STEEL DESIGN AND RELIABILITY USING EUROCODE 3

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CONTENTS

ABSTRACT

The twin aims of this research were to improve the presentation of codified design information and to investigate the methods used to calibrate the partial safety factors applied to resistance functions (γ_R-factors) so as to improve both the economy and the reliability of the predictions.

A restructured version of EC3 (known as F-EC3) was developed by rearranging the design clauses on the basis of design tasks. This system enables the code to become more user-friendly. Hypertext versions of both EC3 and F-EC3 have been created on PC-based Microsoft Windows compatible software. The implications of hypertext on structural codes are investigated.

The procedure used for calibrating the γ_R-factors contained within EC3 (the Annex Z method) was reviewed and an alternative technique involving less assumption is proposed. A comprehensive set of measurements recording the material strength and the geometric properties of steel were obtained and collated. The large data set (over 7000 tests) was sufficient to evaluate the type of probability distribution characterising the variability of the basic material and geometric properties of structural steel. The resulting data were combined with experimental test results to determine the reliability of plate girder design and restrained beam design. The theoretical shear buckling resistance of plate girders (predicted by the simple post-critical and tension field methods) was compared with experimental test results to determine reliability. The analysis demonstrated that plate girder design falls well short of the target reliability and an adjustment to the design methods is required in order to ensure safe design. A series of 4-point bending tests on laterally restrained beams were conducted to establish the accuracy of the $M_{pl.Rd}$ resistance function. This study quantifies the degree of conservatism inherent in the $M_{pl.Rd}$ design method and provides convincing evidence of the need to reduce the γ_R-factor applied to this resistance function. A modification is proposed to the design formulae which improves accuracy and permits the full moment capacity of restrained beams to be utilised.

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LATIN NOTATION

a	clear distance between web stiffeners (plate girders)
A	section area
B	section width
$\bar{b}$	resistance function correction factor
b	section width
b_{max}	maximum value of predicted over experimental resistance
b_{min}	minimum value of predicted over experimental resistance
C_1, C_2	s depending on loading and end restraint conditions
d	web depth (plate girders)
E	young's modulus
$f_{1.5}$	stress corresponding to 1.5% strain
f_u	ultimate tensile stress
f_y	yield stress
f_{yf}	yield stress of flange
f_{yw}	yield stress of web
h	section depth
I_t	torsional inertia
I_w	warping inertia
I_y	major axis second moment of area
I_z	minor axis second moment of area
k	effective length factor(l_e/L)
k_c	ratio of nominal resistance over characteristic resistance
k_τ	is the buckling factor for shear
k_w	factor referring to end warping
l_a	Lever arm
M	experimental moment of resistance
$M_{el.Rd}$	elastic moment of resistance
$M_{f.Rd}$	design moment of resistance
$M_{pl.Rd}$	plastic moment of resistance

$M_{pl.u}$ alternative plastic moment of resistance
M_{pu} modified plastic moment of resistance
n sample size (number of tests)
r resistance
r_d design resistance
r_e experimental resistance value
r_k characteristic resistance
r_m resistance calculated using mean values of basic variables
r_n nominal resistance
r_t theoretical resistance value
s_k characteristic load
t factor taken from Student's t-distribution
t_f flange thickness
t_w web thickness
V coefficient of variation
V_b coefficient of variation of $\overline{b}$
$V_{ba.Rd}$ shear buckling resistance according to the simple post-critical method
$V_{bb.Rd}$ shear buckling resistance according to the tensional field method
V_{exp} experimental shear load at failure
V_{fy} coefficient of variation for yield stress
V_r coefficient of variation of resistance
$V_{wpl.y}$ coefficient of variation of the major axis plastic section modulus
$W_{el.y}$ major axis elastic modulus
$W_{el.z}$ minor axis elastic modulus
$W_{pl.y}$ major axis plastic section modulus
$W_{pl.z}$ minor axis plastic section modulus
X basic variable
x distance from neutral axis
Z_g distance between the shear centre and the point of load application

GREEK NOTATION

α_R	sensitivity factor for resistance
β	safety or reliability index
ϕ	end rotation
γ_M	EC3 "boxed value" for resistance partial safety factor
γ_{M0}	resistance partial safety factor for class 1,2 or 3 cross-sections
γ_{M1}	resistance partial safety factor for class 4 cross-sections
γ_p	additional partial safety factor proposed herein
γ_R	partial safety factor for resistance
γ_R^*	modified partial safety factor ($k_c.\gamma_R$)
$\bar{\lambda}_{LT}$	non-dimensional slenderness
$\bar{\lambda}_w$	web slenderness
μ_r	mean resistance
σ	standard deviation or stress
σ_b	standard deviation of $\bar{b}$
σ_{bb}	tension field strength
σ_R	standard deviation of resistance
τ_{bu}	simple post-critical shear strength
τ_{cr}	elastic critical shear strength

ABBREVIATIONS

CEN	European Committee for Standardisation
COV	coefficient of variation
EC3	Eurocode 3: Design of steel structures
NAD	National Application Document
p.d.f.	probability distribution function
PC	personal computer

Chapter One

INTRODUCTION

1.1 PREAMBLE

This thesis is primarily concerned with the recently published Eurocode 3: Design of steel structures - Part 1.1: General rules and rules for buildings (CEN, 1993). Eurocode 3 contains much of today's knowledge of the structural phenomena relating to the design of steel buildings. It is a limit state code, with the intention that probability of failure should remain relatively constant regardless of the design task or material considered.

This thesis investigates two main aspects of the code: firstly the methods used for presenting the seemingly complex information contained within the code; and secondly the reliability of certain structural elements designed in accordance with the code. Within each of these broad subject areas a number of separate aspects of the code are considered; these are briefly introduced in this chapter.

1.2 THE BACKGROUND TO EUROCODE 3

This brief review is concerned with the methods used for presenting code information, the overall aims of the Eurocodes, together with a history of the development of codes governing the use of structural steelwork. It has been included in order to develop an understanding of how the methods of presenting design information have changed over time, and in so doing, to point the way to how an improved method can be developed for the future. A summary is given of who uses the Eurocodes and what the author considers are their requirements. Finally, a review of the supporting resources that have been developed to assist the transfer from existing national codes to Eurocode 3 is reported.

1.3 ALTERNATIVE METHODS OF PRESENTING DESIGN INFORMATION

Eurocode 3 is the latest code in a series of regulations introduced this century to govern the use of steel in construction. Unfortunately for the designer, the task of implementing new and revised standards is becoming increasingly difficult. Codes are rapidly increasing in size, and locating relevant design material from large and seemingly complex codes can take a considerable amount of time.

Part of the research reported in this thesis has been aimed at improving the way code information is presented to the designer - and in so doing aid the transfer from existing national standards to the Eurocodes. A number of user-friendly versions of Eurocode 3 are already available. However, all these versions use roughly the same method of arranging design material. This work has investigated an alternative format for structuring the design clauses contained within the Eurocodes. In addition, the application of hypertext software to codes has been investigated.

1.4 THE CALIBRATION OF PARTIAL SAFETY FACTORS

As stated previously Structural Eurocodes are written in accordance with the concept of limit state design. Partial safety factors are applied both to the design loading and design resistance. In theory, probability of failure should remain relatively constant regardless of the design type or material considered. Safety factors are derived partly on the basis of statistical analysis and partly based using experience of what has proved safe in the past. The work reported herein has been to establish the validity of the safety factors applied to steel designed using EC3. These factors are known as γ-factors and the statistical method used for calculating them is reviewed. Following this work an improvement to the method is proposed. The justification and effect of the improvement is reported.

1.5 THE VARIABILITY OF MATERIAL AND GEOMETRIC PROPERTIES

During the calibration of the Eurocode 3 partial safety factors, assumptions are made concerning the variability of the material and geometric properties of steelwork. In the background documentation to the Eurocodes, values are specified for the statistical variability of the various basic variables relating to steel. These values are based on work originally undertaken during the 1970's. Since manufacturing methods have improved during the past 20 years, an important question concerning reliability levels is what is the true variability of the basic variables relating to steel design?

Utilising manufacturers' quality assurance records, the author has established a measure of the variability of the material and geometric properties of steelwork manufactured by two leading European producers. The resulting statistical data is compared with the measures of variability assumed during the calibration of the Eurocode 3 partial safety factors. The data is also utilised for the calibration of certain resistance functions in the subsequent chapters.

1.6 THE RELATIVE SAFETY LEVELS OF EUROCODE 3 DESIGN

The variation of the limit state approach to design used in the Structural Eurocodes involves the use of what are termed boxed-values of partial safety factors. These values are specified both within the codes and in the national application documents produced by each CEN member state. This approach gives each member state the freedom to adjust the relative economies achieved by the Eurocodes to the levels already achieved by the existing national standards. However, the system does create a situation where different resistance functions have the same values of partial safety factors applied to them for political, and not safety reasons.

Resistance functions vary in their ability to predict resistance, since the various types of failure mechanisms differ in their degree of repeatability. For example, the pull-out capacity of bolts in tension is a substantially easier failure mechanism to predict accurately than the load required to cause a lateral torsional buckling type failure. In theory, the degrees of uncertainty associated with various resistance functions should combine with the variability of material and geometric properties to produce different probabilities of failure between the various resistance functions; assuming a uniform value of partial safety factor is applied to a range of different resistance functions.

Utilising experimental test results, the author has investigated the reliability of two radically different resistance functions. Firstly, the reliability of plate girder design. A design task that is both complex and associated with a high degree of instability. And secondly, the reliability of restrained beams; which is a comparatively simple design task. Thus, the degree with which the boxed-values approach to limit state design achieves the objective of uniform safety levels across a range of different resistance functions is investigated.

A series of bending tests (Hasan and Hancock, 1988) have demonstrated that the plastic moment of resistance design formula substantially underestimates the bending strength of cold-formed rectangular hollow sections. This underestimation of resistance is caused by strain hardening of the sections during the rotation of the plastic hinges. In order to establish whether class 1 hot rolled open sections also have the potential to substantially exceed their plastic moment capacity, a series of bending tests have been carried out and are reported herein. In addition, two design formulae are proposed that take some account of the additional reserve of strength caused by strain hardening. Utilising the results from the experimental testing, the appropriateness of these formulae is established.

Chapter Two

THE BACKGROUND TO EUROCODE 3

2.1 THE HISTORICAL CONTEXT

Today's codes provide detailed guidance for a huge variety of different design situations. These comprehensive codes are a relatively new phenomenon. In this chapter an attempt has been made to chart the development of the steel codes that are taken for granted today, and in so doing become better informed about the way codes should be structured in future.

2.1.1 THE EARLIEST KNOWN CODES

The earliest known building code dates from Ancient Babylonia. In the 18th Century BC the Code of King Hammurabi stated:

> *"If a builder has built a house for a man, and his work is not strong, and if the house he has built falls in and kills the house-holder, that builder shall be slain."*

This may be considered rather harsh by today's standards, but the intention to ensure good practice remains unchanged. The Ancient Greeks were more lenient to the construction industry. Architects hired craftsmen and supervised construction; with the specifications being written in stone. The Romans devised numerous building regulations, covering the construction of buildings, water supply and sanitation facilities. They also wrote standards for setting out projects.

The first major code this Millennium was the London Building Act, drafted after the Great Fire of London which destroyed 15,000 structures in 1666. Surprisingly no restrictions were made on the use of combustible materials, despite the warnings from Sir Christopher Wren. Throughout history, codes and building regulations have been introduced following major disasters in order to ensure good practice and safeguard the public.

2.1.2 PRE BS449 REGULATIONS

Regulations to govern the use of steel in buildings have been in existence in the UK since the London County Council (General Powers) Act of 1909. This act was one of the first of its kind and was used as a model code by other UK cities and throughout what was then known as the Dominions. The LCC act remained in force for 23 years until the advent of BS449 in April 1932. Prior to 1932 many members of the engineering profession, particularly those involved in the steel industry felt that regulations controlling the use of steel were too restrictive, not fully utilising the excellent properties steel had to offer. Methods of manufacture had improved, along with more precise methods of modelling structural phenomena. It was with these thoughts in mind that the British Steelwork Association approached the Department of Scientific and Industrial Research with a request that the use of steel in structures be investigated. As a result the Steel Structures Research Committee (SSRC) was appointed in August 1929, with the following terms of reference:

- To review existing methods and regulations for the design of steel structures.
- To investigate the application of modern theory to the design of steel structures and translate to practice the results that appear to lead to more efficient and economical design, i.e, the creation of steel design code.

In 1931 Stanley Baldwin, Lord President of the council wrote in his preface to the first SSRC report[5]:

> *"It reflects great credit on the leaders of the structural steel industry that intense trade depression, which has affected their industry more seriously than many others, has not held them back from devoting money and energy towards the studies of the fundamental principles of technique and practice. The British Steelwork Association, in seeking to foster development in this way, has taken a far sighted view."*

These words are strikingly similar to those spoken some 61 years later by Rt. Hon. Michael Heseltine's representative from the DTI at the 1st CIMsteel Convention in Runymeed, in December 1992. The SSRC was in existence for 6 years, during which time it published 3 substantial and far reaching reports (SSRC, 1931), (SSRC, 1934) and (SSRC, 1936). The first task undertaken was a review and comparison of the existing regulations governing the design of steel structures in the UK, the Commonwealth and the rest of the world. A selection of the findings of this review are briefly summarised as follows:

Regulations in the UK by 1931. The building bye-laws for local authorities including Liverpool, Manchester, Bristol and Norwich adopted the standards set by the LCC 1909 Act, with a number of modifications to suit their own requirements. Detailed codes of practice were not available and in the majority of cases steel construction was covered by a clause to the effect that :

> *"The framework shall be of sufficient strength to secure due stability and shall be properly put together and protected with suitable and durable material non-conductive to heat; and the framework shall be filled in with bricks, stone or other hard and incombustible material properly and solidly put together, and of such thickness as shall be necessary to secure due stability to such filling."*

No criteria for checking stability were specified. The Scottish building bye-laws seem to have given more guidance than their English counterparts, with both Edinburgh and Glasgow bye-laws defining minimum superimposed loads for floors. No regulations dealing with steel construction were provided in Ireland.

Regulations in Commonwealth Countries by 1931. In contrast to the UK situation, many cities in the Commonwealth provided in their building bye-laws detailed codes of practice relating to the design of steelwork structures. Some of the more notable bye-laws were: The Municipal County of Sydney, 1917; City of Melbourne, 1923; City of Perth, 1929; City of Auckland, 1925; City of Wellington, 1908; Municipality of Johannesburg, 1925; and finally the Standard specification for steel structures for building, (Second Edition, 1930) produced by the Canadian Engineering Standards Association. J.F. Baker -

one of the most active members of the SSRC - seems to have found wide variation between the various regulations; in (Baker, 1936) he wrote:

> " *The differences between the bye-laws of these eight districts are more easily seen than the reasons for them* ".

Regulations in Other Countries by 1931. As would be found today, the regulations seem to have varied considerably between different countries. In America every large town had its own code of practice. This contrasted with the situation in France, where in the absence of an official code of practice steelwork structures seem to have been built in accordance with the regulations concerned with the design of steel bridges and railway buildings. In Germany regulations relating to permissible stresses for mild steel and high tensile steel used in building construction were issued by the Prussian Minister for Public Welfare in 1929.

It was found that the materials demanded and working stresses allowed were fairly uniform between different countries. Significant differences were found between the various clauses governing the proportioning of members, but it was the loading requirements that showed most variation. New York and German codes specified significantly lower imposed loads than their London counterparts. In some cases the loading specified for London buildings was over twice that specified in New York.

2.1.3 BS449: THE UK's FIRST NATIONAL STEEL CODE

At the time of the formation of SSRC, regulations governing the use of steel in structures were considered unsatisfactory. It was necessary to draw up immediate recommendations for a code of practice. These recommendations, published in the Committee's first report (SSRC, 1932) removed many of the restrictions on the use of steelwork and formed what was in effect a draft version of the BS449 (BSI, 1932).

Although the 1932 version of BS449 was by modern standards a small code it filled a much needed gap in the design of structural steelwork. It was adopted almost immediately by London County Council, the Ministry of Health and H.M. Office of Works. The various clauses were rather brief in nature and covered areas of design such as loading requirements, fire protection, detailing requirements and pressures on foundations. It must be remembered that BS449: 1932 was based on the initial recommendations of the SSRC, before the results of their research were made available. During the period between

1929 and 1936 a series of detailed laboratory experiments and tests on actual buildings were performed. Based on this data the SSRC presented more refined recommendations, covering areas such as multi-storey frames in their final report, issued in 1936. The Chairman of the Committee suggested that these recommendations be permitted as an alternative to the 1932 version of BS449, which the Committee must have considered as out of date. Design was permitted in accordance with these recommendations, although they never became widely used.

A programme of codes of practice for buildings was established under the direction of the Ministry of Works in 1942. This resulted in a code for the use of structural steel in buildings, which was issued as CP113 in 1948. CP113 and BS449 contained basically the same information; the main difference being that BS449 was a mandatory document, whilst CP113 took the form of recommendations that represented a standard of good practice. When the codes of practice council was formed within the BSI, it was decided to incorporate CP113 into the Fourth Revision of BS449, issued in May 1959.

It is the practice of BSI to review all specifications at least every 5 years. Drafting committees are maintained after codes are issued and recalled when code reviews are deemed necessary. During these reviews the codes are updated to take advantage of any developments in the understanding of structural behaviour resulting from research projects. BS449 was revised in 1935, 1937, 1948, 1959 and 1969. Of these the 1948 revision was the most substantial, setting the style followed by later issues.

2.1.4 BS5950: A LIMIT STATE CODE FOR STEEL

During the preparation of the metric version of BS449, issued in 1969, the need for a full revision of the code was identified. Accordingly, the B/20 Committee was re-established with the task of producing a code incorporating recent advances in both design and construction techniques. By this stage the benefits of the limit state design approach were realised, thus the B/20 Committee agreed that the new code should be written in accordance with limit state theory.

