checked against test data.

4.2.1 Effective flexural stiffness of cross sections

For analysis of frames, the uncracked flexural stiffness is used, with allowance for creep by means of an effective modulus, as for beams. For the design of a column length, the de-stabilising effects of creep are allowed for by a different effective modulus. It is taken as

$$E_{ce} = 600\ f_{ck}\ (1 - N_{Gd}/2N_d)$$

for slender columns loaded at low eccentricity, and otherwise as $600f_{ck}$, where

f_{ck} is the characteristic cylinder strength of the concrete,

N_d is the design axial load, and

N_{Gd} is the part of this load that is permanent.

4.2.2 Longitudinal shear in columns

A method is given for the design of shear connection in columns. None is needed if the mean shear stress at the steel-concrete interface is less than 0.6 N/mm² for encased columns and 0.4 N/mm² for filled tubes.

5. SERVICEABILITY LIMIT STATES

5.1 Analysis of the structure, and stresses

The number of structures for which re-analysis is needed for the serviceabiliy limit state has been reduced to a minimum. When it is necessary, all cross sections in class 1 or 2 are deemed to be in class 3, and the methods of analysis, both for the structure and for sections, are essentially as for the ultimate limit state.

No limiting stresses are specified, but approximate and conservative methods are given, for allowing for the influence of any local yielding of steel at internal supports of continuous members on deflections and the widths of cracks.

5.2 Limit states

The limits for deflections in buildings are as in EC3. For bridges, guidance is given on when deflections should be calculated, but no limits are given.

For cracking of concrete, the intention is to use the limits and methods given in EC2; but some revision and extension of these is necessary.

Guidance is given on the avoidance of excessive vertical and horizontal vibration in footbridges. For vibration of floors in buildings, reference is made to EC3.

6. CHAPTERS 6 TO 12

6.1 Chapter 6, Shear connection in beams

Provision is made for the use of studs with or without heads; bar, tee, channel and horseshoe connectors; anchors made from round bars; and friction-grip bolts; and also for the use of headed studs with profiled steel sheeting. Design is for the ultimate limit state. Between critical sections in classes 1 or 2, connectors may be uniformly spaced, and partial shear connection down to 50% is allowed in simple and in continuous beams. Spacing related to the design shear flow may be used when the critical sections are in any class.

The resistance in shear of a headed stud is influenced by the size of the weld collar [6]; but the existing design resistances, given in the Model Code and essentially reproduced in EC4, are based mainly on test data where no details of the collars were reported. Current practice of not requiring studs to have collars has been followed in EC4; but alternative higher design resistances have been given for studs that have collars of at least a specified size, for use when other conditions relating to the strength of the concrete and the stud material are satisfied.

The rules for transverse reinforcement near shear connectors have been aligned with those in EC2, with the result that one of the two design equations is more conservative than in the Model Code, even though the latter is supported by the evidence from tests on composite members.

6.2 Chapter 7, Composite floors with profiled steel sheet

This chapter covers simply-supported and continuous floors spanning parallel to the ribs of the sheeting, with predominantly static loading, slabs at least 90 mm thick, and sheeting of depth not exceeding 80 mm. The design methods are similar to those of BS 5950: Part 4 [7], which were based on the well-known work of ECCS Committee 11, which in turn used work from Professor Badoux' group at Lausanne and from Lehigh University.

The strength of the composite floor slab in longitudinal shear is given by a semi-empirical equation including factors m and k which have to be obtained from tests (which are specified in Chapter 12). Details are given of design for vertical and punching shear, and of serviceability checks.

6.3 Chapters 8 to 12

Chapter 8 covers reinforced or precast slabs used either as permanent formwork, or as complete floors spanning between steel beams and composite with them. It is generally as in EC2 and the Model Code.

Chapter 9, Fatigue, refers to EC3, which gives data on fatigue resistance of shear connectors.

Chapter 10, Prestressing and precambering of beams, is based on the Model Code. It gives details of checks at the serviceability limit state.

Chapter 11, Workmanship and construction, is also based on the Model Code, but with new material on profiled steel sheeting and on through-sheet welding of studs.
Chapter 12, Testing, includes the push test for studs as given in the Model Code and in a British Code (CP117) of 1965. The slabs are small and are reinforced with mild steel bars that are not fully anchored. Subsequent research has shown [6] that although this test may be expected to give relevant results for small connectors set in strong concrete, it is over-conservative for other situations, in which splitting would occur in the narrow test slabs, but not in an unhaunched T-beam. There is need for a better test to be standardised; but there are so many relevant variables that its calibration would be a costly process.

The probability-based methods for deducing design resistance from the results of tests are consistent with those in EC3, but differ from the existing practices for push tests and for shear tests on composite slabs.

7. CLOSURE

It is hoped that the preceding outline of the scope and principal features of Eurocode 4 will be of interest, at least to those who use it for trial designs and calibration studies, and make comments on it.

As noted earlier the main division in EC4 is between design methods applicable only to members with critical sections in class 1 or class 2, and more restrictive methods of general applicability. The limiting slendernesses given for class 2 are thus the most significant numbers in the code. They are based on rather limited evidence.

There is need for research on these limits, for if they could be relaxed, the cost of composite structures for buildings could in many instances be reduced.

7.1 Acknowledgement

The writer is deeply grateful to the other members of the Drafting Panel for EC4, Mr. H. Mathieu, Professor K. Roik, and Mr. J. Stark, for their substantial contributions to the work.

REFERENCES

1. Joint Committee IABSE-CEB-FIP-CECM, Model Code for composite structures, Construction Press, 1981.

2. IS 3898, Bases for design of structures - Notations - General symbols, 1976; Addendum 1, 1982; Addendum 2, International Organisation for Standardisation, 1983.

3. IS 3893, Concrete - Classification by compressive strength, International Organisation for Standardisation, 1977.

4. ROIK, K. and BERGMANN, R., Zur Traglastberechnung von Verbundstützen, Stahlbau, 1, 8-15, 1982.

5. JOHNSON, R.P. and SMITH, D.G.E., A simple design method for composite columns, Structural Engineer, 58A, 85-93, March 1980.