The style adopted for structuring clauses changed from the BS449 system to that of the Australian Standard AS CA1. Another change was in the method of code drafting. For the first time BSI employed consulting engineers to draft the clauses. Previous codes had been drafted by committee members. The majority of work on BS5950 was conducted by a single engineer whose job was to prepare discussion papers and trial clauses for submission to the various sub-committees for comment. This approach was subsequently repeated for other BSI codes.

BS5950: Part 1 was initially issued in draft version in 1977, then commonly known as the B20 draft. As a result of comments received the Committee decided that the code needed redrafting into a shorter, more streamlined form. In 1978 the European Convention for Constructional Steelwork (ECCS), published the European recommendations for Steel Construction (ECCS, 1978). These recommendations were a synthesis of specifications and codes in force at the time, combined with the most up-to-date knowledge on structural behaviour, applied using limit state and plastic design principles. It was viewed as a model code for its time, and Constrado, the organisation delegated the task of redrafting B20, was instructed to prepare the final version of BS5950 using a style and content as close to the ECCS recommendations as possible, since the future EC3: Part 1.1 (CEN, 1993) would be based on the ECCS recommendations.

The resulting code was completely different to BS449, both in layout and format, technical content and design procedures. Some of the more obvious changes were:

- BS5950 provides detailed guidance for plastic design. By comparison, BS449 accepted plastic design but provided no guidance.
- Tables and graphs were supplemented by the formulae from which they were derived.
- BS5950 provides two separate sets of design requirements for certain of the more complex design tasks, such as those for the design of laterally unrestrained beams; one set of simplified rules that provide a quick, though conservative design and another set that provide a more complete and accurate model of the design considered.

2.2 THE INTRODUCTION OF STRUCTURAL EUROCODES

The package of Structural Eurocodes are currently being issued, with many of them now at the ENV stage. This means that design may still be conducted using existing national codes, though designs conducted in accordance with the Eurocodes will be acceptable in all European Union (EU) member states. It is expected that conversion to the full EN status will be completed in about five years time. When this happens a decision will be made between CEN, the EU and member states on whether existing national standards should be withdrawn or allowed to coexist alongside the Eurocodes for a given period of time.

Structural Eurocodes form part of the EU's overall goal of "unification" for Western Europe. One of the main methods of completing "unification" is through the elimination of internal barriers to trade. It has long been felt that national codes, varying in style and content create internal barriers to trade. Thus it is the expectation that through the harmonisation of structural codes considerable progress towards the EU's objective of an "internal market" can be made. The construction industry is considered particularly important since it accounts for a significant proportion of the total EU's GNP.

In 1985 the European Commission (EC) published a White Paper entitled "Completing the Internal market" listing the programme and measures needed to ensure the free flow of goods, services, people and capital throughout the EU. This programme has been further expedited by the Single European Act of February 1986, which for the first time amended the EC founding charter, the 1957 Treaty of Rome. The Act speeds decision making by removing the right of member states to veto on issues relating to the Internal Market, hence allowing qualified majority voting. The Act has the following objectives:

- complete the internal market by 1992
- improved research and development
- progress towards economic and monetary union
- improve working environment and conditions

The task of creating the harmonised technical standards has been given to the European standardisation bodies set up by industry. These include:

- CEN (European Committee for Standardisation)
- CENELEC (European Committee for Standardisation in the Electrotechnical field)
- CEPT (European Conference of Postal and Telecommunications Administrations)

Of these organisations CEN, the largest regional standards group in the world is responsible for developing the Structural Eurocodes. The Structural Eurocodes are being written under the guidance of CEN Technical Committee TC250 and cover the design of a wide range of structures. The structural materials covered include: concrete, steel, composite, aluminium, timber and masonry. Also being prepared are documents covering loading, geotechnics and seismic action.

2.3 THE AIM OF THE EUROCODES

In additional to specifying design requirements, Eurocodes have the following aims:

- to harmonise design standards across the EU's "Internal Market";
- to facilitate the free flow of engineering expertise throughout the EU;
- to provide a consistent legal framework and terminology for construction related contracts;
- to provide more comprehensive codes by combining the resources of member states.

The cost of research is high and it is expected that savings can be made by spreading the cost between a number of different countries. By combining the work of several organisations, design procedures can be more accurately calibrated. Developments in the modelling of structural phenomena can therefore be tested more thoroughly and a much wider range of design situations can be addressed.

The ultimate aim of the Eurocodes is that the structures designed by using them will become less costly due to more economic designs, greater competition and the resulting economies of scale. The benefits are obvious. However, the size of these very comprehensive codes tends to be much greater than the equivalent national documents with which engineers are presently familiar. Finding methods for combining the benefits of scale with the production of practical, user-friendly codes is not easy; the problem is made more complicated by understandable attempts to amalgamate the current design rules and design thinking of EC member states into the new Eurocode rules. In addition, modern more accurate models of structural behaviour tend to be more complicated than established methods. It is hardly surprising that the present structure of the Eurocodes finds it difficult to combine ease of use with fully comprehensive coverage of structural design. The view has developed that Eurocodes will rely heavily on supporting documents and computer software to achieve usability.

2.4 WHO USES EUROCODES?

Structural codes of practice are aimed at engineers with a sound knowledge of applied mechanics and the behaviour of structures. They are not intended for people uneducated in the field of structural analysis and hence assume a certain degree of knowledge in the user. Listed below is a summary of the existing users of codes, and what the author thinks their requirements are:

Clients. The general requirement is for structures to be of low initial cost, adequate reliability and low maintenance cost. Comprehensive codes are advantageous to the client, as designs conducted without the benefit of codes are difficult to check in order to ensure reliability. Complex design procedures are of no concern, provided the economy of the finished structure is not adversely affected.

International practices of consulting engineers. These organisations, competing in world markets, prefer codes that provide guidance on the wide variety of design types they encounter. The use of a single code in all the EFTA member states will allow easier access to foreign markets. Experienced engineers concerned with the design of unusual and complex structures may prefer to refer to fundamental knowledge and want freedom to work outside the scope of the design formulae of the codes. The principle/application rule approach allows for this freedom and is likely to prove popular. Comprehensive codes providing guidance on the design of unusual design tasks would also be beneficial, combined with clearly defined design clauses that list the limitations and applicability of the various design functions.

Designers with limited experience, concerned with simple repetitive design tasks, clearly prefer simple, all embracing design rules allowing for speed in application. These problems have been alleviated to a large extent by computer aided engineering. Most large consultancy practices have extensive suites of design software. This should allow the inexperienced and less able engineers to design to Eurocodes, largely unaware of the complex rules to which they conform.

Small practices of consulting engineers. Designs tends to be limited to certain structures. Extensive suites of design software are unlikely to be available. Where designs fall outside the range covered by the software and hand calculation is necessary, then short, simple codes that allow for speed in application are beneficial. Codes making use of design charts and tables are particularly useful.

Steelwork fabricators. Design tends to be highly automated with hand calculation unusual. Detailed codes providing accurate and economical design guidance for a wide range of design situations, particularly design of connections and joints would be useful. Long design procedures should not unduly affect design time as most design is conducted using computer software. Quality standards that are clear and easy to implement, that do not impede flow of work and require changes to established procedures are preferred.

Site based construction engineers. A limited amount of design is conducted on site. The site based engineer is likely to prefer simplified codes covering only a limited range of design types.

Regulatory authorities. The checking engineer wants a clearly defined set of rules that he or she can check have been complied with. Comprehensive codes are advantageous, designs conducted without the benefit of codes are difficult to check.

Civil engineering students and lecturers. The civil engineering academic wants to know the background to the rules. Students want clear, simple rules that are easily located with the minimum of time. Clear design procedures are particularly attractive allowing less scope for mistakes.

Software houses. Software engineers prefer clauses organised on the basis of the area of structural behaviour to which they relate, rather than the design task to which they relate. Information specifying the limits of applicability of the various design functions is useful. Design charts and tables are of little use with the raw analytical models being preferred.

2.5 SUPPORTING RESOURCES FOR USE WITH EC3

The Eurocodes will never satisfy all the needs of the construction industry. They merely set out guidelines for the design of certain structures. They do however provide a platform of knowledge, from which design guides can be developed in order to satisfy the specific needs of individual groups of engineers. An attempt to meet these needs has been made with the following supporting literature produced in parallel with the Eurocodes:

C-EC3 - Concise EC3 (SCI, 1992). This simplified version of EC3 is intended as a self contained, stand alone design guide, that will introduce designers to the provisions and style of EC3 by building on familiar ground. It is hoped that when designers become accustomed to C-EC3, they will progress to using the full EC3. Differences between C-EC3 and BS5950 are clearly explained, together with procedure tables that list the steps necessary for the design of certain structural elements. C-EC3 is the ideal document for engineers involved with hand calculations, that may otherwise become overwhelmed by the sheer size and complexity of the full code.

Introduction to C-EC3 (SCI, 1993). This publication contains a series of flow charts that provide a step-by-step approach to design, together with a series of comprehensive design examples. This document combined with C-EC3 will prove useful during the initial transition between BS5950 and EC3. Changes in style and design philosophy are clearly explained, with the clauses relating to particular design tasks clearly identified.

E-EC3 - Essentials of EC3 (ECCS, 1991). Unlike C-EC3, E-EC3 is not a stand alone document. It is intended as a complimentary document to EC3, containing only those clauses that are used in day-to-day design.

Structural steel sections: Dimensions and properties to BS4 and BS4848 for use with EC3 (SCI, 1992). This document lists the section properties and classification of cross-sections for UK steelwork section sizes. This information is invaluable as the classification of cross-sections can be time consuming if carried out directly from EC3.

Eurocode background documentation. A series of detailed background documents have been prepared by the drafting committees. They are extremely specialised in nature, listing the test results used for calibrating the design functions. They could prove useful if the calculations used to develop some of the tables are required, otherwise they are of little use in everyday design.

Worked examples for the design of steel structures (BRE, 1994). This publication provides detailed worked examples conducted in accordance with EC3. Some of the areas covered include the design of braced and unbraced frames, roof trusses and gantry girders. The worked examples link into C-EC3 and are very comprehensive. This document will undoubtedly help during the transition period between BS5950 and EC3.

European Steel Design Education Programme (ESDEP). A vast library of lecture notes, worked examples, videos, slides and software have been produced for the purpose of teaching steelwork design to students within the European Community. Design is conducted in accordance with the Eurocodes.

Reference standards. As with previous codes, Eurocodes rely heavily on references to various other standards. Eurocodes make references to various CEN and ISO standards, many of which have yet to be written. Where this is the case the National Application Documents reference the relevant national standard, until such time as all the standards are written.

Software packages. Many software packages are available offering design in accordance with the Eurocodes.

Chapter Three

A FUTURE STRUCTURE FOR EUROCODE 3

3.1 INTRODUCTION

The main functions of structural codes of practice (henceforth referred to as codes) are:

- to define the design values of applied loading;
- to provide methods of modelling structural phenomena;
- to quantify "failure".

The function of providing methods of modelling structural phenomena inevitably leads to regular code revision, a feature that has become the nature of modern codes. Complaints arising from designers each time codes are revised or new codes are introduced are all too familiar. This is not a new problem and existed when the (BCSA, 1959) published a brochure explaining the changes to BS449 due to the 1959 revision. In the forward to that document it is written:

> *"The onset of new or revised regulations invariably heralds a trying period for the unfortunate people who have to work to such regulations. This applies both to those who have to comply with, and those who have to administer, such regulations".*

Codes will continually be revised to keep them up to date as knowledge of the behaviour of structures improves. This places code writers in a difficult position, since they must prepare modern codes that accurately reflect developments in understanding, hence taking advantage of progress in engineering knowledge, whilst at the same time ensuring that codes do not become more complicated than designers can cope with. Thus code writing is itself becoming an increasingly difficult process.

Such problems are aggravated by the style adopted for modern codes. In the UK - apart from the change to A4 - the appearance of structural codes has hardly altered in 60 years, despite great changes in publishing, design fashion and the general availability of a wide range of information. Code clauses are still organised on a structural phenomenon basis. Whilst this format may have been suited to the small codes of the 1930's, it now results in a commonly identified problem for designers - that of extracting relevant information in a logical order from an unfamiliar document. Since codes appear to be increasing in size at an exponential rate, a method needs to be found for designers engaged on a particular task to locate the relevant clauses quickly, without becoming overwhelmed by the shear mass of technical information. The rapidly increasing size of codes is illustrated by Fig. 3.1.

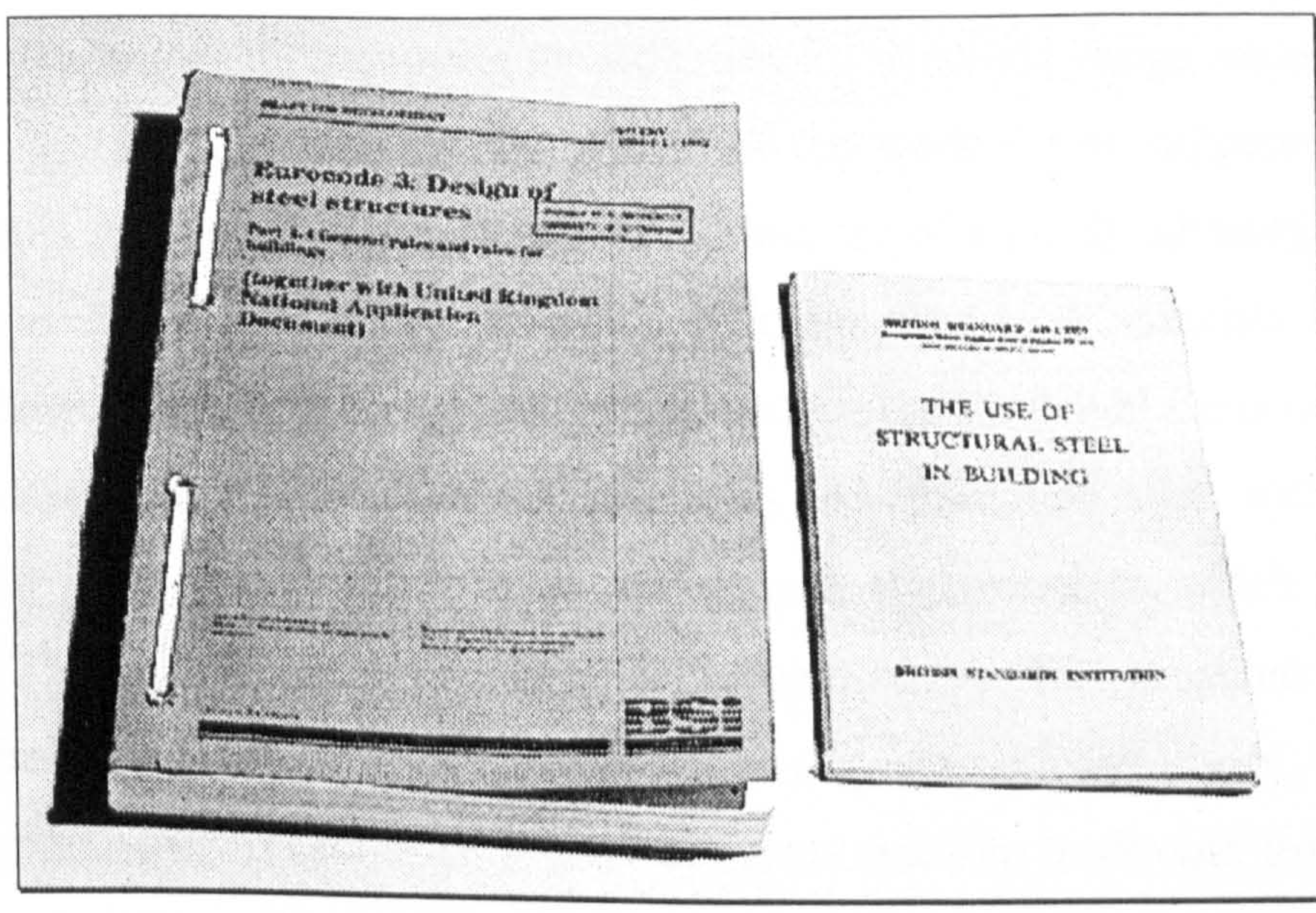


Fig. 3.1: The effect of 33 years of increasing the size of steel design codes

EC3: Part 1.1 (1992) left, BS449 (1959) right.

This chapter reports work carried out to develop a future structure for Eurocode 3: Design of steel structures - Part 1.1: General rules and rules for buildings (CEN, 1993). The Author has restructured Eurocode 3 into an alternative format called F-EC3 (Byfield and Nethercot, 1994), which differs from Eurocode 3 in that clauses are arranged on a design task basis. This arrangement has proved helpful to engineers unfamiliar with the code, since all the clauses relevant to the particular design task actually being undertaken are clearly identified.

3.2 RELATED WORK

F-EC3 is by no means the only attempt at improving the ease with which designers transfer from their national codes to the Eurocode. The (SCI, 1992) have created the "Concise Eurocode 3" (C-EC3) and the (ECCS, 1991) have created the "Essentials of EC3" (E-EC3). Both C-EC3 and E-EC3 contain selected Eurocode 3 material that is supplemented with additional information intended to aid the transition between national standards and the Eurocode. F-EC3 differs from these documents since it contains the complete Eurocode 3: Part 1.1 text and no attempt has been made to change or supplement the code content; clauses are simply reorganised.

Various methods for improving the way rules for structural design are codified were investigated by (Moffatt and Dowling, 1980). In this work it was suggested that code clauses be split into two different classes: those that are of a purely advisory nature, and those that are enforceable, with different type-settings used to differentiate between the two. CEN have adopted an almost identical system for the Structural Eurocodes. Clauses are split into two classes known as principle rules and application rules, with application rules printed in italics. Principle rules are general statements to which there is no alternative. The application rules are generally recognised rules that constitute a means for satisfying the principle rules requirements, though they are not compulsory. This appears to be a popular development since it gives designers freedom to deviate from otherwise rigid, comprehensive code requirements, providing that the design satisfies the principle rules. Moffatt and Dowling also proposed radically changing the way design material is codified. A system by which the codes reference "technical information sheets" was proposed. The data sheets would provide detailed guidance for specific design situations. Codes would henceforth cease to be used in near isolation but would become a source of basic information, design principles and requirements, and would be used as a source of references to the individual data sheets. The idea is attractive because codes would be reduced in size and results from research work could rapidly be used for design purposes by incorporation into data sheets. The system may, however, involve more overall complexity than the existing arrangement, since technical information sheets would be likely to proliferate, leaving designers the task of following up numerous references.

One of the pioneers into the development of logical methods for organising code material was Fenves. The concept of using decision tables for representing the detailed decisions made during the application of codes to design tasks was investigated (Nyman and Fenves, 1975). Unfortunately simple design tasks often require seemingly complex decision tables, which are therefore most suited as the basis from which computer programs can be written.

3.3 THE PRESENT STRUCTURE OF EUROCODE 3

Modern Eurocodes use the same principle for arranging design clauses as was applied to BS449 (BSI, 1932), i.e. they are arranged on the basis of the structural phenomena to which they relate, not on the basis of the material required for the design of particular items.

Since Eurocode 3 is a large document, designers unfamiliar with the Code may find the task of locating relevant information time consuming. There is the possibility that essential clauses may be missed in the sheer volume of technical information. Thus a situation could be envisaged in which a design engineer simply finds a pre-prepared worked example (BRE *et al*, 1994) that he believes closely matches his particular requirements, and follows this in the expectation that all relevant checks will be covered.

Eurocode 3 has deliberately been prepared with the aim of covering a wide variety of possible applications. It is thought to be largely for this reason that the present arrangement of clauses has been adopted. Of course, it does have the advantage that material relating to a specific phenomenon appears only once and that the user is not confused by having to refer to clauses in sections relating to different types of structural element e.g. clauses in the beam sections when designing columns. Such an arrangement does, however, place a substantial requirement on the user to be aware of all the checks necessary for the task in hand and to be capable of locating relevant design assistance.

3.4 F-EC3: A USER-FRIENDLY STRUCTURE FOR EC3

Using the principle of arranging clauses on a design task basis, Eurocode 3 has been reorganised into an alternative format called F-EC3. No attempt has been made to change the content of the clauses, only the order in which they appear. Extracts from the contents of F-EC3 are listed on the following page to illustrate the format used, whilst the reorganised code is contained in Appendix 1. It should be noted that F-EC3 is only partially completed.

Chapter 1: Introduction

Chapter 2: Selection of materials

Chapter 3: Design requirements
3.1 General
3.2 Actions
3.3 Deflections
3.4 Dynamic effects
3.5 Durability
3.6 Fire resistance
3.7 Fatigue
3.8 Disproportionate collapse

Chapter 4: Analysis of structures
4.1 Basis
4.2 Simple multi-storey construction
4.3 Continuous multi-storey braced frames
4.4 Continuous multi-storey unbraced frames

Chapter 5: Member design
5.1 General
5.2 Laterally restrained beams
5.3 Laterally unrestrained beams
5.4 Columns
5.5 Struts and ties
5.6 Purlins and side rails
5.7 Plate girders
5.8 Lattice girders

Chapter 6: Connection design
6.1 General
6.2 Detailing requirements
6.3 Beam to column connections
6.4 Beam to beam connections
6.5 Column splices
6.6 Column baseplates
6.7 Bracing connections
6.8 Lattice girders

Chapter 7: Design of welds and fasteners
7.1 General
7.2 Bolts
7.3 Rivets and pins
7.4 Welds

Chapter 8: Fabrication and erection

Chapter 9: Design assisted by testing

Whilst F-EC3 represents only a reorganisation of Eurocode 3, it does appear to be substantially easier to use for the less experienced or for those transferring from national codes. The user simply needs to identify the subsection relating to the design task being undertaken. Clauses relevant to each design task are clearly identified. Sub-sections form design procedures, with clauses arranged in the same order followed in design. Thus if an engineer is faced with the design of a column, but is unfamiliar with the Code, he need simply locate the relevant sub-section to find code material presented in a logical order. By contrast, the engineer transferring from the much smaller BS5950 (BSI, 1990) to Eurocode 3, will have to read large sections of complex code material, simply in order to identify relevant clauses. The logical arrangement of F-EC3 is illustrated by Table 3.1 below, which shows the location of material necessary for the design of a restrained beam.

F-EC3	Page	EC3	Page
5.2 Laterally restrained beams	64	5.1.5 Beams	54
5.2.1 Bending	64	5.4.5 Bending moment	87
5.2.1.1 Basis	64	5.4.5.1 Basis	87
5.2.1.2 Bending with low shear	65	5.4.5.2 Bending about one axis	88
5.2.1.3 Bending with high shear	65	5.4.7 Bending and shear	90
5.2.1.4 Holes for fasteners	66	5.4.5.3 Holes for fasteners	88
5.2.2 Shear	66	5.4.6 Shear	89
5.2.3 Resistance of webs...	67	5.7 Resistance of webs...	117

Table 3.1. The location of clauses necessary for the design of restrained beams using EC3 and F-EC3

Whilst the easier identification of relevant material represents the most attractive feature of the proposed format, F-EC3 has other benefits:

- There is no need for annexes; since most information in the annexes relates to specific design tasks it is better suited to the main body of the text.
- The format used for F-EC3 is equally well suited to other Eurocodes.
- The restructuring process is made easier because codes are available on disc. Providing a master copy is available on file, re-structuring may be undertaken by the user working with his own computer. This enables individual organisations to develop F-EC3 in a way most suited to their own particular requirements.
- Most design clauses are relevant to only one design task. This results in less cross-referencing than might be imagined, since no attempt is made to change the content of the clauses.
- Generally speaking, clauses in Eurocode 3 are of a brief and specific nature. Many of them contain the formulae necessary for the quantitative evaluation of a particular design check. Clearly intelligent use of some of the more complex of these would be assisted if a greater explanation of the background, possible interpretations and limitations was available. Whilst F-EC3 represents only a rearrangement of the clauses contained in Eurocode 3, additional material of an

explanatory nature could readily be incorporated as an additional section within each design procedure, with its source and status clearly labelled.

- Sections are arranged in the same order followed during the design of buildings i.e. analysis of structures, member design, connection design, design of welds and fasteners, and finally fabrication and erection.
- Additional material can be included in the Code without affecting the time taken for designers to locate the information they require. This is a major benefit since the trend for codes to be continually revised and increased in size seems set to continue.
- The principle / application rule clause classification used for the Eurocodes gives organisations the opportunity to draft their own application rules. These rules could be incorporated into an in-house version of F-EC3 tailored to the organisation's particular requirements for use by less experienced colleagues. Code material of little use to the organisation could be deleted with certain material highlighted to improve the overall efficiency of the design process.

3.5 CONCLUSIONS

Figure 3.1 illustrates the extent to which codes have increased in size. This increase has been undertaken without changing the method used for arranging code material; i.e. clauses are arranged on the basis of the structural phenomena to which they relate. This arrangement was suited to relatively small documents such as BS449 but it now creates a commonly identified problem; how does the designer rapidly locate material relating to the design task undertaken from large, seemingly complex and unfamiliar codes? Fortunately codes can be made user-friendly. This can be done not by reducing the technical content but by changing the method used for arranging clauses. Providing clauses are arranged on the basis of design tasks, seemingly complex codes become user-friendly.

Chapter Four

HYPERTEXT CODES

4.1 INTRODUCTION

Electronic codes have been discussed since the 1970's, yet they are still not available in design offices. Will this situation change or will the perception that they are an over sophisticated method of replacing the paper document remain ?

At the present day it is unlikely that designers are likely to opt for electronic codes. Most firms of consulting engineers have well stocked libraries, hand calculations are still commonplace, and designers are well informed about the particular codes that they most commonly design to. Despite these difficulties we are moving into the digital age and it is likely that future design documents and codes will be stored and distributed electronically.

Low cost, powerful PC's have been available since the late 1980's and can easily store large libraries of documents. In order to compete in future markets the practices of consulting engineers will have to fully integrate computers into the design process. Designers will inevitably become at ease working in this digital environment, and it is these engineers that are likely to be receptive to the introduction of electronic codes. This is particularly so if they are available in a multi-document form, offering a wide range of documents at a lower cost than is presently available. The situation can easily be envisaged where a standard design library was purchased by practices of consultants, loaded on to each engineers PC, and from which relevant information is extracted when required and printed in paper form.

Before electronic codes become a practical option a method to rapidly locate information must be found. A method of navigating easily through electronic documents is now available. The method is called Hypertext and following the results of this research it has been found well suited to the task of creating practical electronic codes.

4.2 A DESCRIPTION OF HYPERTEXT

Hypertext is a means of quickly navigating through electronic documents. Information can be located quickly using search and indexing facilities and printed out or read directly from the screen. There are currently a number of commercial hypertext packages available. For the purpose of this research hypertext versions of EC3 and F-EC3 have been created using Lotus Smartext, which is Microsoft Windows compatible. Windows compatibility is considered important because it enables more than one application to be run at a time, with the freedom to switch between applications at will. For instance Hypertext, a structural design package and a word processing package can all be operated simultaneously.

Current trends towards hypertext documents lend themselves to "free text". "Free text" is so called because of the clipboard facility within Microsoft Windows. It is possible to copy text from a hypertext document onto the Windows clipboard, and thereafter to paste it into another piece of software such as a word processor for direct use or editing.

"Free equations" could become the norm of the future. Using this technique it would be possible to copy a "free equation" from a hypertext British or European standard and paste it into a spreadsheet or even into the new concept software TEDDS (a CIMSTEEL project) recently developed by CSC (UK) Ltd, which enables engineering calculations to be performed within a word processor. Thus engineering equations in the future could be used direct from their source with no room for error.

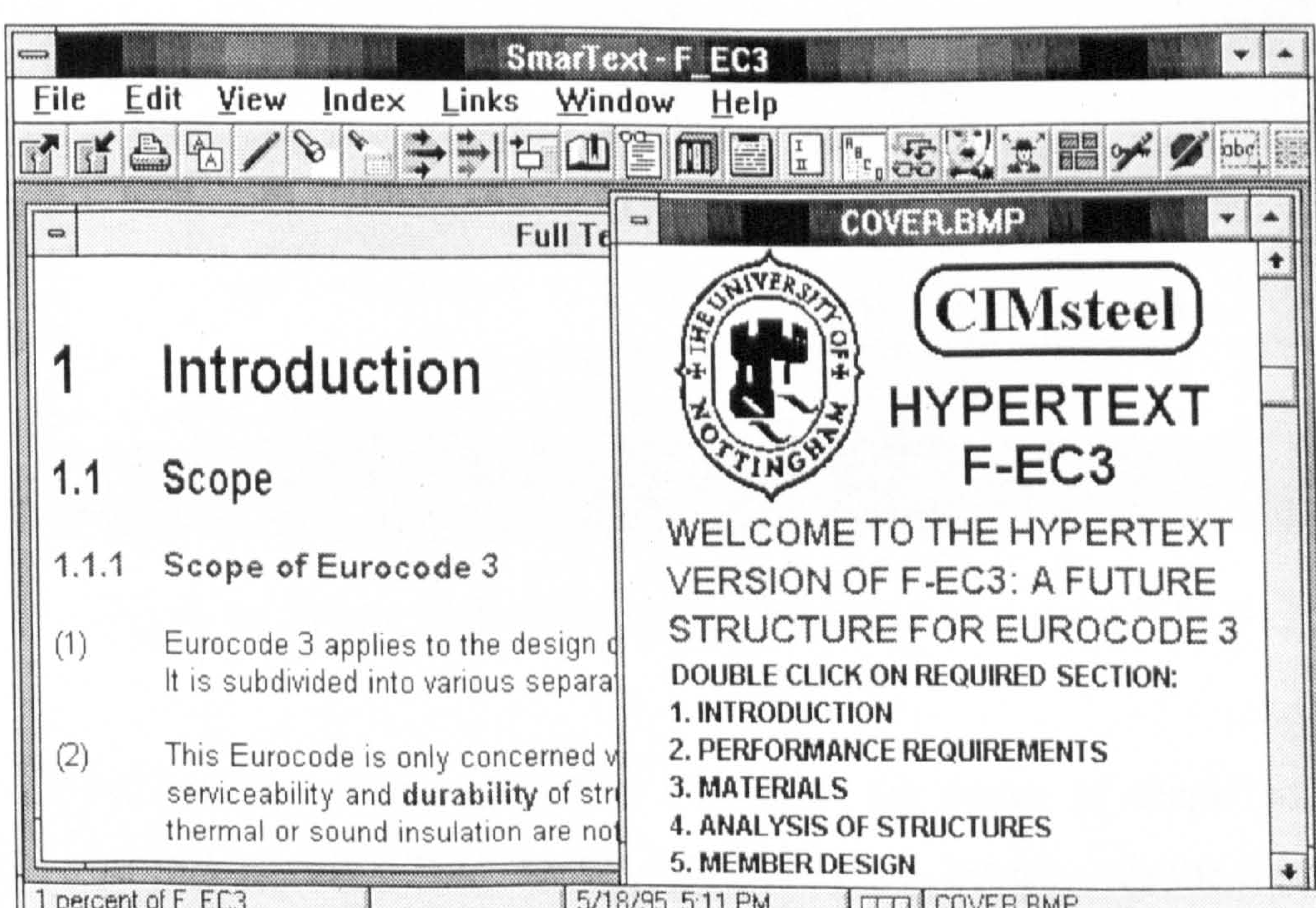


Fig. 4. 1: Typical screen shot of Hypertext F-EC3

In an attempt to illustrate the benefits of Hypertext codes, a series of screen shots taken from the Hypertext version of F-EC3 are given on the following pages. The shots show the method used by an engineer faced with the problem of locating all the material necessary for the design of a simple beam. The engineer using the conventional paper version of Eurocode 3 will need to study a large section of Code in order to identify the clauses relevant to this design task. Using the Hypertext version of F-EC3 this task can be performed in seconds.

Fig. 4. 1 shows the screen that greets the user when this hypertext code is activated. The central window entitled "COVER.BMP" has a click sensitive index. The user can move directly to the chapter of choice simply by clicking the cursor on the text of interest. The cover page is sitting on top of the document window. The text contained is that of Chapter 1. The user can move up and down in the document using the scroll bar. All references to clauses, chapters, figures and tables can be made simply by clicking the cursor on the reference contained within the text.

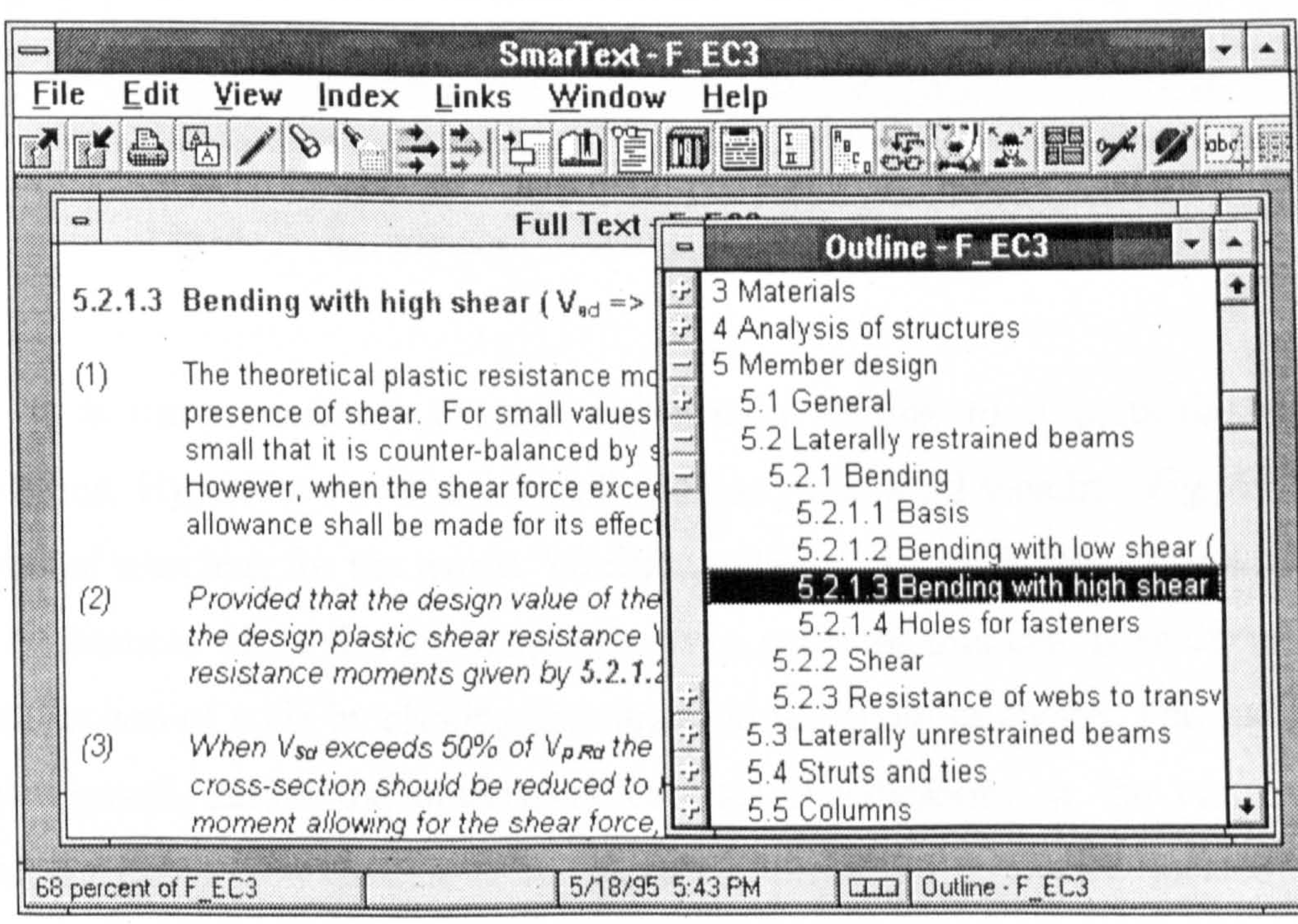


Fig. 4. 2: Screen shot showing the contents of Hypertext F-EC3

Since the user wants information relating to the design of simple beams, the document outline (contents) shown in Fig. 4. 2 is activated. Initially only the chapter titles are listed. Each chapter can be expanded by clicking the cursor on the + button located next to each line. In this instance Chapter 5: Member design has been expanded to reveal the subsections. Each sub-section can be expanded to reveal the individual clauses. In this example the user moves directly to the section relating to the design of restrained beams with high shear. The user clicks the curser on clause 5.2.1.3 within the contents window in order to locate the corresponding code material shown in the background window.