6. JOHNSON, R.P. and OEHLERS, D.J., Analysis and design for longitudinal shear in composite T-beams, Proc. Instn Civil Engrs, Part 2, 71, 989-1021, December 1981.

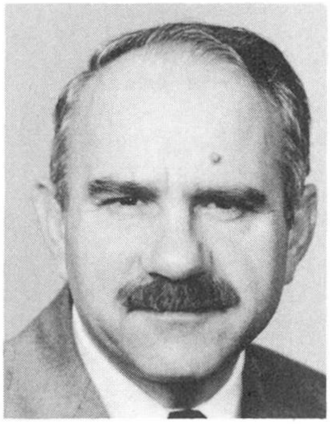
7. BS 5950, Structural use of steelwork in building; Part 4, Code of practice for design of floors with profiled steel sheeting, British Standards Institution, 1982.

Design of Steel Structures with Load and Resistance Factor Design Specifications

Conception des structures métalliques et spécifications aux états limites

Neue Grenzzustands-Normen für Stahlbauten

Theodore V. GALAMBOS
Prof. of Civil Engineering
University of Minnesota
Minneapolis, MN, USA



Ted Galambos, born 1929, received his Ph. D. from Lehigh University. He has been engaged in teaching and research on steel structures for the past 30 years.

Ivan M. VIEST
Consulting Engineer
Bethlehem, PA, USA

Ivan Viest, a native of Czechoslovakia, received his Ph. D. from the University of Illinois. His experience includes design, teaching, research, and market development. He has played a leading role in the development of Load and Resistance Factor Design Specifications in the USA.

SUMMARY
This paper describes the background and the major provisions of the new load and resistance factor design specifications for steel building structures.

RÉSUMÉ
Cet article décrit les bases et les points essentiels des nouvelles spécifications aux états limites pour les structures en acier.

ZUSAMMENFASSUNG
Dieser Artikel beschreibt die Grundlagen und die Hauptrichtlinien der neuen Grenzzustands-Normen für Stahlbauten.

1. INTRODUCTION

The Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, issued and maintained by the American Institute of Steel Construcion (AISC) has been the pre-eminent steel design code in the United States of America since 1923. While it strictly applies for the design of building structures fabricated from hot-rolled steel components, it is clearly the model and the benchmark for all other steel design specifications used in the USA, and for codes in many other countries.

The current (1985) official version of the AISC Specification has two parts: Part 1 is nominally in an Allowable Stress Design (ASD) format, while Part 2 (Plastic Design) is in a Limit States Design (LSD) format. However, Part 1 is, in fact, also based on LSD principles which are disguised, sometimes quite transparently, to accommodate the traditional ASD format.

Research on the development of a separate and new LSD specification for steel building structures was initiated by the American Iron and Steel Institute (AISI) in 1969. The results of this work are culminated in the new (AISC) Load and Resistance Factor Design specification (LRFD) which will be issued in 1985. This paper describes this LRFD specification.

2. HISTORY OF THE DEVELOPMENT OF THE LRFD SPECIFICATION

The LRFD specification is based on LSD principles and the partial factors were determined by First-Order Second Moment (FOSM) probabilistic theory. It is entirely independent of the currently official (1985) AISC Specification, and decision theory was used as the basis for organizing its contents. It also contains a number of up-dated design procedures, notably for the design of beams, beam-columns and composite beams.

The initial research effort (1969-1978) culminated in a preliminary draft of the LRFD specification (1) and in a series of eight papers in the Sept. 1978 Journal of the Structural Division of ASCE which detailed the methods and the statistical data used to develop the criteria. This preliminary draft was not only the result of the effort of a number of specialists in probablistic design, decision theory logic, and structural steel behavior, but it also contained practical input from extensive trial designs performed in two consulting offices. In 1978 it was evident that 1) LRFD was a possible and desirable design method, but 2) the AISC LRFD specification could not function without a common basis for load and load factor determination which is shared by the structural design codes for all structural materials, especially reinforced concrete, masonry, aluminum and wood. As a consequence, the action shifted to the American National Standards Institute (ANSI) Committee on Building Code Requirements for Minimum Design Loads in Buildings which issued its load standard (2) in 1982. This standard contains the new load factors and load combinations to go with any building material, and its basis is the FOSM method, as detailed in a number of publications (3,4,5). The load factors were determined to give a target reliability index $\beta=3.0$ for gravity load, and $\beta=2.5$ for load combinations involving wind loads. The various material groups then were provided with a method to determine resistance factors to go with their respective design codes such that roughly the same reliability was achieved as implied by the load factors.

In the meantime various subcommittees of the AISC Specification Committee undertook to rework the LRFD draft, and the LRFD specification was approved in principle in 1981. A draft for review and trial was issued in 1983, and currently work is concluding on the final specification, including a commentary and a design manual.

3. FEATURES OF THE NEW SPECIFICATION

3.1 FORMAT

The format of the design inequality contains partial factors for load effects, γ, and for resistances, ϕ:

$$\phi_k R_{nk} \geq \gamma_D Q_D + \gamma_{Ei} Q_{ni} + \sum_{j=i}^{m} \gamma_{Ej}^{*} Q_{nj} \tag{1}$$

where the subscript n denotes nominal (code specified) values of the resistance R and the load effects Q, the subscript k denotes different applicable resistance limit states, the subscript D means dead load, and the subscript E defines the time-varying load effects due to occupancy, wind, snow, earthquake, etc. loads. The load factors for these latter quantities count on one of the time-varying loads to have its maximum life-time value (a 50 year life is assumed) while the others take on their arbitrary-point-in-time- values (3,5). Following is an array of some load combinations to illustrate the combinatorial process:

$$1.4D \tag{2a}$$

$$1.2D + 1.6L + 0.5S \tag{2b}$$

$$1.2D + 1.6S + 0.5L \text{ OR } 0.8W \tag{2c}$$

$$1.2D + 1.3W + 0.5L + 0.5S \tag{2d}$$

D,L,S and W are dead, live, snow and wind load effects, respectively. The comparison with the corresponding load factors in the 1978 ECCS Code are shown in Table 1.