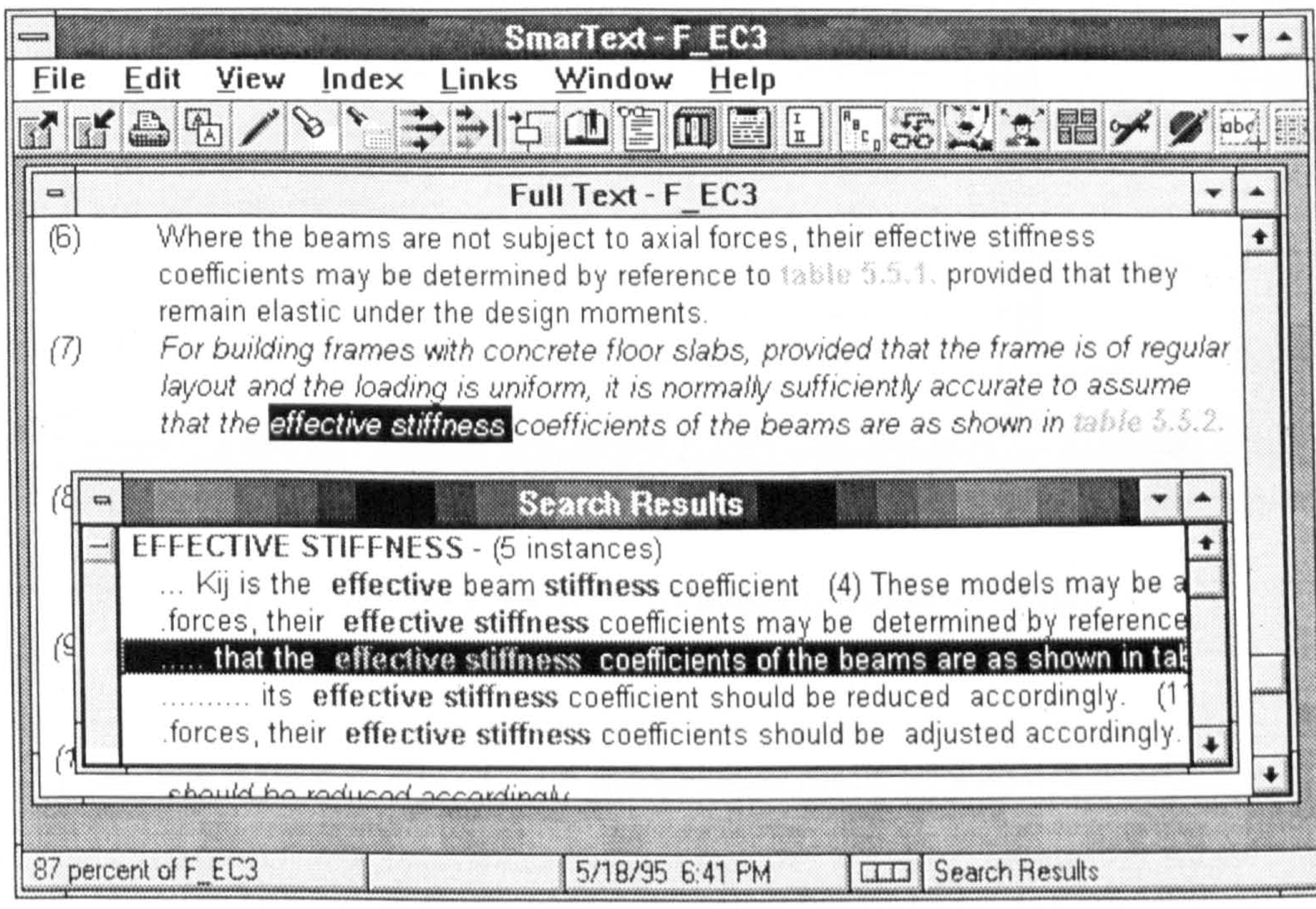
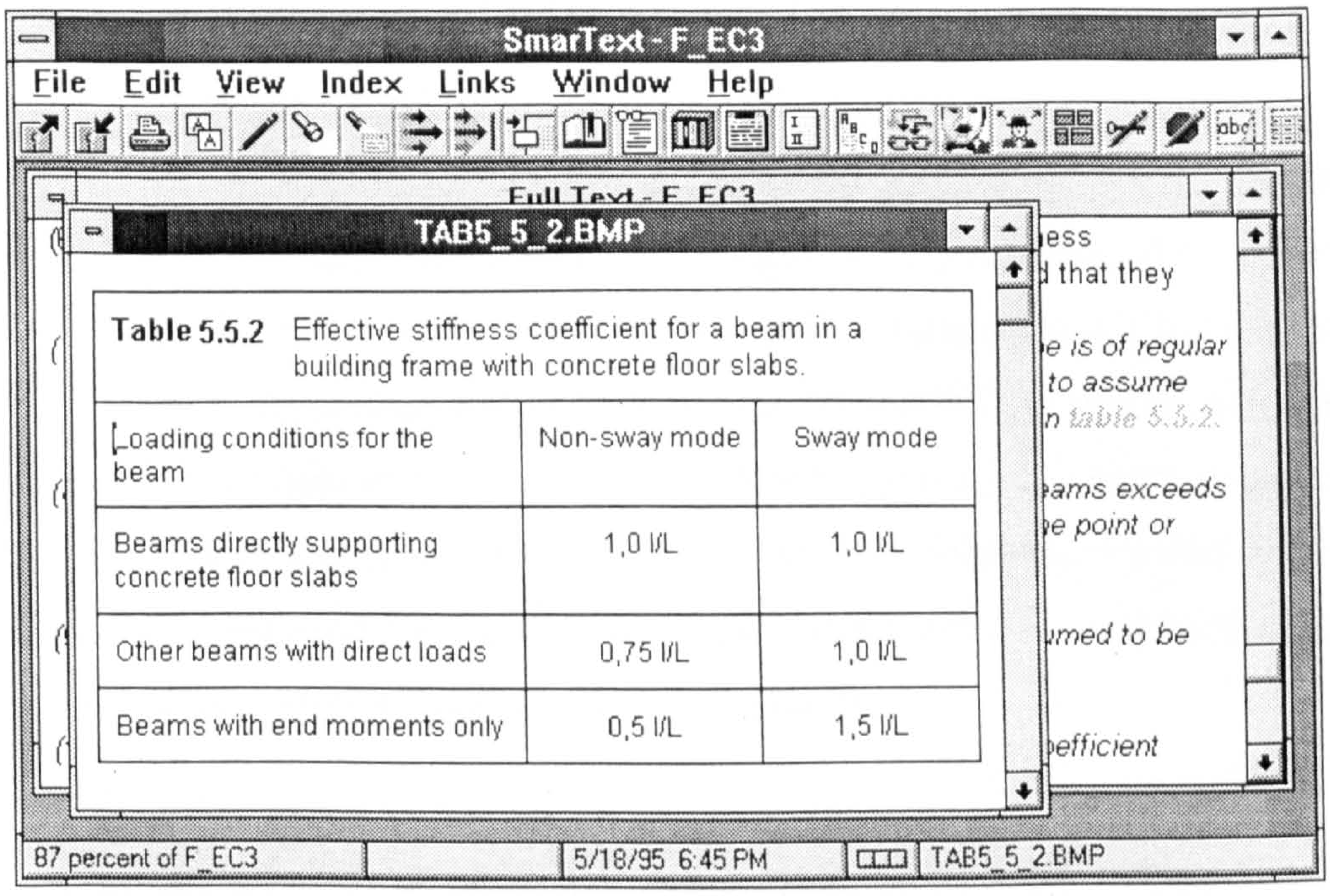


Fig. 4. 3. Screen shot showing Hypertext search facilities

It is often necessary to seek information relating to a particular structural phenomenon. Hypertext contains the facility to carry out word searches. Fig. 4. 3 shows the result of searching for the words "effective stiffness". Listed in the active window are all the 5 instances where this quote appears. Once again the user can move directly to the relevant section of code by clicking the curser on the quote of choice. This facility is of potential benefit during the drafting of contract specifications or the preparation of material for design submissions, since relevant material can be quickly located, cut out electronically and pasted into word processed documents. Similarly, design formulae can be cut out of the hypertext document and pasted into spreadsheet files. This may help alleviate the problem of the incorrect copying of the often complex equations that characterise modern structural design codes.



Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	1,0 I/L	1,0 I/L
Other beams with direct loads	0,75 I/L	1,0 I/L
Beams with end moments only	0,5 I/L	1,5 I/L

Fig. 4. 4: Screen shot showing a hypertext table.

In the text shown in Fig. 4. 3 a reference is made to Table 5.5.2. The reference is written in a different colour to ordinary text since the user can locate the table by clicking the curser on the reference, the result of which is illustrated in Fig. 4. 4.

Whilst it is not possible to change the content of the hypertext code, it is possible to make notes on the document. The status of these notes is clearly identified and they can be read by clicking the curser on the notes symbol contained within the text.

4.3 THE DESIGN LIBRARY

As a result of the complexity of modern codes, designers need increasingly to specialise in one particular material if they wish to fully utilise the potential of that material. With the availability of cheap, powerful computers it will not be long before most engineers have a PC from which most design will be conducted. The increased complexity of the codes will not noticeably affect the speed of design packages, and hopefully design of structures will become more economical with greater use being made of the more advanced models of structural behaviour contained within Eurocodes.

Of course there will always be the need for hand calculations. Small practices of consulting engineers may find it more economical to design unusual structural members by hand, rather than purchase the relevant software. More complex designs may be more appropriately designed by hand, with software used for checking.

The existing range of documents used in the design process is large. Finding the relevant piece of information can be time consuming. If the engineer has all the information at hand the desk will be crowded. For example, work conducted using EC4 (composite construction), must be accompanied by EC2 (concrete) and EC3. It would seem a natural progression with the transfer to a more computerised approach to design, together with the modern hypertext packages available, for the relevant documents required in design to be integrated into a design library, that may easily be loaded on to the hard disk of the engineers PC, or alternatively onto the company network.

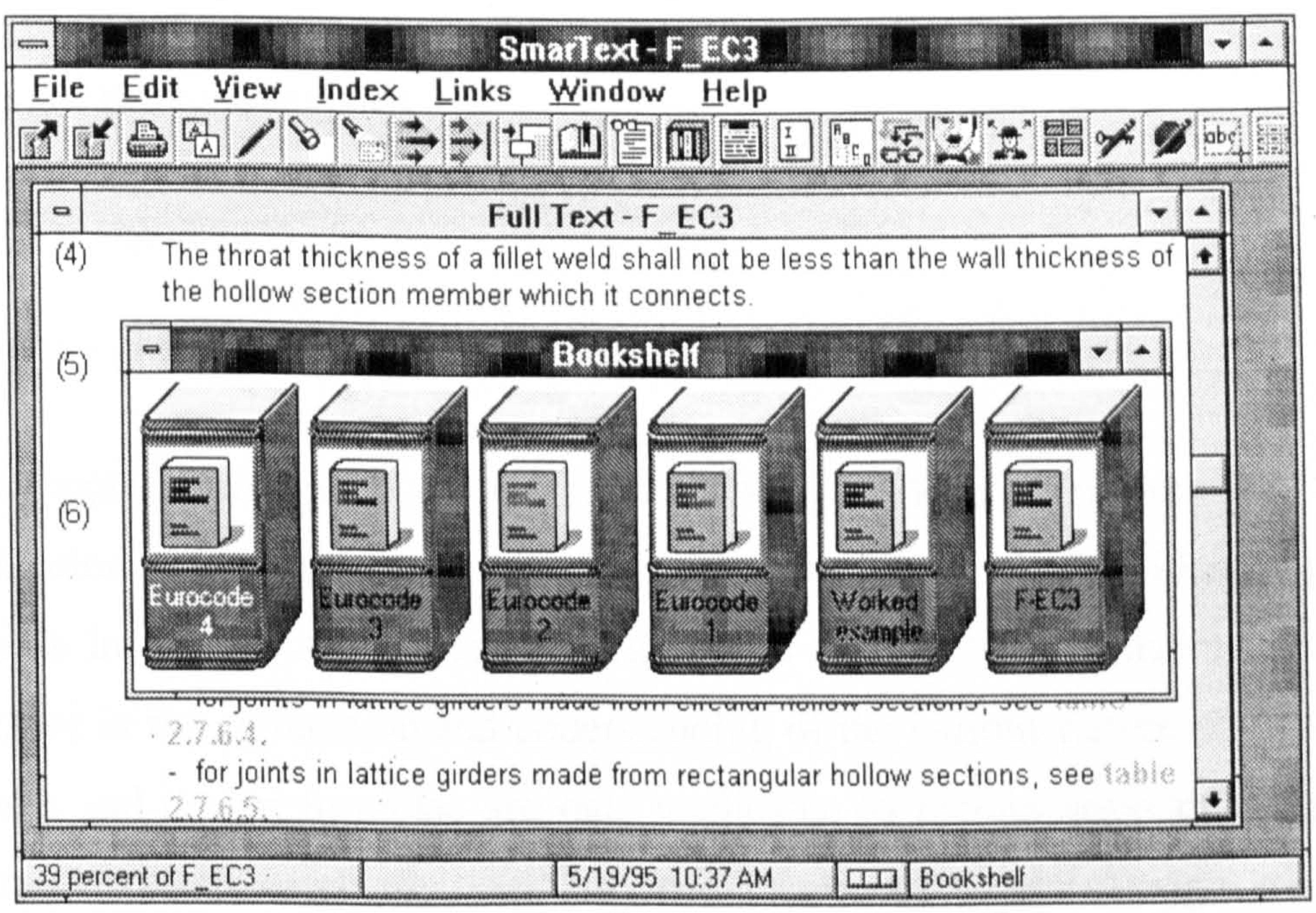


Fig. 4. 5: The design library

A standard design library could look similar to the window shown in Fig. 4. 5. This is the "book case" that greets the user when hypertext is accessed. Any document can be opened simply by clicking the curser on the book of choice.

A comprehensive library of design documents could be hypertexted and read or distributed using the Internet. This would give everyone access to all the codes, providing they have a PC, Modem and telephone. Undoubtedly this will prove useful to the increasing number of engineers working part of the time from home and those engineers stationed overseas. The systems that make this technology possible are now well established, but they are not as yet widely used by the construction industry.

4.4 SUITABLE MATERIAL FOR INCLUSION IN A DESIGN LIBRARY

Listed on the following pages is some of the information that could ultimately be included in a standard design library.

- A full range of Eurocodes. Although all the codes would not be used at once referral between codes is common due to the CEN regulation that the duplication of information in codes is prohibited. References to EC2 and EC3 from EC4 are particularly common. Theoretically, references could be made instantaneously using hypertext.

- A full set of reorganised Eurocodes along the line of F-EC3.

- Relevant British Standards.

- Building regulations.

- A full list of National Application Documents.

- Background information to clarify and provide detailed information on the application and limitations of the individual design clauses and to identify possible hazards in design. At present the Eurocodes give little information to aid the engineer in the application and understanding of the various design clauses. Many clauses and design formulae are only applicable to certain areas of design. The limitations of these clauses are described in various papers, design guides and more particularly the actual background documents of the Eurocodes which describe the basis of the various design formulae. All this information is extremely detailed but a slimmed down version of information accessed using a hypertext system may prove valuable to the designer faced with more unusual and complex problems.

- Design examples. A series of detailed design examples have been developed by organisations such as the SCI and ECCS in order to help the engineer through the maze of complex clauses contained in the codes.

- Design aids such as charts, tables and graphs. Many previous codes disguised their rather complex models of structural behaviour by providing design information in the form of charts, tables and graphs. The designer simply calculated some basic variables and read off the result from a table or graph. Eurocodes have opted away from this system, providing the raw and sometimes extremely complex design formulae. A series of these design aids will prove invaluable to the small consultancy practice involved in hand calculations, as complex calculations may be avoided enabling rapid analysis. This is particularly important in view of the fact that design is an iterative process using for the most part rapid and approximate calculations.

- Basic design tables such as the SCI 'Blue Book', structural flooring design charts, re-bar tables, unit weight tables, etc.
- National structural steelwork specification along with similar documents for other materials.
- Flow charts that indicate simplified if perhaps slightly conservative methods of design.
- The ESDEP course of lecture notes.

4.5 CONCLUSIONS

Using the latest software the old perception that electronic codes are an overcomplicated and unnecessary replacement of paper documents may begin to change. Listed briefly below are some of the advantages electronic codes offer:

- Formulas can be cut out of hypertext codes and pasted into other applications, such as Microsoft Word (word processor), MS Excel (spread sheet) or CSC's TEDDS software. This offers a significant advance as complicated equations often get incorrectly copied onto spreadsheet type programs, a problem that hypertext has the potential to eliminate.
- Specifications can be written faster. Relevant information can be located using search facilities and pasted into the specification.

- Hypertext documents can be distributed or read using Internet. This could prove particularly valuable to engineers working in foreign countries or from home.

- Once hypertext codes are 'built', changes to their content are not possible. This will safeguard against the possible deletion of clauses. Notes can be made on the hypertext documents, though the status of these notes is clearly indicated.

- Hypertext codes are of low cost in memory terms. EC3 take less than 2 megabytes of memory.

- Quality assurance problems of ensuring engineers use up-to-date codes are alleviated. Revised codes can be re-hypertexted, distributed and re-installed onto networks or PC's.

Given these significant advantages and the user-friendly nature of windows compatible software, it is likely that a market for electronic codes will exist when Eurocodes move from the ENV to the full EN status. Prior to this change in status widespread use of the Eurocodes is unlikely.

Chapter Five

THE CALIBRATION OF PARTIAL SAFETY FACTORS

5.1 INTRODUCTION

The relationship between applied loading S and the resistance R of a component in the limit states approach to structural design is conventionally expressed as:

$$\gamma_S . s_k \leq r_k / \gamma_R \qquad (5.1)$$

in which:

s_k is the characteristic load

r_k is the characteristic resistance

γ_S, γ_R are partial safety factors

Characteristic values are representative figures based on statistics e.g. knowledge of the means and standard deviations, whilst values for the γ's are normally based on a combination of calibration and judgement.

One of the most important works on the subject of calibration (CIRIA, 1977) contains detailed descriptions of the various levels of approach that are possible. The particular technique used for calibrating the Structural Eurocodes (Brozzetti and Janss, 1992) - in particular the resistance expressions of Eurocode 3: Design of steel structures (CEN, 1993) - is largely based on work originally undertaken in the Netherlands (Bijlaard *et al*, 1988). This method is described in some detail in Annex Z of Eurocode 3; it will thus be referred to as the Annex Z method.

Close examination of the Annex Z method by the author during work to investigate the relative safety levels of structural design in accordance with Eurocode 3 (Nethercot and Byfield, 1993) highlighted certain aspects where improvements were possible. An alternative approach that simplifies the seemingly complex procedure has been devised and is reported herein.

5.2 THE CALIBRATION OF γ_R

5.2.1 THE ANNEX Z METHOD

The objective of calibration is to provide a scientific basis for selecting values for the γ-factors that ensure a given (or target) level of confidence in achieving safe design; i.e. the probability of (R-S) < 0 is suitably small. If the statistical distributions of R and S are known as illustrated in Fig. 1, then Pr [(R-S) < 0] may be represented in terms of the safety index β, where β is the number of standard deviations of the distribution of (R-S) between the average of (R-S) and the origin as illustrated in Fig. 5. 1.

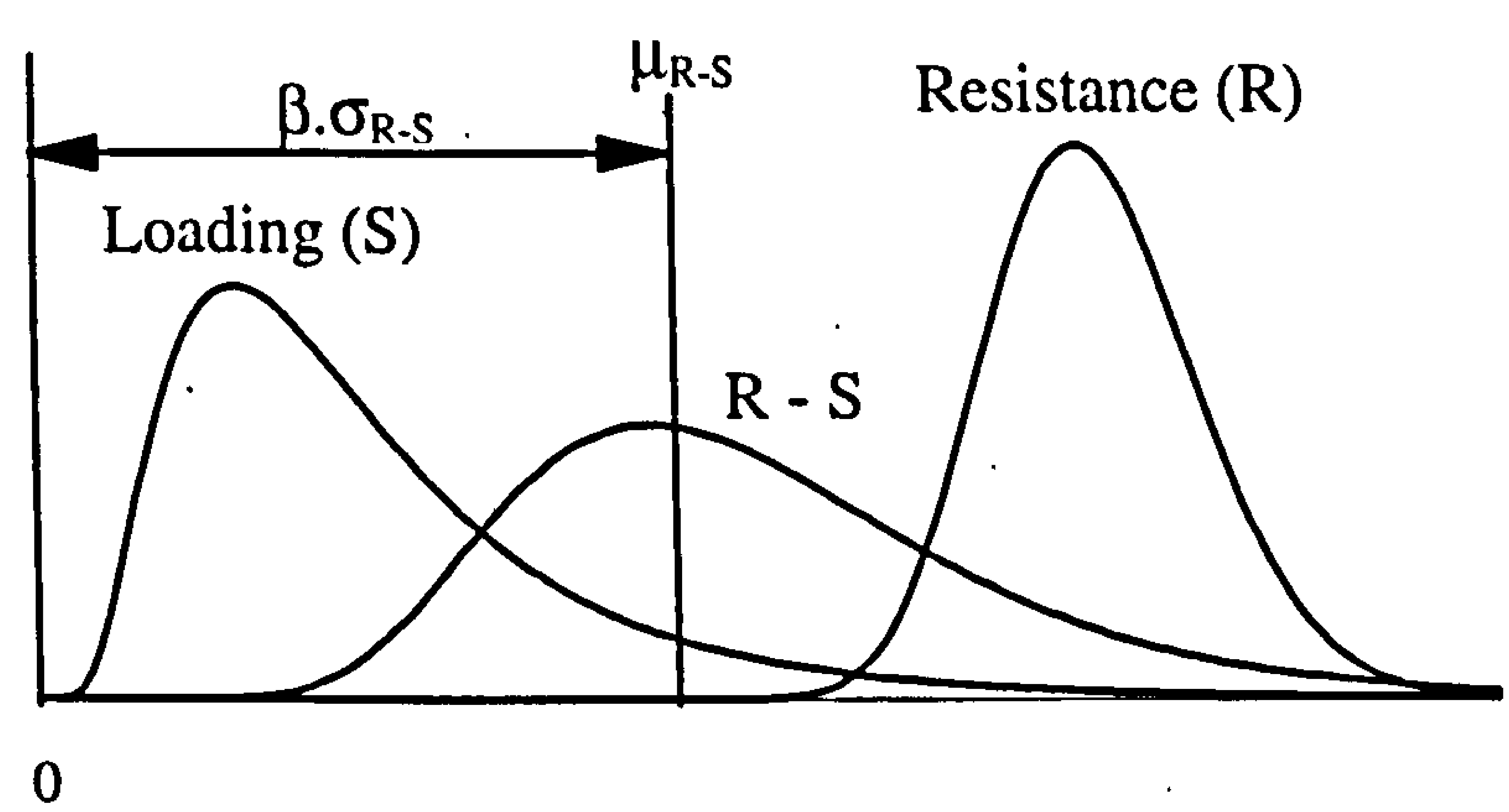


Fig. 5. 1: Assumed distributions of R, S and R-S

The logarithmic normal probability distribution function, henceforth referred to as the log-normal p.d.f. is used to model the probability distribution of resistance. Basic geometric and material properties are also assumed to be log-normally distributed (CEN, 1993a). The use of log-normal distribution has the advantage that it will not produce negative values; a characteristic that is correct for geometric properties and material strengths. It should be noted that the differences between log-normal and normal distributions are only noticeable where the lower tail of the distributions is near the origin. This effect is illustrated by the log-normal p.d.f.s sketched in Fig. 5. 1; the degree of skewness is increased as the lower tails of the distributions approach the origin. It should be noted that the distribution of R-S cannot in fact be log-normally distributed, since log-normal distribution does not produce negative values.

The Annex Z method is based on the assumption that resistance calculated using nominal values of basic variables will be achieved by 95% of constructed steel. This resistance is termed the characteristic resistance r_k and is defined using the following expression:

$$r_k = \mu_r \exp(-0.5V_r^2 - kV_r) \qquad (5.\ 2)$$

where:

V_r is the coefficient of variation of resistance.

k is 1.645 (the number of standard deviations between μ_r and r_k, see *Fig. 5. 2*)

The mean resistance μ_r is defined as:

$$\mu_r \cong \bar{b}.r_m \qquad (5.\ 3)$$

where:

r_m is the resistance calculated using mean values of basic variables.

$\bar{b}$ is the correction factor; this is a measure of any difference between experimental and predicted resistances; i.e, a $\bar{b}$ of 1.10 represents a resistance function that on average underestimates resistance by 10%.

Design resistance is assumed to be achieved by 844 in 845 samples. This target probability is derived on the basis that resistance is log-normal and design resistance is located 3.04 standard deviations from the mean, see Fig. 5. 2. Design resistance is defined by the following expression:

$$r_d = \exp(\ln\mu_r - 0.5V_r^2 - \alpha_R\beta V_r) \qquad (5.\ 4)$$

where:

$$\alpha_R\beta = 0.8x3.8 = 3.04 \qquad (5.\ 5)$$

since,

$$\gamma_R = \frac{r_k}{r_d} \qquad (5.\ 6)$$

then substituting equations (5. 2), (5. 3) and (5. 4) into equation (5. 6) gives:

$$\gamma_R = \frac{\bar{b}.r_m.\exp(-0.5V_r^2 - 1.64V_r)}{\bar{b}.r_m.\exp(-0.5V_r^2 - 3.04V_r)} \qquad (5.\ 7)$$

As the expressions defining r_d and r_k are almost identical, most of the terms shown in equation (5. 7) cancel, including $\bar{b}$, producing the restricted equation given below:

$$\gamma_R = \exp(1.4V_r) \qquad (5.\ 8)$$

Equation (5. 8) is ultimately used for calculating γ_R according to the Annex Z method. A slight modification is made to account for sample size using Student's t-distribution. The reliability index β is a relative measure of design reliability; its calculation is extensively discussed in (Galambos *et al.* 1982) and (Ravindra and Galambos 1978).

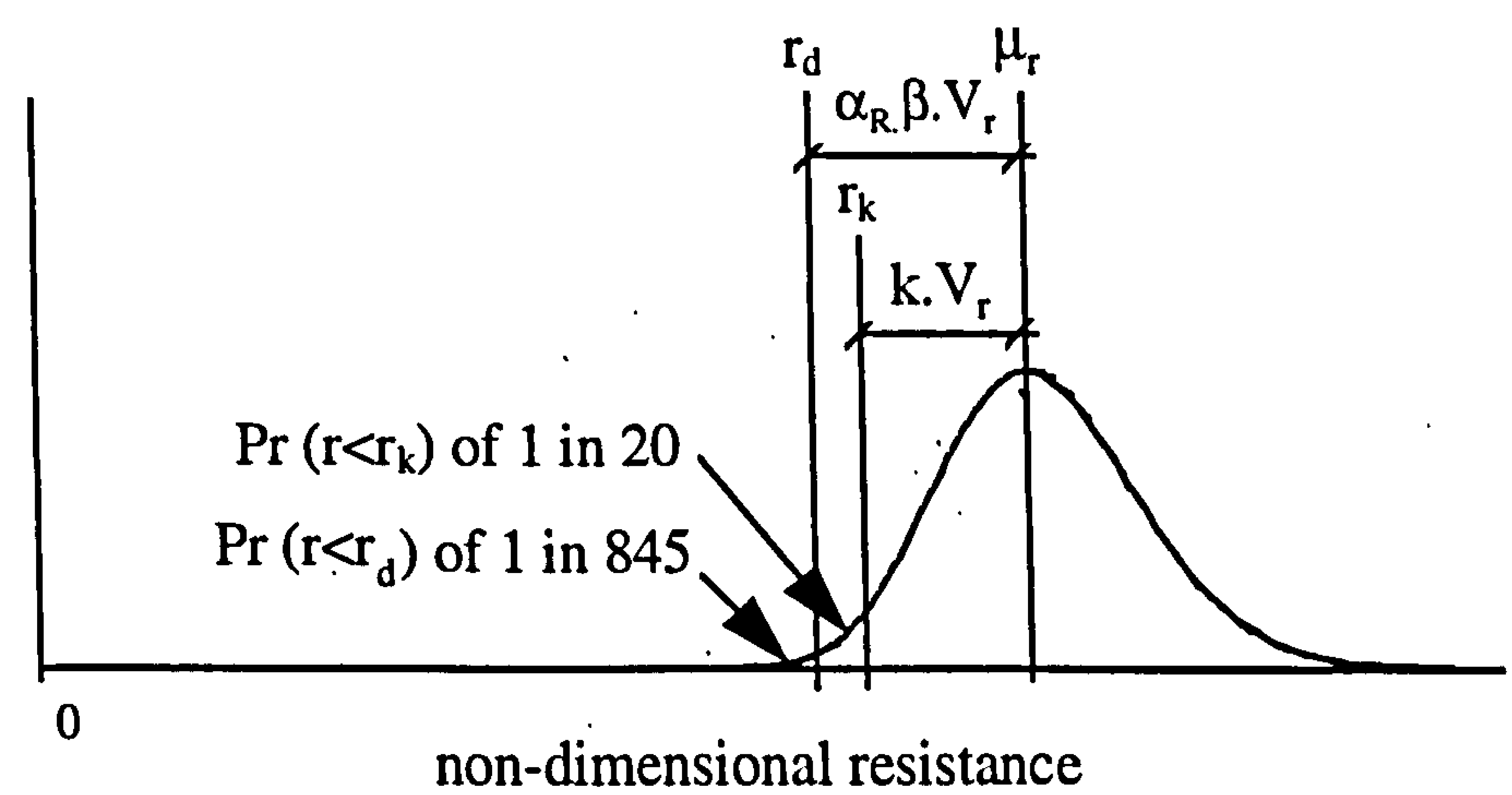


Fig. 5. 2: The physical basis of design resistance and characteristic resistance assumed by the Annex Z method

5.2.2 AN ALTERNATIVE METHOD

The difference between the established method of section 5.2.1 and the alternative method proposed herein is in an assumption concerning r_k. The alternative method concentrates on the relationship between r_d and r_n. Nominal resistance is not assumed to relate to any particular probability of failure. As with the Annex Z method, r_d is calculated using the same rules for log-normal variables to produce the following:

$$r_d = \exp(\ln \mu_r - 0.5V_r^2 - t.V_r) \qquad (5.\ 9)$$

since,

$$\gamma_R = \frac{r_n}{r_d} \qquad (5.\ 10)$$

and
$$\mu_r \cong \bar{b}.r_m \tag{5. 11}$$

Then by substituting equations (5. 9) and (5. 11) into (5. 10) and rearranging gives:

$$\gamma_R = \frac{r_n.\exp(0.5V_r^2 + t.V_r)}{\bar{b}.r_m} \tag{5. 12}$$

if n>150 then
$$\gamma_R = \frac{r_n \exp(0.5V_r^2 + 3.04V_r)}{\bar{b}r_m} \tag{5. 13}$$

The same principles can be applied if resistance is shown to be a normally distributed variable to give equation (5. 14) below:

$$\gamma_R = \frac{r_n}{\bar{b}.(r_m - t.\sigma_r)} \tag{5. 14}$$

n	5	10	15	20	25	30	40	60	80	100	150	150+
t	5.67	4.04	3.65	3.48	3.38	3.32	3.25	3.17	3.14	3.12	3.09	3.04

Table 5. 1: Values from Student's t-distribution

Factor t is taken from Table 5. 1. These values are calculated using Student's t-distribution and correspond to a probability of $r<r_d$ of 1 in 845. Student's t-distribution is a method for accounting for the additional uncertainty due to a small sample size.

5.2.3 COMPARISON BETWEEN METHODS

The equations used for calculating γ_R vary considerably between the Annex Z and alternative method reported herein. An important question concerns the way in which these differences will affect the γ_R-value. This may best be appreciated by means of an example and an illustration using restrained beams has been selected. For this the governing design expression is:

$$M_{pl.Rd} = W_{pl.y}\, f_y \;/\, \gamma_R \tag{5. 15}$$

where:

$M_{pl.Rd}$ is the plastic moment of resistance

W_{pl} is the plastic section modulus

f_y is the yield stress

For the purpose of this example 18 bending test results from (Hasan and Hancock, 1988) will be used. These tests were conducted on cold-formed SHS and the results are summarised in Table 5. 2.

	D (depth) measured /nominal	B (width) measured /nominal	t (thickness) measured /nominal	f_y measured /nominal	$M_{pl.Rd}$ experimental /predicted
st.dev	0.007	0.006	0.024	0.093	0.076
Mean	0.998	1.004	1.017	1.068	1.205
COV	0.007	0.006	0.023	0.087	0.063

Table 5. 2: Summary of Hasan and Hancock's bending test results

Applying the figures contained in Table 5. 2 to the calibration of a 254x254x9.5SHS gives a member with the statistical variability listed in Table 5. 3.

	h (depth) mm	B (width) mm	t (thickness) mm	f_y N/mm^2	$M_{pl.Rd}$ kN.m
nominal	254.0	254.0	9.5	350.0	284.7
mean	253.5	255.0	9.7	373.9	308.7
st.dev	1.791	1.448	0.225	32.646	N/A

Table 5. 3: Data used for basic variables

Equation (5.16) represents an established method for calculating standard deviation of the resistance σ_r. Alternative methods are available such as those described in (Thoft-Christensen and Baker, 1982), though equation (5.16) is of sufficient accuracy for this calibration example.

$$\sigma_r^2 = (\sigma_b . r_m)^2 + \left(\frac{\delta r_m}{\delta h}\sigma_h\right)^2 + \left(\frac{\delta r_m}{\delta B}\sigma_B\right)^2 + \left(\frac{\delta r_m}{\delta t}\sigma_t\right)^2 + \left(\frac{\delta r_m}{\delta f_y}\sigma_{f_y}\right)^2 \qquad (5.16)$$

Applying the basic variable data listed in Table 5. 3 to equation (5.16) gives:

$$\sigma_r^2 = (0.076\text{x}308.7)^2 + \left(\frac{4.34}{2.5}1.791\right)^2 + \left(\frac{2.25}{2.6}1.448\right)^2 + \left(\frac{2.68}{0.1}0.225\right)^2 + \left(\frac{3.09}{3.7}32.646\right)^2$$

$\sigma_r = 36.37$ kN.m

$$V_r = \frac{\sigma_r}{r_m} = \frac{36.37}{308.7} = 0.12 \qquad (5.17)$$

Calculating γ_R using the Annex Z method gives:

$$\gamma_R = \exp(1.4x0.12) = 1.18 \qquad (5.18)$$

By comparison using the alternative method:

$$\gamma_R = \frac{284.7\exp(0.5x0.12^2 + 3.04x0.12)}{1.205x308.7} = 1.10 \qquad (5.19)$$

The alternative method has produced a significantly lower value for γ_R. The main reason for this is that the resistance function on average underestimates resistance by 20.5% ($\overline{b}$=1.205). This margin of safety is recognised by a reduction in γ_R with the alternative method. For reasons discussed previously, the Annex Z method is insensitive to this effect.