TABLE 1: LOAD FACTORS

A.I.S.C. CODE,1985	E.C.C.S. CODE,1978
1.33D+1.78L+.56S	1.3D+1.5L+.75S
1.33D+1.78S+.56L	1.3D+1.5S+.75L
1.33D+1.44W+.56(L+S)	1.3D+1.5W+.75(L+S)

3.2 ORGANIZATION OF THE LRFD SPECIFICATIONS

The specification provides the nominal resistance R_{nk} and the resistance factors ϕ_k for the various limit states appropriate to each type of member or connection. The resistance factors were determined by FOSM and they provide reliability index values from about 2.5 to 3.0 for members and 4.0 to 5.0 for connection under dead plus live loads.

The LRFD specification is organised as follows:

A. General Provisions.
B. Design Requirements.
C. Frames and Other Structures.
D. Tension Members.
E. Columns and Other Compression Members.
F. Beams and Other Flexural Members.
G. Plate Girders With Tension Field Action.
H. Members under Combined Stress, Torsion and Combined Stress and Torsion.
I. Composite Members.
J. Connections, Joints and Fasteners.
K. Strength Design Considerations.
L. Serviceability Design Considerations.
M. Fabrication, Erection and Quality Control.

4. DESCRIPTION OF SELECTED PROVISIONS.

Following is a brief discussion of selected provsisions of the LRFD specification so that comparisons can be made with the corresponding rules in the EUROCODE 3.

4.1 COLUMNS

One column formula is used in the LRFD specification for all types of compression members. The basis for it is an initial out-of-straightness of 1/1500 of the column height; end-restraint which results in an elastic effective length factor of 0.96 is assumed. The basic column curve (non-dimensional slenderness ratio -versus-critical stress ratio) is compared to the European column curve b in Fig. 1. The LRFD curve is seen to be above the European curve, especially in the intermediate slenderness range. It should be realized, however, that the resistance factors and the load factors are not the same for the two codes. With a resistance factor $\phi = 0.85$ and 1.0, respectively, for the AISC and the ECCS codes, it turns out that in most instances the latter is more liberal. For example, a permissible design force P of 670 kN and 706 kN is obtained by the AISC and SSRC method, respectively, for the following column:

W12 x 65, $F_y = 248$ MPa, Length = 4.57 m

$P_{dead} = P$, $P_{live} = P$, $P_{wind} = P/2$

4.2 PLASTIC DESIGN

Plastic design is permitted when the reference slenderness λ is less than or equal to a maximum plastic slenderness λpd. The comparative values of λpd are shown in Table 2 for the limits of flange buckling, web buckling and lateral-torsional buckling. The criteria are more liberal for the LRFD specification.

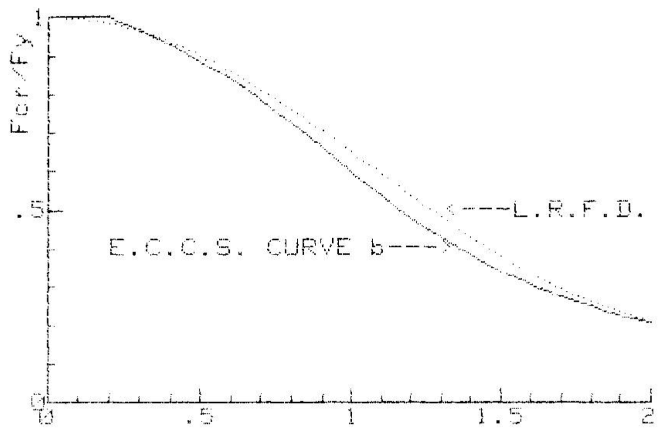


Fig.1: COMPARISON OF COLUMN CURVES

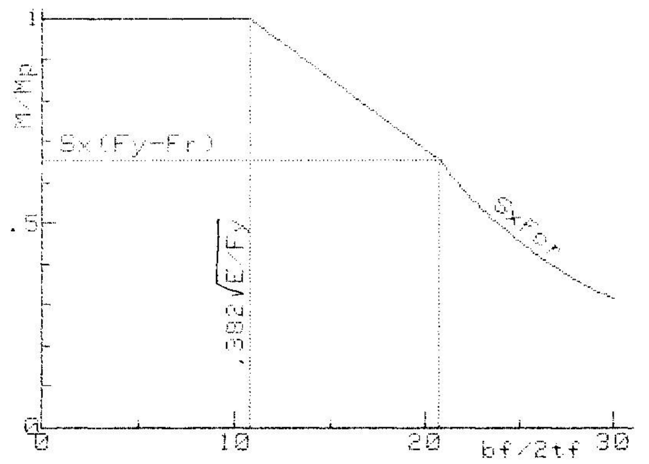


FIG. 2: LIMIT STATE FLANGE BUCKLING

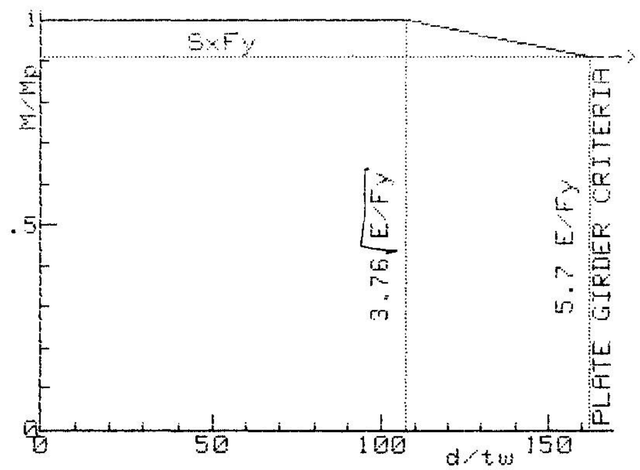


FIG. 3: LIMIT STATE WEB BUCKLING

TABLE 2: SLENDERNESS LIMITS: PLASTIC DESIGN

RATIO	LRFD LIMIT	ECCS LIMIT
(bf/tf)/2	$0.382\sqrt{E/Fy}$	$0.300\sqrt{E/Fy}$
d/tw	$3.76\sqrt{E/Fy}$	$2.4\sqrt{E/Fy}$
Lb/ry *	0.048E/FY=40 **	$1.37\sqrt{E/FY}=40$

* UNIFORM BENDING

** FOR E/Fy=850

4.3 BEAM DESIGN

When the forces in the structure are determined by elastic analysis, then beam design is governed by the limit states of flange local buckling (FLB), web local buckling (WLB), and lateral-torsional buckling (LTD) as shown in Figs. 2, 3 and 4, respectively. The limit state moment is the plastic moment, M_p, up to a slenderness λ_p, then a straight line transition in the range of inelastic buckling, and finally either the elastic buckling moment M_E (for FLB and LTB) or the moment including post-buckling strength (for WLB) controls for slender beams. For the limits FLB and LTB the change from inelastic to elastic buckling occurs when the stress equals the yield stress less a residual stress of 69MPa. The equivalent moment factor C_b is used to account for reduced yielding along the length of the member if the moment field is non-uniform, as shown in the construction of Fig. 4. The LRFD criteria tend to be more liberal, as seen by the example in Fig. 5.

4.4 BEAM-COLUMNS

Beam-columns are designed by a new set of interaction equations, illustrated here for major axis bending of wide-flange members:

$$\frac{P_u}{\phi_c P_{cr}} + \frac{8}{9}\frac{M_u}{\phi_b M_{cr}} <= 1 \qquad \text{for } \frac{P_u}{\phi_c P_{cr}} >= 0.2 \quad (3a)$$

$$\frac{1}{2}\frac{P_u}{\phi_c P_{cr}} + \frac{M_u}{\phi_b M_{cr}} <= 1 \qquad \text{for } \frac{P_u}{\phi_c P_{cr}} < 0.2 \quad (3b)$$

This interaction relationship is shown in Fig. 6. The symbols in the formulas are defined as follows:

1) $\phi_c = 0.85$ and $\phi_b = 0.9$ are the resistance factors for columns and beams, respectively.
2) P_{cr} is the critical column buckling load, including the effective length factor accounting for frame buckling.
3) M_{cr} is the critical beam moment-capacity of the member.

$$M_u = B_1 M_{nt} + M_t \qquad (4)$$

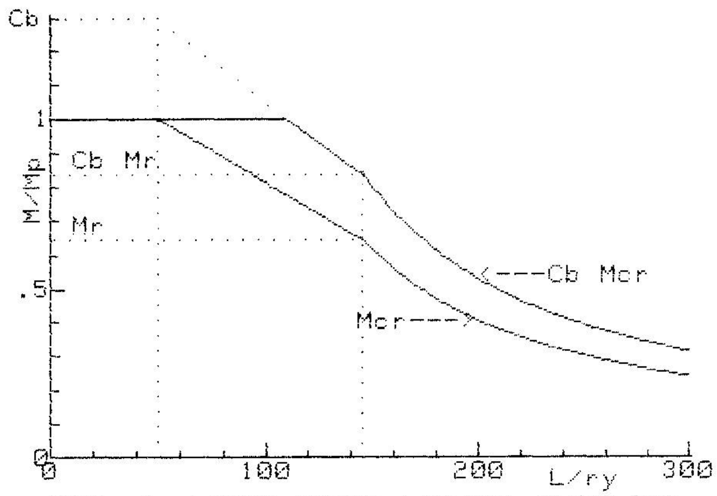


FIG. 4: LIMIT STATE LATERAL BUCKLING

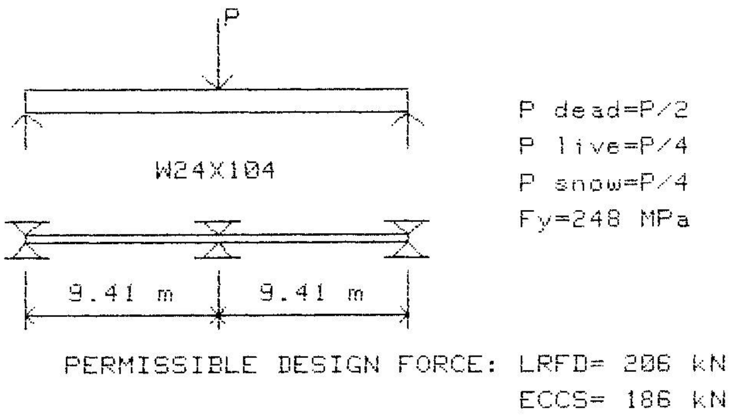


FIG. 5: BEAM DESIGN EXAMPLE

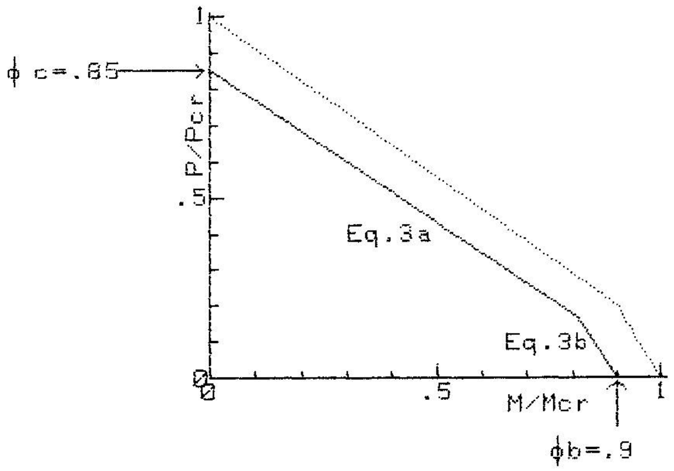


FIG. 6: INTERACTION CURVES FOR BEAM-COLUMNS

where

$$B_1 = \frac{C_m}{1-P_u/P_e} >= 1 \qquad (5)$$

P_e is the elastic buckling load in the plane of the moment assuming no side-sway buckling and C_m is the equivalent moment factor. The moment M_{nt} is determined as if the joints were restrained from translation, and M_t is the moment due to lateral and restraining forces, including second order story translations. Approximate formulas are provided for the amplification of M_t if it is computed by first-order methods, but an actual second-order analysis is recommended.