In an attempt to understand more about the differences between the methods, the previous example has been repeated 3 times below, each time with one of the following parameters varied:

- mean value of yield stress
- correction factor $\overline{b}$
- standard deviation of observed error terms σ_b

As illustrated by Fig. 5. 3, the Annex Z method is insensitive to the relationship between the nominal and mean values of basic variables. As with many materials, steelwork has a significantly higher mean yield stress than the nominal value assumed in design. The effect of this is an inherent safety margin which is reflected in a lower value of γ_R for the same target reliability. This calibration example is unusual because the nominal yield stress (350 N/mm^2) occurs less than one standard deviation (33 N/mm^2) from the mean (374 N/mm^2), see Table 5. 3. Typically for European hot rolled steel the nominal yield stress occurs two standard deviations from the mean (Nethercot and Byfield, 1993). If that were the case in this calibration example, i.e. mean yield stress was 440 N/mm^2, then γ_R would have required a value of less than 1.0 for the same target reliability, using the alternative method. In this case the inherent margin on material strength would effectively more than cover the required factor on resistances.

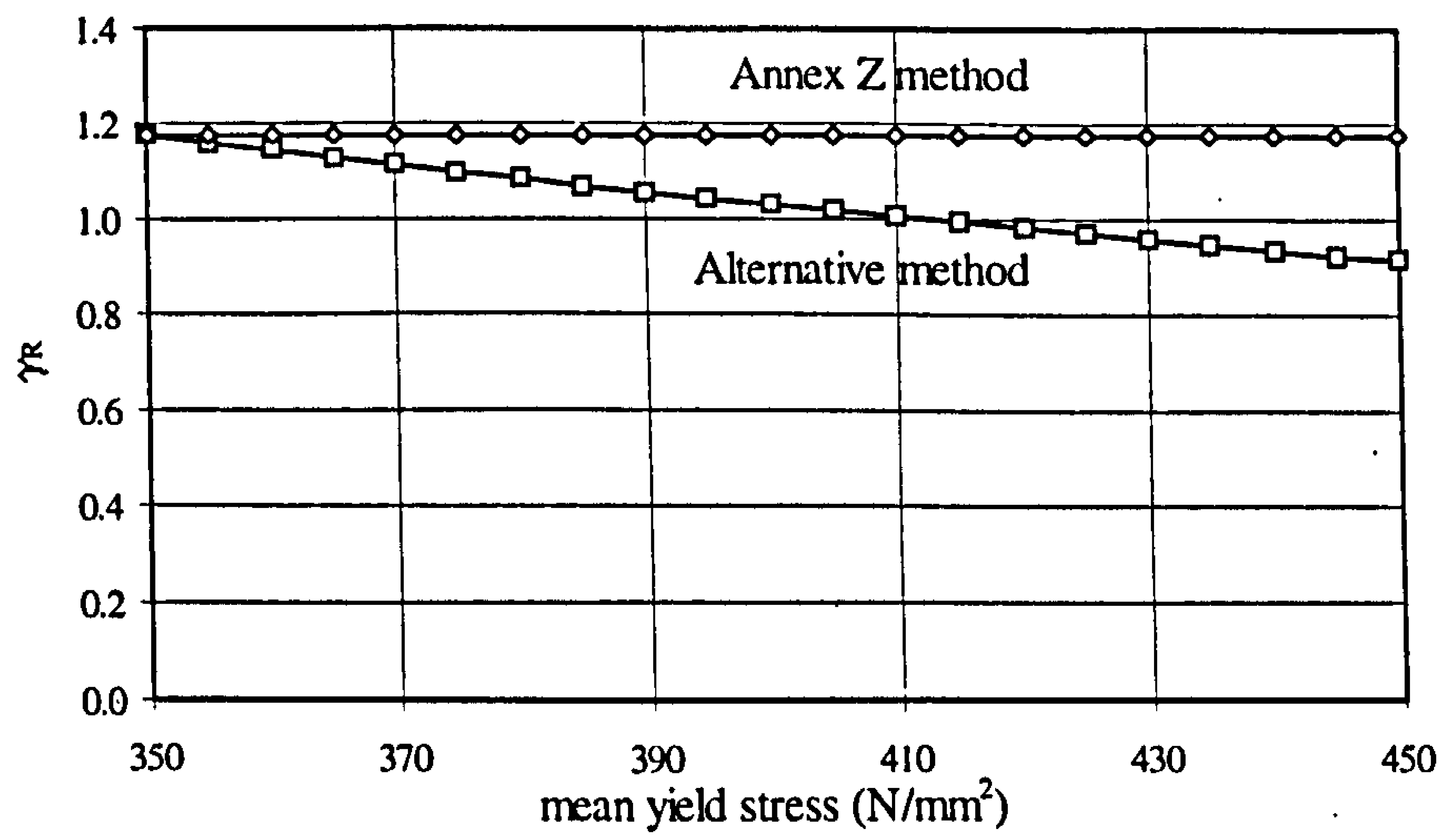


Fig. 5. 3: Effect of variations in mean yield stress

Fig. 5. 4 shows the effect of varying the correction factor $\bar{b}$ ($\bar{b}$ of 1.10 represents a resistance function that on average underestimates resistance by 10%). With the alternative method, γ_R reduces if the resistance function is underestimating strength. The reason for this is clear, if $\bar{b}>1$ then the resistance function contains in effect its own degree of safety, reducing the value of γ_R required to achieve the target reliability specified for Eurocode 3.

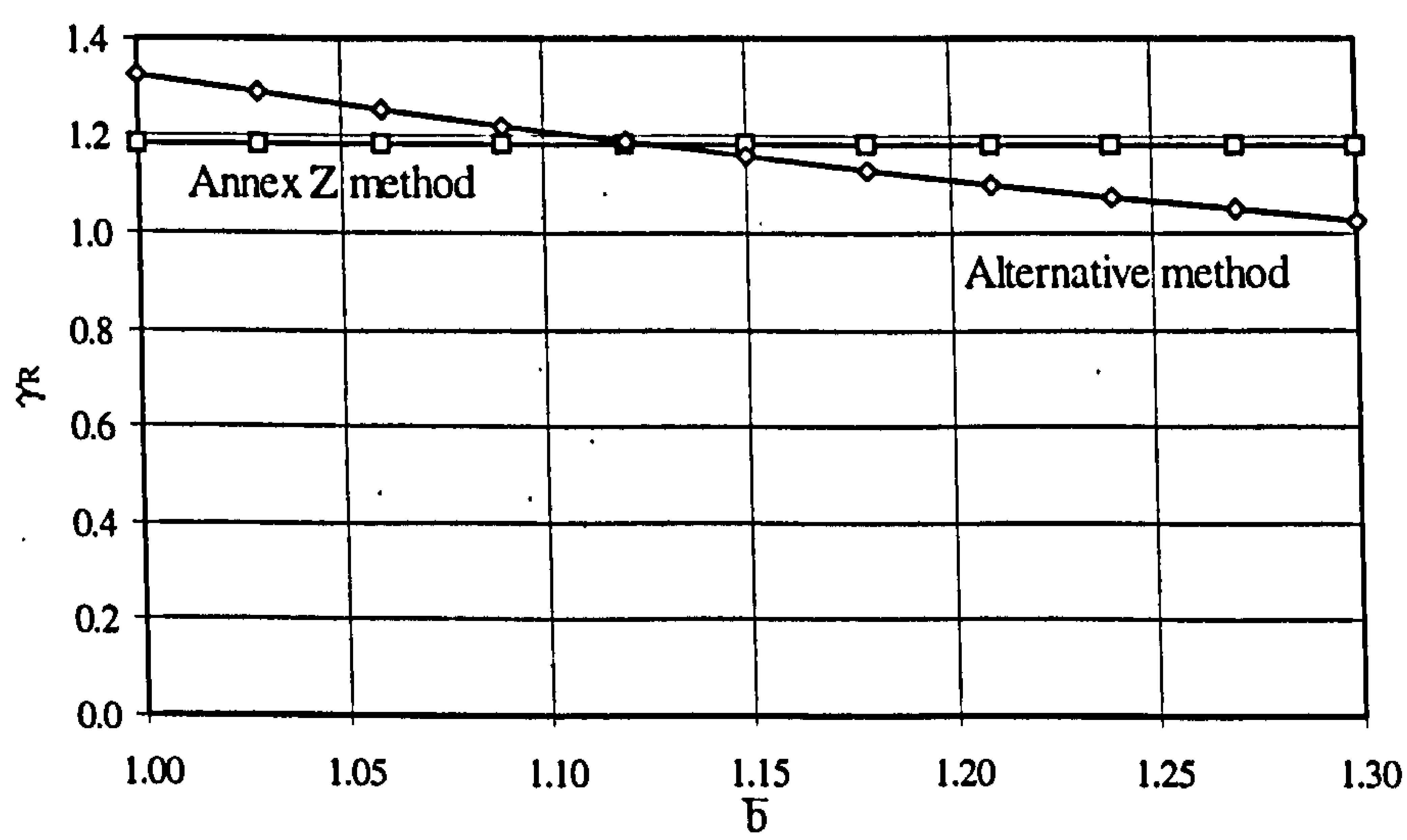


Fig. 5. 4: Effect of varying $\bar{b}$

Fig. 5. 5 illustrates the effect of varying σ_b, the measure of scatter between predicted and experimental resistances. Both methods produce the same trends in this graph, with the difference being the greater sensitivity exhibited by the alternative method.

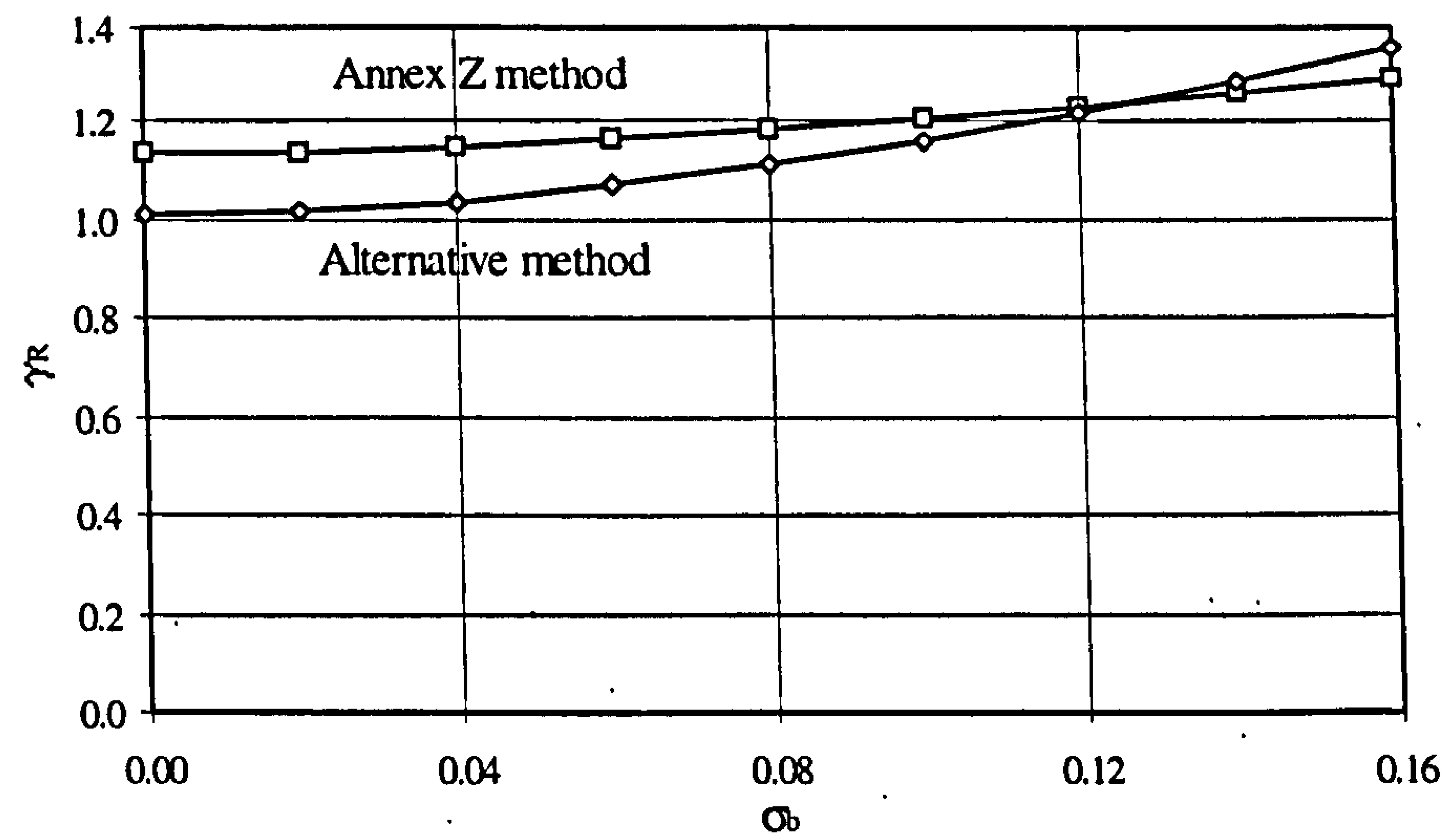


Fig. 5. 5: Effect of varying σ_b

5.3 THE INTRODUCTION OF γ_R^*

Section 5.2 demonstrates that the original Annex Z method produces a γ_R-factor insensitive to certain key effects. This insensitivity has been overcome through the introduction of the factor γ_R* which replaces γ_R; where γ_R* is equal to γ_R multiplied by a modification factor termed k_c. Factor γ_R* is thus derived from the following:

$$\gamma_R^* = k_c \cdot \gamma_R \qquad (5.20)$$

$$k_c = \frac{r_n}{r_k} \qquad (5.21)$$

since

$$\gamma_R = \frac{r_k}{r_d} \qquad (5.22)$$

inputting equations (5. 21) and (5. 22) into (5. 20) gives:

$$\gamma_R^* = \frac{r_n}{r_k} \cdot \frac{r_k}{r_d} = \frac{r_n}{r_d} \qquad (5.\ 23)$$

Ignoring the effect of sample size and using the definition of design resistance given in equation (5. 4)

$$\gamma_R^* = \frac{r_n \exp(0.5V_r^2 + 3.04V_r)}{\bar{b}r_m} \qquad (5.\ 24)$$

Thus γ_R* gives the identical solution to the alternative method, see equation (5. 13), although by a somewhat indirect method. The γ_R* factor does not appear in (Bijlaard *et al*, 1988), which is the method on which the Annex Z method (CEN, 1993a) has been based.

5.4 CONCLUSIONS

The Annex Z method for calibrating γ_R-factors assumes characteristic resistance is achieved by 95% of samples. Since the method defines γ_R as being equal to the characteristic resistance divided by design resistance, and the equations defining both are similar, then most of the terms cancel. The result is a restricted expression for γ_R that is insensitive to certain key effects. Under what may be considered as normal conditions, the Annex Z method will produce conservative values for γ_R.

This chapter demonstrates that the basic Annex Z method can be improved by defining γ_R directly as being equal to the nominal resistance divided by design resistance. Nominal resistance is equal to resistance calculated using nominal values of basic variables. It does not correspond to a target probability. The logic behind the resulting method is transparent and it involves less assumption; in addition, the method simplifies a seemingly complex procedure and produces more economical results.

The shortcomings of the Annex Z method have been addressed through the introduction of γ_R*; where γ_R* is equal to γ_R multiplied by a modification factor known as k_c. Factor k_c may be regarded as a convenient method for reintroducing the cancelled terms. It therefore produces the same result as the alternative technique proposed herein. Unfortunately the logic behind the method Annex Z method becomes unclear, it is unnecessarily complex and involves the assumption that resistance calculated using characteristic values of basic variables corresponds to a target probability.

The work undertaken herein was based on (Bijlaard *et al*, 1988); a document regarded as the original Annex Z method. No reference was made to γ_R* in that document; thus, the alternative technique for calculating γ_R proposed herein was devised in order to overcome the shortcomings of that method.

Chapter Six

THE VARIABILITY OF MATERIAL AND GEOMETRIC PROPERTIES

6.1 INTRODUCTION

Before the numerical value of γ_R^* can be determined the statistical variability of resistance must first be quantified; this is a function of three types of variability:

- material strength variability
- geometric variability
- resistance function inaccuracy

It is the objective of this chapter to quantify the variability of both material strength and the geometric properties for commercial quality structural steelwork; in particular, universal column and universal beam members. These uncertainties have been quantified herein through a statistical analysis that utilises steel producers own quality control measurements. Since the data used has been provided on a confidential basis, the identity of the steel producers has been omitted. Rather, they are identified by the letters A or B. It may be noted that A and B are major steel producers located within the European Union. A total of 7660 mill test results have been used to quantify the variability of material properties. Out of these mill tests, 689 are accompanied by geometric measurements. This data has been used for quantifying geometric variability.