5. CONCLUSION

This paper described the salient features of the new Load and Resistance Factor Design specifications of the AISC. Comparisons are made with the corresponding European code.

REFERENCES

1. AISI Proposed Criteria for Load and Resistance Factor Design of Steel Building Structures, AISI Bulletin No. 27, Jan. 1978.

2. AMERICAN NATIONAL STANDARD Minimum Design Loads for Buildings and Other Structures, ANSI A58.1-1982.

3. ELLINGWOOD, B.; GALAMBOS, T.V.; MACGREGOR, J.G.; CORNELL, C.A.; Development of a Probability-based Load Criterion for American National Standard A58. Washington, D.C.: US Dept. of Commerce, National Bureau of Standards, June 1980, NBS SP 577.

4. GALAMBOS, T.V.; ELLINGWOOD, B.; MACGREGOR, J.G.; CORNELL, C.A.; Probability-based Load Criteria - Assessment of Current Design Practice. J. Structural Div., ASCE. 108(ST5): 959-977; May 1982.

5. ELLINGWOOD, B.; MACGREGOR, J.G.; GALAMBOS, T.V.; CORNELL, C.A. Probability-based Load Criteria - Load Factors and Load Combinations. J. Structual Div., ASCE. 108(ST5): 978-997; May 1982.

Standards Representation and Processing

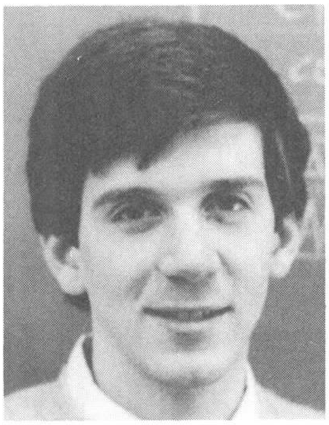
Représentation et traitement des normes

Richtlinien für Entwurfsnormen und deren Anwendungen

Steven J. FENVES
Professor
Carnegie-Mellon University
Pittsburgh, PA, USA

Dr. Steven Fenves received his Ph.D. from the Univ. Illinois, where he taught until 1972. His research activities center around computer-aided engineering, with an emphasis on expert systems, standards processing and databases. At Carnegie-Mellon Univ., he has been Head of the Dep. of Civil Engineering and Dir., Design Research Center.

James H. GARRETT, Jr.
Research Assistant
Carnegie-Mellon University
Pittsburgh, PA, USA



Jim Garrett received both his B.S.C.E. and M.S.C.E. from Carnegie-Mellon Univ. and is currently pursuing a Ph.D. in civil engineering at C-MU under the direction of Dr. Fenves. His Ph.D. thesis topic involves the development and implementation of a prototypical interface between design standards and CAD programs.

SUMMARY
This paper summarizes the formal representation of standards that has evolved over 15 years, starting with the 1969 AISC Specification for the Design of Steel for Buildings. The paper concentrates on recent developments in two types of standards processing, namely, for standards formulation and for standards use in a computer-aided design environment. An important aspect of the developments described is the incorporation into standards processing of the rapidly developing area of knowledge-based expert systems.

RÉSUMÉ
Cet article résume les travaux des 15 dernières années dans le domaine de la représentation formelle des normes. L'article met l'accent sur les développements récents pour deux types de traitement des normes, pour la formulation de celles-ci et pour leur utilisation dans un environnement CAO. L'utilisation des systèmes experts pour le traitement des normes constitue l'un des aspects importants des développements décrits dans cet article.

ZUSAMMENFASSUNG
Dieser Beitrag fasst die fünfzehnjährige Entwicklung formaler Richtlinien, beginnend mit der AISC Entwurfsnorm im Stahlbau aus dem Jahre 1969, zusammen. Der Vortrag bezieht sich besonders auf neue Entwicklungen, die zwei Gebiete der Bearbeitung und Anwendung von Richtlinien betreffen: Formulierung von Richtlinien und deren Gebrauch im computergestützten Entwurf. Ein besonders wichtiger Gesichtspunkt in der hier beschriebenen Entwicklung ist die Anwendung von Expertensystemen (Knowledge Based Expert Systems).

1. INTRODUCTION

Standards and design specifications are the primary means whereby analytical, experimental and empirical results on adequate structural behavior are transmitted to practitioners. They also become legal documents defining, or at least limiting, adequate design practices. To serve this dual purpose, standards must be complete, clear, unambiguous and usable with ease and confidence. The term "use" increasingly means incorporation of standards provisions into computer programs for design, detailing, and conformance checking.

Formal representation of standards has two purposes. First, the representation can be used by standards development organizations to insure that a proposed or modified standard possesses the requisite properties of completeness, clarity, lack of ambiguity, and ease of use before the standard is issued or promulgated. Second, the representation can serve as the starting point in the development of computer programs incorporating specification provisions, largely eliminating individual interpretations and reducing much of the expense of modifying application programs when standards are updated, or when a program must process differing standards of multiple jurisdictions.

2. THE REPRESENTATIONAL MODEL OF A STANDARD

The current model for representing a standard has evolved over many years. Many modifications were made during this evolution. Research prior to 1979 was presented in [3] and [4]. The next section describes the current state of the model which includes the contributions of Rasdorf and Fenves [15], Harris and Wright [11], and Howard and Fenves [13].