During the design process, design calculations are based on the nominal material and geometric properties specified by manufacturers; who in turn manufacture sections in accordance with product standards that include: EN 10025 (CEN, 1990), BS 4360 (BSI, 1990) and BS 4 (BSI, 1980). Since probability of failure is influenced by the requirements specified in these standards, γ_R^*-factors are calibrated using measures of material variability determined partly on the basis of the tolerance limits specified in the product standards. Whilst this is a logical basis for the calculation of γ_R^*, the target reliability specified for EC3 will only be achieved if manufactured steel is of a sufficiently high quality. If producers manufacture steel to a higher quality than that necessary to comply with the relevant product standards, then the target reliability may be substantially exceeded. If that were the case, then γ_R^*-factors could be reduced, providing the relevant product standards were adjusted accordingly.

From examination of the Annex Z method (CEN, 1993a) and the Eurocode 3 Background Documentation e.g. (Sedlacek *et al*, 1989), it would appear that the γ_R^*-factors contained in EC3 have been determined using the following assumptions concerning the variability of material and geometric properties:

1. all the basic variables of the resistance function are considered approximate to the log-normal distribution;
2. no correlation (statistical dependence) exists between the basic variables of the strength function;
3. geometric properties have a mean value equal to the nominal value specified for the purpose of design;
4. the nominal value of yield strength is a characteristic value, i.e. a 95% confidence limit;
5. the coefficients of variation for basic variables approximate to the values listed in Table 6.1.

Basic variable	Coefficient of variation
yield stress, f_y	0.07
area, A	0.03
moment of inertia, I_y	0.03
torsional inertia, I_t	0.03
warping inertia, I_w	0.03

Table 6.1: Estimates of COV used for calibration EC3

From inspection of the background documentation it would appear that the values of COV listed in Table 6.1 are based on work originally undertaken during the 1970's (Alpsten, 1977). The work reported herein will examine the extent to which these estimates of material variation are valid today - given that manufacturing methods are likely to have improved in the past 15 to 20 years.

6.2 THE VARIABILITY OF MATERIAL PROPERTIES

6.2.1 THE INFLUENCE NOMINAL YIELD STRESS HAS ON RELIABILITY

In 1993 the Department of the Environment commissioned a study to evaluate the relative safety levels of designs in accordance with the Eurocodes. The overall objective was to determine the suitability of the γ_R^*-factors. Leeds University carried out the review of EC2, Nottingham University reviewed EC3 and EC4 was reviewed at Warwick University.

Work undertaken to evaluate the relative safety level of steel design was carried out by the author. This work is not reported in detail herein, since the study was brief and completed within 3 months. Despite this an important finding was outlined in the final report (Nethercot and Byfield, 1993). Analysis clearly showed that material properties have the largest influence on the safety index of structural steelwork, where the safety index β is equal to the number of standard deviations between mean resistance and design resistance. The β-factor is a measure of reliability that can be directly translated into a probability of resistance falling below the design resistance.

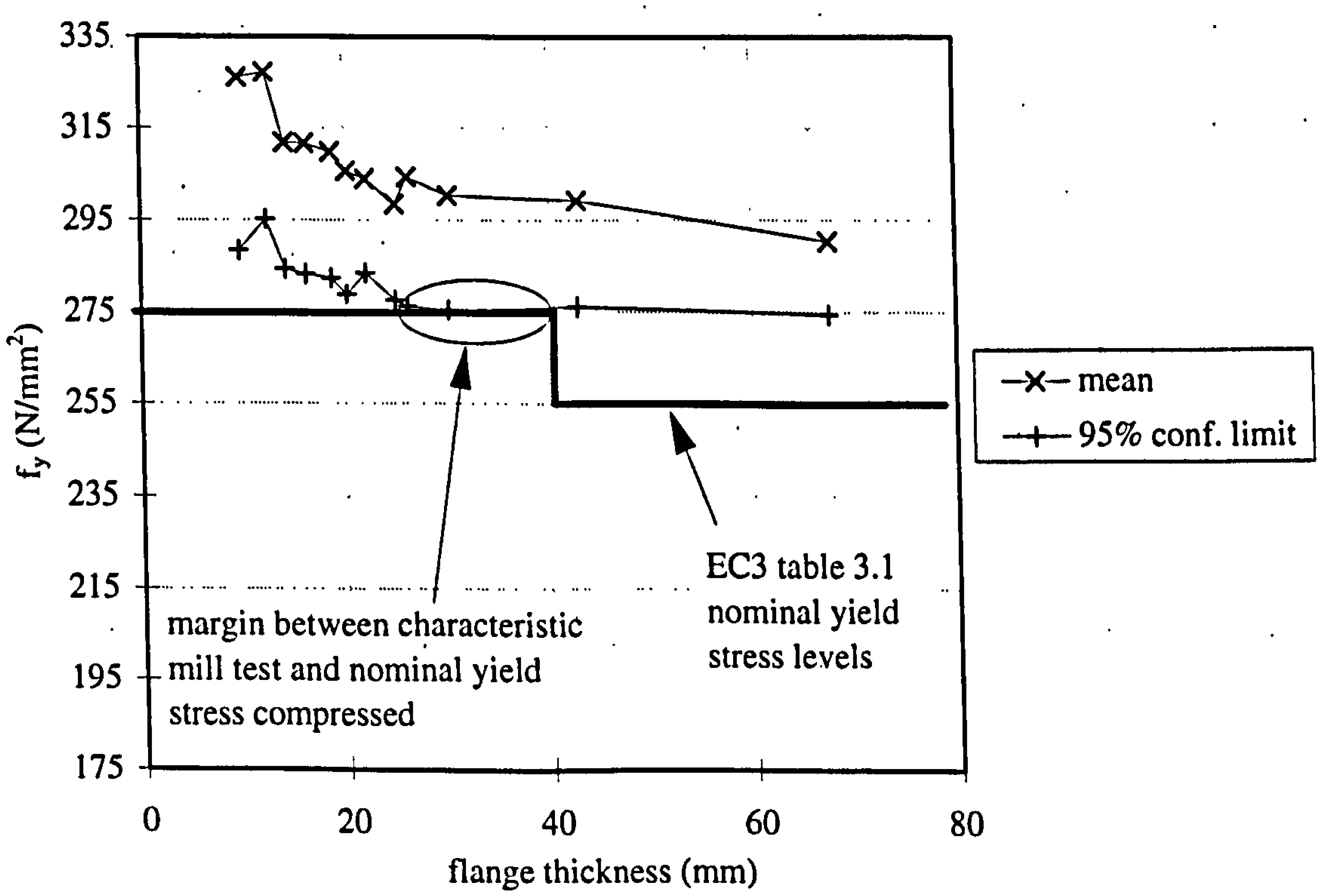


Fig. 6.1: Mill test vs. flange thickness (Nethercot and Byfield, 1993)

The study found that geometric variability of steel members is relatively low. Reliability was influenced mostly by the margin of safety existing between the 95% confidence limit for mill test results (the characteristic value) and the nominal yield stress used in design. The reason that safety index is influenced by flange thickness is explained in part by Fig. 6.1 (Nethercot and Byfield, 1993). As may be expected increasing flange thickness is accompanied by decreasing yield stress.

The nominal yield stress levels contained in EC3 attempt to mirror the yield stress vs. flange thickness relationship in order to ensure that the margin between characteristic mill test and the nominal yield stress is roughly uniform across the range of flange thicknesses. The choice of nominal yield stress levels was found to have important implications on γ_R^*, with the margin between the characteristic mill stress and the nominal yield stress providing what is in effect a reserve of reliability, thereby reducing the value required for γ_R^*. Thus, if this margin is compressed, reliability is reduced, a factor resulting in an increase in the value of γ_R^* necessary to achieve the specified target reliability.

Table 3.1 Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel to EN 10025 or prEN 10113.

Nominal steel grade	Thickness t mm*)			
	$t \leq 40$ mm		40 mm $< t \leq 100$ mm**)	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
EN 10025:				
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490
prEN 10113:				
Fe E 275	275	390	255	370
Fe E 355	355	490	335	470

*) t is the nominal thickness of the element.
**) 63 mm for plates and other flat products in steels of delivery condition TM to prEN 10113-3

Table 6.2: Table 3.1 taken from EC3 listing nominal material strengths

Table 3.1 of Eurocode 3 lists the nominal yield stress levels to be used for design purposes (see Table 6.2). Containing only one step in the nominal yield stress levels (at 40mm), these values of yield stress are unable to accurately mirror the relationship between material thickness and yield stress. Fig. 6.1 shows the relationship between mill test and flange thickness based on over a 4000 mill test results. Plotted alongside are the EC3 Table 3.1 nominal yield stress levels. These levels appear to provide a poor model of the relationship existing between yield stress and flange thickness; resulting in a compression of the margin of safety between the characteristic mill stress and nominal yield stress for flange thicknesses of between 20 and 40mm. This compression reduces the reliability of steelwork within this range of flange thicknesses, as measured by the reduced β-factor calculated within this zone.

Therefore, the choice of nominal yield stress levels is critical, since it affects the degree of scatter in reliability levels about the average over the complete range of structural sections. Since γ_R^* should be calibrated for the lowest reliability that could reasonably occur in a worst case scenario, the safety levels at flange thickness of between 30 to 40mm will produce high γ_R^*-values for all other steel.

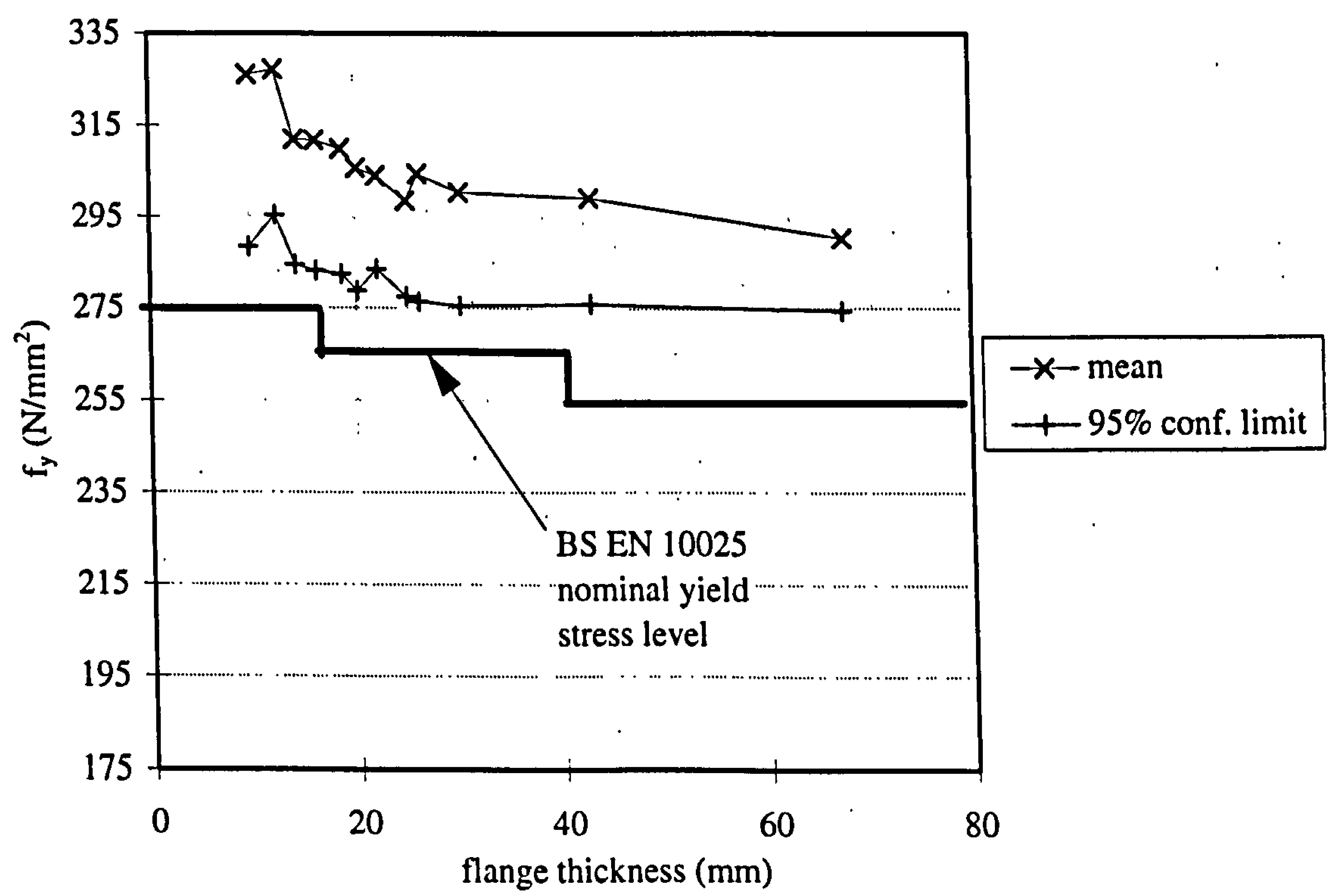


Fig. 6.2: Mill test vs. flange thickness (Nethercot and Byfield, 1993)

By comparison, the UK NAD of EC3 references nominal yield stress values for steelwork from the reference standards BS4360 (BSI, 1990) and BS EN 10025 (CEN, 1990). Table 6.3 lists the nominal yield stress levels set by these standards. These levels are shown graphically in Fig. 6.2. In comparison with Fig. 6.1, these levels mirror the actual variability of mill stress with flange thickness with a high degree of accuracy. The resulting degree of scatter of β-factors about the mean as calculated from a range of section sizes is correspondingly reduced; thus γ_R^*-determined for steel will also be reduced.

Designation			Yield stress (N/mm²)				UTS (N/mm²)	
New EN	Old EN	Former UK	t (mm) <=16	t (mm) >16 <=40	t (mm) >40 <=63	t (mm) >63 <=80	t (mm) <3	t (mm) >=3 <=100
S235	Fe 360	Grade 40	235	225	215	215	360	340
S275	Fe 430	Grade 43	275	265	255	245	430	410
S355	Fe 510	Grade 50	355	345	335	325	510	490
S460	FE E 460	Grade 55	460	440	430	410	550	550

Table 6.3: Nominal material properties specified for EC3 in the UK NAD reference standards (BSEN 10025, and BS4360)

This research has been carried out using the nominal yield stress levels as stated in the UK NAD reference standards, since they provide a superior model of the relationship between yield stress and flange thickness. From inspection of Table 3.1 of EC3 (Table 6.2 here), it would appear that the table is out of date given that the steel grade designation used has subsequently been changed. With this thought in mind, it is hoped that the table will be revised in later issues of the code; to come into line with the UK NAD recommended values.

6.2.2 ANALYSIS OF MILL TEST DATA

The nominal yield stress levels prescribed in EC3 and the relevant supporting standards correspond to a characteristic value i.e. the 95% confidence limit. Theoretically at least, a certain percentage of steel will possess a yield stress lying below the nominal yield value. In practice producers are reluctant to produce

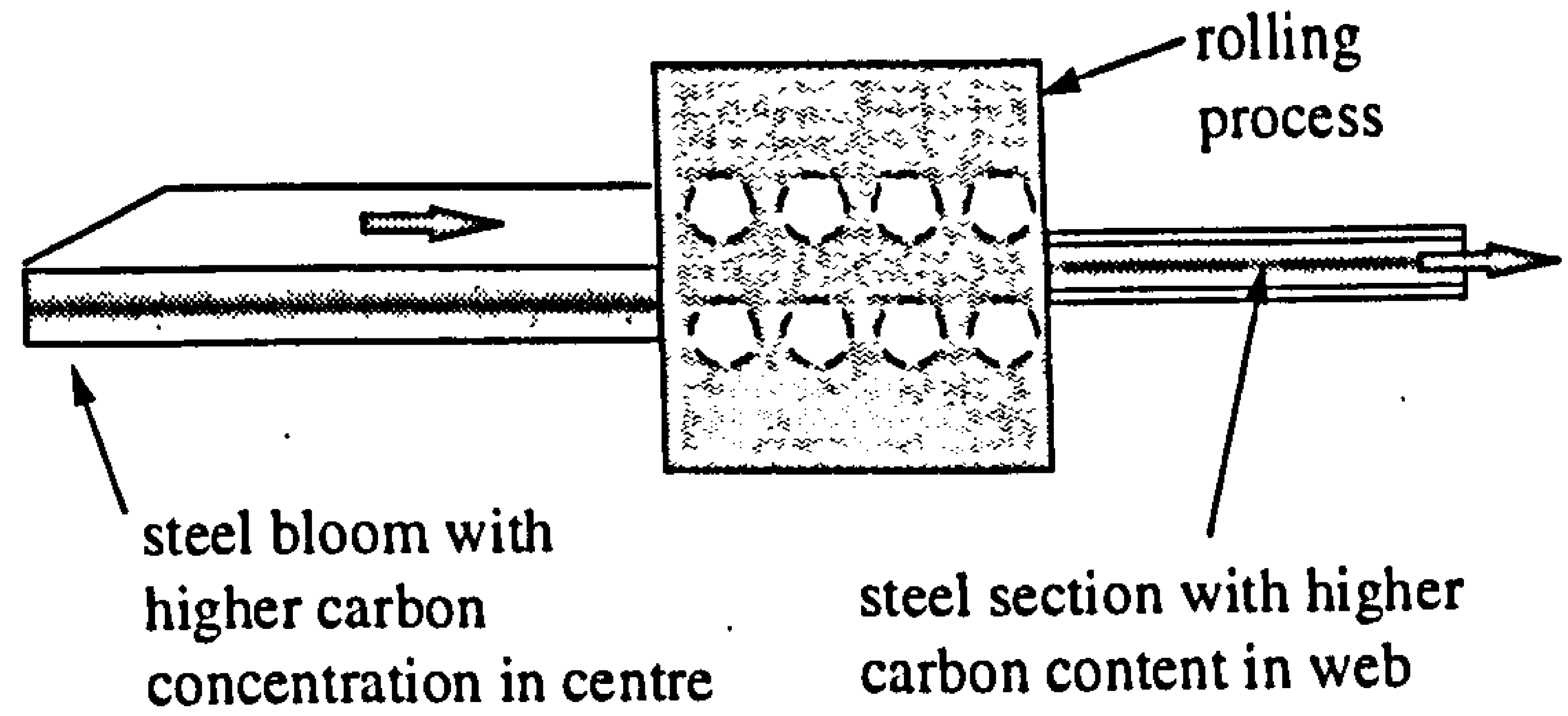


Fig. 6.3: Illustration of the mechanism by which web material obtains a higher carbon content than flange material

steel with a yield point lower than the nominal value since that yield value is likely to apply to a whole batch of steel. Thus the analysis of mill test results reported herein shows the eventuality of mill tests actually falling below the nominal yield value is rare.