2.1. Current State of Model

The model for representing design standards consists of four components:

1. **data items** representing every variable that is found in the standard;
2. **decision tables** representing the logic used to determine the values of data items;
3. **information networks** representing the precedence relations among the data items; and
4. **organizational system** representing the organization (arrangement and scope) of the standard.

These four components are discussed in more detail in the next four subsections.

2.1.1. Data Items

Data items, or datums, represent all the variables occurring in the standard. The total set of data items, plus the relations between them, are intended to contain all the substantive information in the standard. Data items are one of four value types: numeric, a member of an enumerated set, "satisfied" or "violated", or boolean.

2.1.2. Decision Tables

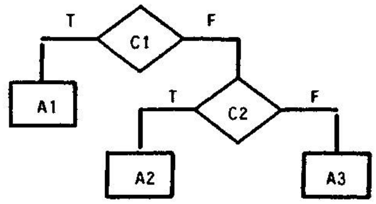
A decision table is an orderly presentation of the reasoning leading to the assignment of a value to a data item of the standard. Each table is responsible for producing a value for one, and only one, data item. A decision table is composed of conditions, actions, and rules, as shown in Figure 2-1a. A condition is a boolean expression that can only have the values of "TRUE" or "FALSE". As used in the model, an action can only be an assignment of a value to a datum. A rule is a prescription of a certain action, given that a specific combination of values of the conditions exists. A complete decision table for a datum gives an exhaustive set of rules for evaluating that datum.

Decision trees can be automatically generated from decision tables. They represent the same

	RULE 1	RULE 2	RULE 3
CONDITION 1 : F1(B,C)	T	F	F
CONDITION 2 : F2(C,D)	F	T	F
CONDITION 3 : F3(B,D)	F	F	T
ACTION 1 : A = E4(D)	X		
ACTION 2 : A = E5(C)		X	
ACTION 3 : A = E6(B)			X

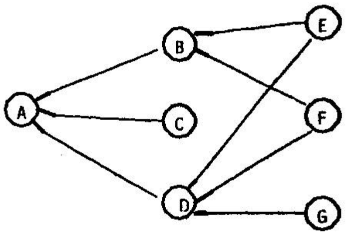
E1 - E6 are arbitrary expressions

a) DECISION TABLE FOR DATA ITEM "A"



b) DECISION TREE FOR DATA ITEM "A"

ASSUME: 1. E, F, and G are Basic Data Items
2. B depends on E and F
3. D depends on E, F, and G



c) ASSOCIATED INFORMATION NETWORK

Figure 2-1: Sample Decision Table, Decision Tree, and Information Network

information as found in the decision tables, but they also prescribe the most efficient order in which to evaluate conditions and choose an appropriate action. A sample decision tree generated from the decision table in Figure 2-1a is shown in Figure 2-1b.

2.1.3. Information Network

The information network is used to represent the precedence relationships between the data items of the standard. The network is composed of nodes, each node representing one data item in the standard, with each node connected to its ingredients and dependents by directional branches. The *ingredients* of a data item "A" are the data items needed to evaluate "A". The *dependents* of a data item "A" are the data items that have "A" as an ingredient.

An example of an information network is shown in Figure 2-1c. The left part of the information network for datum "A" shows that data items "B", "C", and "D" are needed to calculate the value of "A". However, the network does not show how the value for "A" is computed from its ingredients. It is the decision table that contains the precise relationship between the ingredients of a datum and the datum itself.

The information network can be used to:

- trace all the datums that have any possible influence on the datum in question , known as the **global ingredience** of that datum; and
- to trace all the datums that might be influenced by the datum in question, known as the **global dependence** of that datum.

Datums can also be classified according to their interrelations with other datums: *basic* datums have no ingredients, *derived* datums have both ingredients and dependents, and *terminal* datums have no dependents.

Rasdorf and Fenves [15] provided a method for expanding a datum within an information network into its subnetwork using the datum's associated decision table, as well as a method for compressing a subnetwork into a single higher-level datum. These methods provide a flexible interface between the information network and the decision tables.

2.1.4. Organizational System

Design standards are collections of provisions (terminal datums) which a design must satisfy. Each provision evaluates to either "satisfied" or "violated". The satisfaction of a provision depends on the satisfaction of one or more requirements, where a requirement has the form: <object> has <performance attribute>. The requirement's value is determined by a set of criteria contained within the standard.

The user of a standard must determine which provision(s) of a standard applies for the situation at hand. If this task is to be performed with the least amount of frustration and error, the standard must be organized and systematically defined such that this information is readily available. Decision tables only provide the logic within the provisions, and the information network only represents the precedence relations between the data items. An organization of the provisions is needed for accessing the individual provisions of the standard.

The standard provisions are placed into an organizational system in the following manner. First, the scope of the standard, i.e., the subjects covered, is decomposed into several trees of classifiers. A minimal classification system requires two trees of classifiers to represent the information contained in a standard: one for the physical entities covered and one for the performance attributes required for the entities [2]. In such a minimal system, all classifiers at the same level in the same tree must be:

1. mutually exclusive to guarantee that the selection of a provision according to associated classifiers is unique, and
2. collectively exhaustive to insure that every provision can be associated with its appropriate classifiers.

A more flexible classification system, developed by Harris and Wright [11], uses several independent classification areas, called *fields*. Each field is subdivided into a hierarchy of classifiers, called a *facet*. Partial facets from two fields are shown in Figure 2-2. The two fields are physical entities and limit states, respectively, where limit state refers to the failure mode of the physical entity. After the classifier facets have been built, appropriate classifiers from each facet are associated with each provision providing a medium for accessing the provisions.

Figure 2-2: Partial Facets of Classifiers taken from [11]

Howard and Fenves [13] developed a Generic Classification system for use in comparing standards. Their system uses facets of generic classifiers in order to classify all of the provisions in the standards being compared. They also developed a condensed information network which only shows the relationships between basic data items and requirements.

2.2. SASE - An Implementation of the Model

SASE (Standards Analysis, Synthesis, and Expression) is an integrated set of procedures being developed by the National Bureau of Standards (NBS) for computer aided formulation and expression of standards using the representational model presented [6]. The intended uses of SASE are to:

- *analyze* existing standards for completeness and uniqueness of its provisions, for connected-

ness and proper cross-referencing between provisions, and for completeness and clarity of the standards organizational system;
- *synthesize* new standards or modify existing standards by: providing a repository of information about the standard, allowing the standard's developer to analyze each provision as it is built, and allowing the standard's developer to analyze the partially completed standard; and
- aid in the *expression* of standards by providing several alternate frameworks for expressing the information content of the standard, such as outlines and indexes.

An expert system front-end is being planned for SASE to assist in the analysis of decision tables. The system will accept the conditions, actions and initially known condition entries, and then assist in completing the table based on dependencies among conditions. If this effort is successful, other expert systems may be attached, e.g., to assist in the ordering of information networks and outlines for clear and convenient expression.

3. RESEARCH IN STANDARDS PROCESSING

In this section, methods which use the representation presented above for processing standards in conformance checking and design are discussed. As will be shown, conformance checking is actually a subprocess of design.

3.1. Computer-Aided Checking of Standards

When performed manually by a novice designer, the process of checking design quantities for conformance with a standard expressed in conventional text form is a difficult task because:

- many of the applicable provisions are spread throughout the standard;
- proper cross-referencing among related provisions is usually not provided; and
- the performance attribute required is not explicitly stated.

The model presented provides a representation for orderly and efficient computer-aided standards checking. Several computer programs have been written that check designs for conformance with standards that are represented in this form.

In 1969, Goel and Fenves described a computer program that checks a design (i.e., a set of values for basic datums) for conformance with a standard represented by a network of decision tables [10, 1]. Goel's program requires that evaluation subroutines, i.e., subroutines used to evaluate the condition and action stubs (the expressions appearing on the left side of decision tables) of a decision table, be separately programmed in advance. A considerably modified program [18] contains a preprocessor that converts each stub into executable subprograms, leaving only the task of inputing the tables and the expressions appearing in the stubs to the user. A third program [16] extends the preprocessor by generating decision trees from the decision tables, producing source code IF-THEN-ELSE templates from the decision trees, and inserting the stubs into the source code, thus generating executable subprograms for evaluating decision tables. A fourth program, written in LISP [8], evaluates the conditions and actions of decision tables during execution, thus requiring no precoding or precompilation of the expressions. The treatment of the condition and action stubs is the only essential difference between the first, second and fourth programs. In these programs, a decision table is evaluated rule by rule. In the third program, the order of condition evaluation is prescribed when the decision tree is generated. In all four programs, when a data item value is needed during the evaluation of a decision table or tree and the data item is not yet bound to a value, the current decision table evaluation is suspended and the decision table or tree for the needed data item is evaluated. This evaluation method is called recursive, top-down, conditional, or backward chaining.

In 1984, Lopez began development of a knowledge-based system called SICAD which incorporates the representational model presented, a database management system, and an expert system application program which uses the checker and the database [14]. Besides the use of expert system and database technologies, SICAD differs from three of the previously described programs in that it expects that decision tables have been converted into decision trees and evaluates the decision trees.

3.2. Computer-Aided Design with Standards

This section discusses three research efforts in processing standards for the purpose of design. The first deals with the use of constraints in a data base, the second deals with methods for symbolically manipulating the provisions of a standard, and the third deals with a proposed interface between computer-aided design (CAD) programs and standards.

3.2.1. Standards Provisions as Database Constraints

The role of database management systems (DBMS) in structural engineering was presented in [5]. One of the key functions of DBMS is to enforce integrity constraints, that is, insure that the data present in the database are consistent with respect to these constraints. Furthermore, the relational DBMS model provides a methodology, called normalization, for removing dependencies among attributes of a relation. Examples of the various levels of normalization are given in [17].

A design database, containing information about an emerging design, must serve not as a passive repository of data, but as an active agent performing many of the consistency checks that are currently done manually. In some cases, it should also be able to do a limited amount of design, in the sense of assigning values to basic data items such that the applicable constraints are satisfied. One class of constraints deals with the satisfaction of the provisions of standards.

A mechanism of treating such constraints is presented in [7]. Briefly, the mechanism consists of the following components:

1. an additional constraint status attribute (with possible values of "satisfied" or "violated") is appended for each constraint on the attributes in the relation, and a function is provided to evaluate this attribute;
2. if desired, any constraint can be recast into an assignment procedure, which computes the value of a dependent attribute subject to the constraint and sets the constraint status attribute to "satisfied"; and
3. three commands are added to the DBMS: one to *invoke* the applicable constraint(s) on the current state of the database, flagging all tuples for which the status attribute(s) are "violated"; one to *activate* the constraint(s) for all future transactions, prohibiting any database modification that would violate the constraint; and one to *deactivate* constraint(s) if a design diverged or a new alternate design is to be started.

3.2.2. Holtz's Symbolic Manipulator of Standards

In references [1, 10], the passive or checking form of representing standard provisions is justified in the following manner. When performing design using a standard, different designers will choose different basic data items (dependent variables to be solved for subject to satisfying the applicable standard provision) to be the design quantity. Also, the same designer may switch the data item being designed at different stages of the design. Thus, the standard provision must be expressed in such a way that no preference is given to any one design method or sequence. For this reason, Holtz states, " A feature common to all constraints is that they are almost invariably formulated as a check on the adequacy of a design in terms of known or assumed values [12]." Holtz's work deals with the symbolic reformulation of such design constraints in order to produce bounds on the allowable value for certain basic data items, called "designable quantities" by Holtz. These bounds provide a range of values that gives a design which conforms to the standard, from which the user can choose. Symbolic reformulation of design constraints allows constraints to be used as "design" tools as well as "checking" tools.

Holtz developed a program, called CONMAN, that has three distinct uses, depending on the status of the basic data:

1. If all the basic data items have values, i.e. are bound, then CONMAN passively processes the standard and evaluates the network of decision tables in a manner similar to the programs described above.
2. If only one or a few data items are unbound and specified as designable, CONMAN asks which data item is designable and produces numeric bounds on the value of that data item such that any value chosen from within the bounded region conforms to the standard. The user then chooses a value from the feasible region and the program can proceed. This aspect of CONMAN is very useful for interactive design.
3. If all the data items are unbound and some are specified as designable, CONMAN produces symbolic bounds on the value of the designable data items. These symbolic bounds can be compiled into assignment procedures for repeated use. However, note that someone, i.e. the designer, must still specify how to choose a value from within the feasible region found by CONMAN.

The second use of CONMAN could automatically produce the assignment procedures for the database constraints discussed in the previous section. The third use of CONMAN could be used to generate automatically the standard-dependent portions of code in CAD programs. Holtz encountered difficulties and inefficiencies in the symbolic reformulation of constraints containing inequalities, making runtime reformulation impractical.

3.2.3. A Generic Design Standards Processor

A prototype standards processor is being developed that will act as an interface between CAD programs and design standards [9]. This proposed interface will take a component design or check request from a CAD program, perform the desired standard dependent task, and return the results to the CAD program. Knowledge of design techniques, materials, use of specifications, etc, will be represented using Artificial Intelligence and Knowledge-Based Expert Systems Techniques such as scripts and rules. This knowledge will be used by the standards processor to perform the CAD program's design requests.

The standards processor will call pre-made subprograms that perform the more common design and checking tasks. When a unique design task is requested, the standards processor will call upon an expert system to generate an executable task from existing subprograms and available knowledge. The expert system will also be capable of regenerating the executable subprograms for common design tasks when the version of the standard is modified. Thus, the standards processor can provide the processing efficiency of procedural programs, but can also provide the versatility of knowledge-based systems.

This separation of standard dependent knowledge (in the GDSP) and standard independent knowledge (in CAD program) has several advantages:

- changes in the standard produce no changes in the application CAD program;
- the standard need only be coded once for use by the generic standards processor, making it feasible to have an expert interpret and translate the standard provisions; and
- the application program can be made valid for any standard by simply changing the data, i.e., standard dependent knowledge, which the standards processor uses to satisfy the application program's requests.

References

[1] FENVES, S. J., E. H. GAYLORD, and S. K. GOEL, *Decision Table Formulation of the 1969 AISC Specification*, Civil Engineering Studies SRS 347, University of Illinois, August 1969.

[2] FENVES, S. J., K. RANKIN, and H. K. TEJUJA, *The Structure of Building Specifications*, Building Science Series 90, National Bureau of Standards, Washington, D. C., September 1976.

[3] FENVES, S. J. and R. N. WRIGHT, *The Representation and Use of Design Specifications*, Technical Note 940, National Bureau of Standards, Washington, D. C., 1977.

[4] FENVES, S. J., "Recent Developments in the Methodology for the Formulation and Organization of Design Specifications," *Engineering Structures*, Vol. 1, pp. 223-229, October 1979.

[5] FENVES, S. J. and W. J. RASDORF, "Role of Database Management Systems in Structural Engineering," *Informatics in Structural Engineering*, International Association of Bridge and Structural Engineers (IABSE), pp. 229-242, 1982.

[6] FENVES, S. J., R. N. WRIGHT, and F. STAHL, *Introduction to the Analysis Synthesis and Expression of Standards*, Draft , National Bureau of Standards, March 1983.

[7] FENVES, S. J. and W. J. RASDORF, "Treatment of Engineering Design Constraints in a Relational Database," *Engineering With Computers*, Vol. 1, No. 1, to appear in 1985.

[8] GARRETT, JR., J. H. and S. J. FENVES, *Decision Table Evaluator Users Manual*, unpublished internal course manual, Carnegie-Mellon University, 1984.

[9] GARRETT, JR., J. H., *A Generic Design Standard Processor for Use in Computer-Aided Structural Design*, unpublished Ph.D. Thesis Proposal, Carnegie-Mellon University, 1985.

[10] GOEL, S. K. and S. J. FENVES, *Computer-Aided Processing of Structural Design Specifications*, Civil Engineering Studies SRS 348, University of Illinois, November 1969.

[11] HARRIS, J. R. and R. N. WRIGHT, *Organization of Building Standards : Systematic Techniques for Scope and Arrangement*, Building Science Series NBS BSS 136, National Bureau of Standards, Washington D. C., March 1980.

[12] HOLTZ, N. M., *Symbolic Manipulation of Design Constraints - An Aid to Consistency Management*, Technical Report DRC-02-12-82, Design Research Center, Carnegie-Mellon University, April 1982.

[13] HOWARD, H. C. and S. J. FENVES, *Representation and Comparison of Design Specifications*, Department of Civil Engineering R-83-141, Carnegie-Mellon University, March 1983.

[14] LOPEZ, L. A. and S. L. ELAM, *SICAD: A Prototype Knowledge Based System for Conformance Checking and Design*, Technical Report, University of Illinois, 1984.

[15] RASDORF, W. J., *Design Specification Representation at the Outline-Information Network-Decision Table Interface*, unpublished Master's Thesis, Carnegie-Mellon University, August 1979.

[16] RUDNICKI, R., *Decision Table Based Software for Designing With Standards*, unpublished Master's Thesis, Carnegie-Mellon University, 1984.

[17] SCHAEFER, M. J., D. R. REHAK, and S. J. FENVES, "Introduction to Relational Databases Using Structural Engineering Examples," *Journal of Technical Topics in Civil Engineering*, Vol. 110, No. 1, pp. 1-18, May 1984.

[18] STIRK, J. A., *Two Software Aids For Design Specification Use*, unpublished Master's Thesis, Carnegie-Mellon University, October 1981.