The position from which the test coupon is taken has a critical effect on the corresponding material strength recorded. If the coupon is taken from the web - as specified in the ASTM standard - then a relatively high mill test value will be recorded. This is mainly for two reasons: firstly webs are thinner than flanges - and strength tends to increase with decreasing thickness; secondly, webs tend to have a higher carbon content than flanges because carbon tends to accumulate via crystallisation in the centre of the steel blooms from which sections are rolled. This is reflected by an increase in carbon content in the centre of the web; a concept illustrated in Fig. 6.3.

The British specifications BSEN10025 and BS4360 specify mill test coupons are to be cut from the flange. Since flange material contributes most of the strength against bending, this is a logical location. The difference resulting from whether the coupon is cut from the web or flange varies for the particular section, though the difference rarely exceeds 10 to 15% (McGuire, 1968).

Mill tests are carried out at a relatively high rate of loading and the material strength recorded - often known as the 0.5RT - is the stress corresponding to the 0.5% total strain (this is not a 0.5% proof stress, or a specific upper or lower yield point).

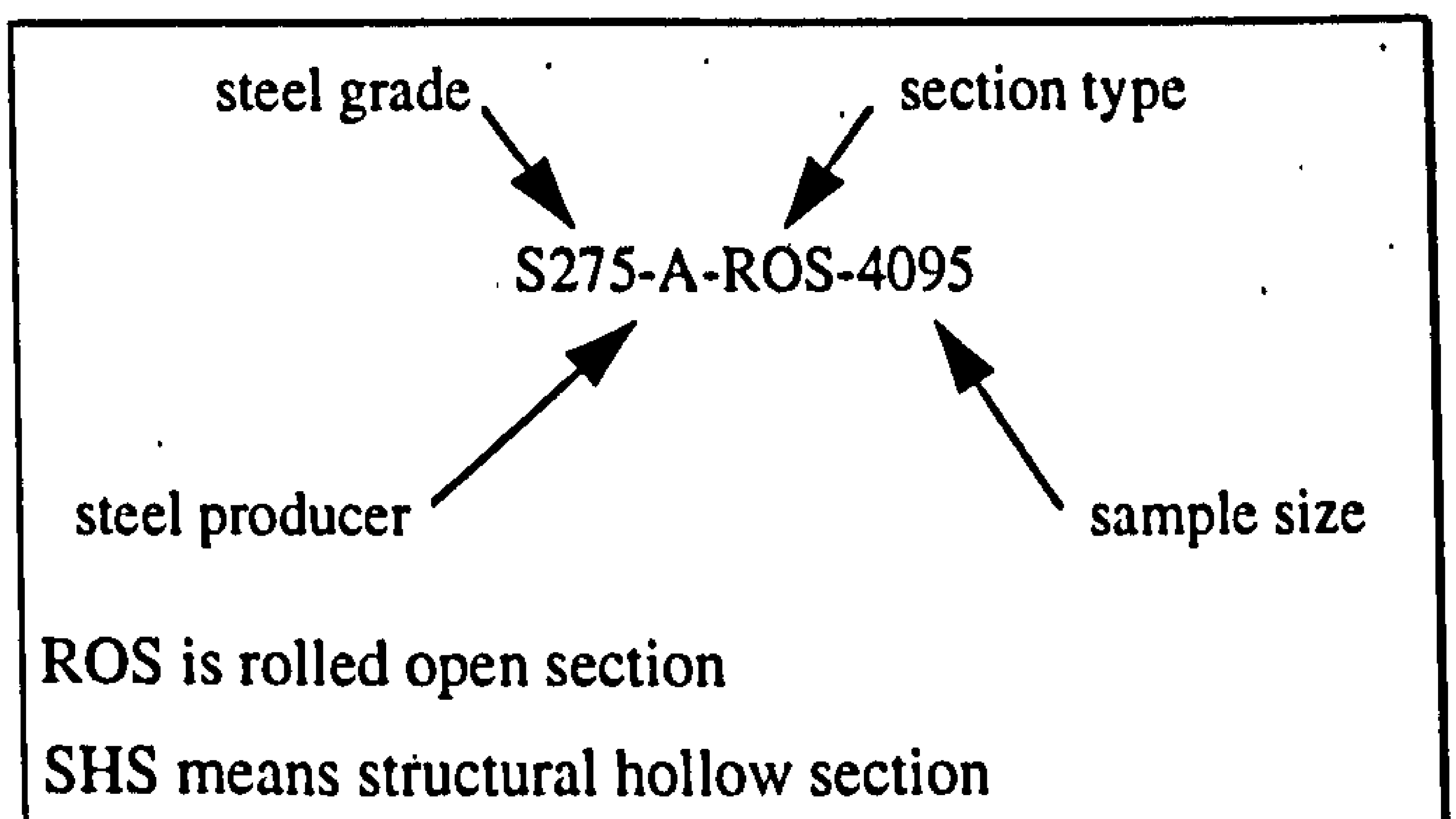


Fig. 6.4: Mill test coding system

In this study mill tests have been used for calibration purposes. Low strain rate tests are normally only undertaken for the purpose of scientific laboratory testing. They are therefore difficult to collect in large numbers. By comparison, mill tests provide a rich source of material strength data, since they form a standard part of the steel production process. Thus they can be collected in numbers large enough for accurate estimates of population mean and standard deviation to be made.

Listed in Appendix 2 are the tabulated summaries of mill test results, with each table listing the data for a particular steel grade from a particular steel producer. Also listed in Appendix 2 are figures of mill test results vs. flange thickness. Using the data from each table, two figures have been created: one showing mill test vs. flange thickness; and the other one normalised mill test (mill test/nominal yield stress) vs. flange thickness. On each figure is plotted the mean and 95% confidence limit. According to assumptions made whilst calibrating EC3, the 95% confidence limit should correspond to the nominal yield stress. Thus the margin between the nominal and 95% confidence limit will affect reliability. The graph of mill test vs. flange thickness should show a higher yield stress for a lower flange thickness; this trend should flatten out on the normalised mill test vs. flange thickness graphs. Finally, the coding system illustrated in Fig. 6.4 has been used to indicate: steel producer, steel grade, section type and sample size.

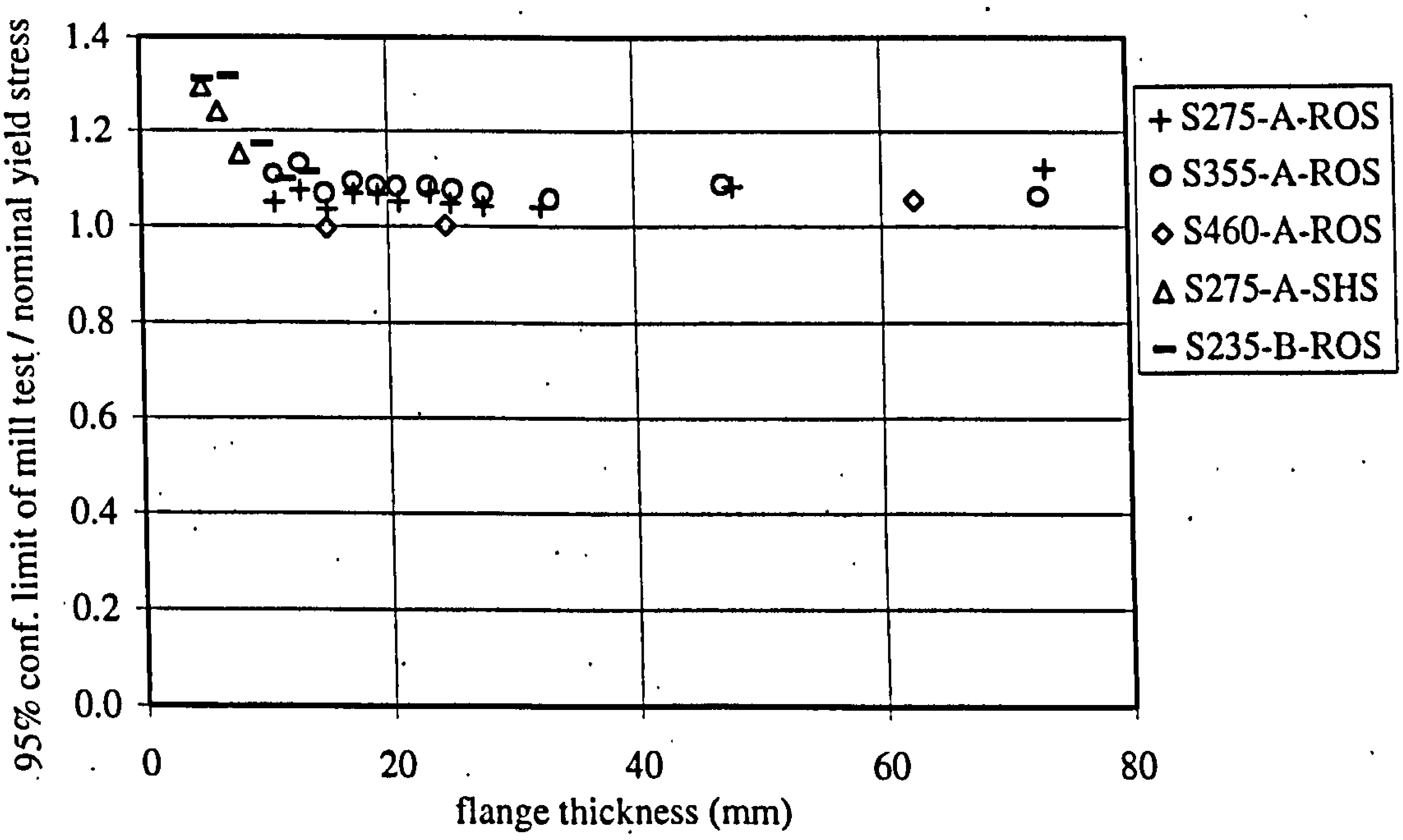


Fig. 6.5: Characteristic values of normalised mill test results vs. flange thickness (data based on 7660 mill test results)

6.2.3 DISCUSSION

If structural steelwork is to achieve the target level of reliability, then 95% of steel members should possess a yield stress in excess of the nominal value specified by manufacturers. The nominal yield stress levels specified in EN 10025 (CEN, 1990) are intended to ensure that steel reliability should remain unaffected by material thickness or steel grade.

The figures contained in Appendix 2 show the relationship between mill test and flange thickness, based on the data analysed for this survey. These figures confirm that increasing material thickness is associated with decreasing material strength. The characteristic profile of the relationship is of uniform material strength for sections with flange thickness greater than 40mm, with a sharp increase in material strength for sections with flange thickness less than 10mm. Between 10 and 40mm, material strength reduces, though less rapidly.

The nominal yield stress levels listed in (CEN, 1990) are intended to reflect this relationship; and thereby negate the adverse effect that variable material strength has on steel reliability. Listed in Appendix 2 are figures showing the relationship between the normalised mill test strength and flange thickness. These figures confirm that (CEN, 1990) nominal yield stress levels do provide a sufficiently accurate model of the material strength vs. flange thickness relationship. Thus, the relationship between normalised mill test and flange thickness becomes roughly linear, for sections with flange thickness greater than 10mm. For sections with flange thickness less than 10mm, the nominal yield stress levels underestimate mill test strength.

Fig. 6.5 shows the combined normalised characteristic mill test results plotted from the Appendix 2 figures. Contained are the mill test results for a total of 4 different steel grades based on data from 2 different producers. In total, the figure is based on the analysis of 7660 mill test results. The figure confirms that the reliability of steel remains constant, regardless of the steel grade for steel with a thickness of greater than 10mm. The 95% confidence limit of the normalised mill test strength lies just above unity. Thus, the material strength of steel members is consistent with the nominal yield stress levels.

For sections with a flange thickness greater than 10mm, the normalised characteristic mill test varies little, regardless of flange thickness or material specification. Thus material strength does not affect reliability for steel with flange thicknesses in excess of 10mm. Reliability levels increase where flange thickness is less than 10mm. An increase in the nominal yield stress level of $10N/mm^2$ for sections with flange thicknesses of less than 8mm would make reliability more uniform across the range of section sizes. Since relatively few sections have flange thickness less than 10mm, results suggest that the nominal yield stress levels are appropriate.

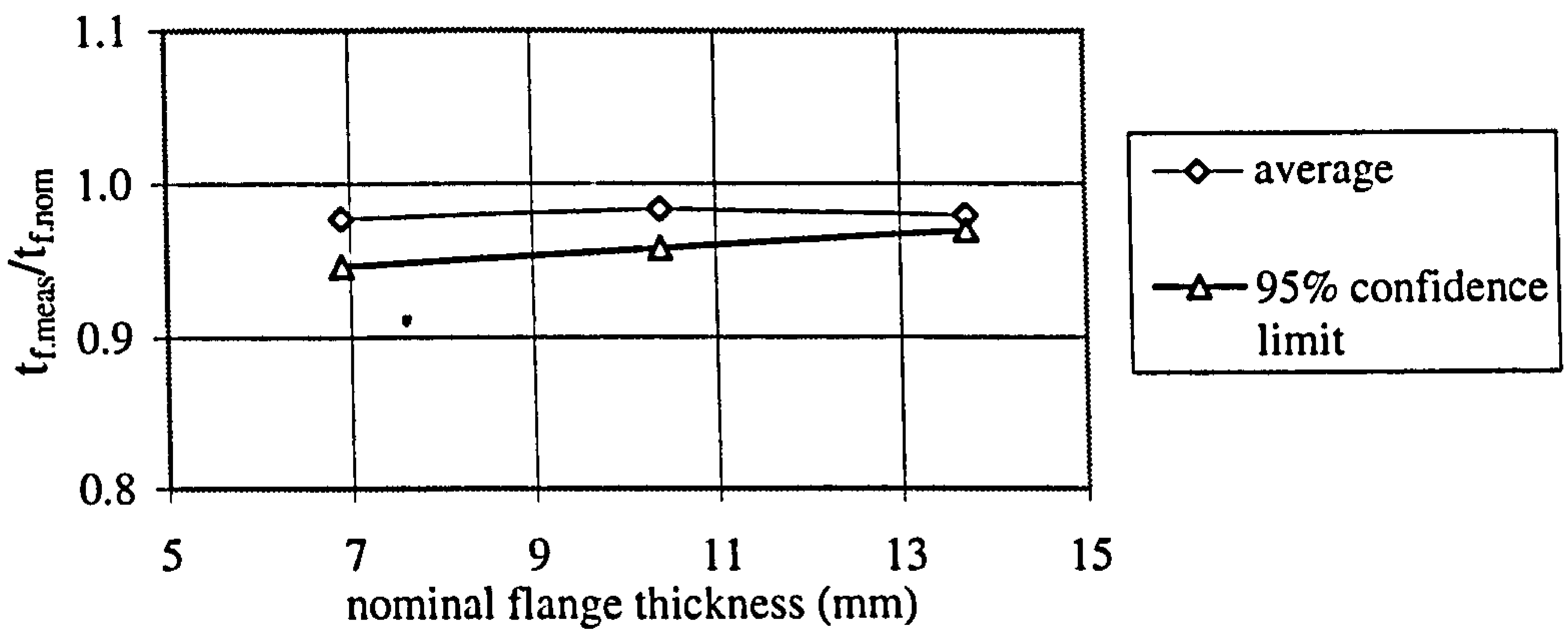


Fig. 6. 6: The relationship between the nominal flange thickness and flange thickness variability - based on S235-B-ROS-689.

An additional step in the yield stress levels where flange thickness is less than 8mm may be omitted, if the positive effect of increasing yield stress is negated by increased geometric variability. Fig. 6. 6 illustrates the relationship between geometric variability and flange thickness, as observed during this survey; it shows that decreasing flange thickness is not associated with a significant increase in geometric variability. Therefore, a further step in the nominal yield stress levels at 8mm would be justified. An additional step would offer a small enhancement to the efficiency of lightweight steel sections.

During the calibration of EC3, the following assumptions (CEN, 1993a) have been made about the variability of material properties:

1) No correlation (statistical dependence) exists between the basic variables of the strength function;
2) the variability of material properties approximates to the log-normal distribution;
3) $V_{fy} = 0.07$ (Sedlacek *et al*, 1989);
4) the nominal value of yield strength is a characteristic value, i.e. a 95% confidence limit.

This survey confirms that these assumptions do indeed provide a realistic measure of the variability of material properties. Briefly, these assumptions are discussed separately as follows:

Assumption (1): *No correlation (statistical dependence) exists between the basic variables of the strength function.* The analysis reported herein demonstrates that the normalised yield strength (measured/nominal) is independent of flange thickness, where t_f>10mm, Fig. 6.5. Since the majority of steel used in construction has t_f>10mm, this is a valid assumption.

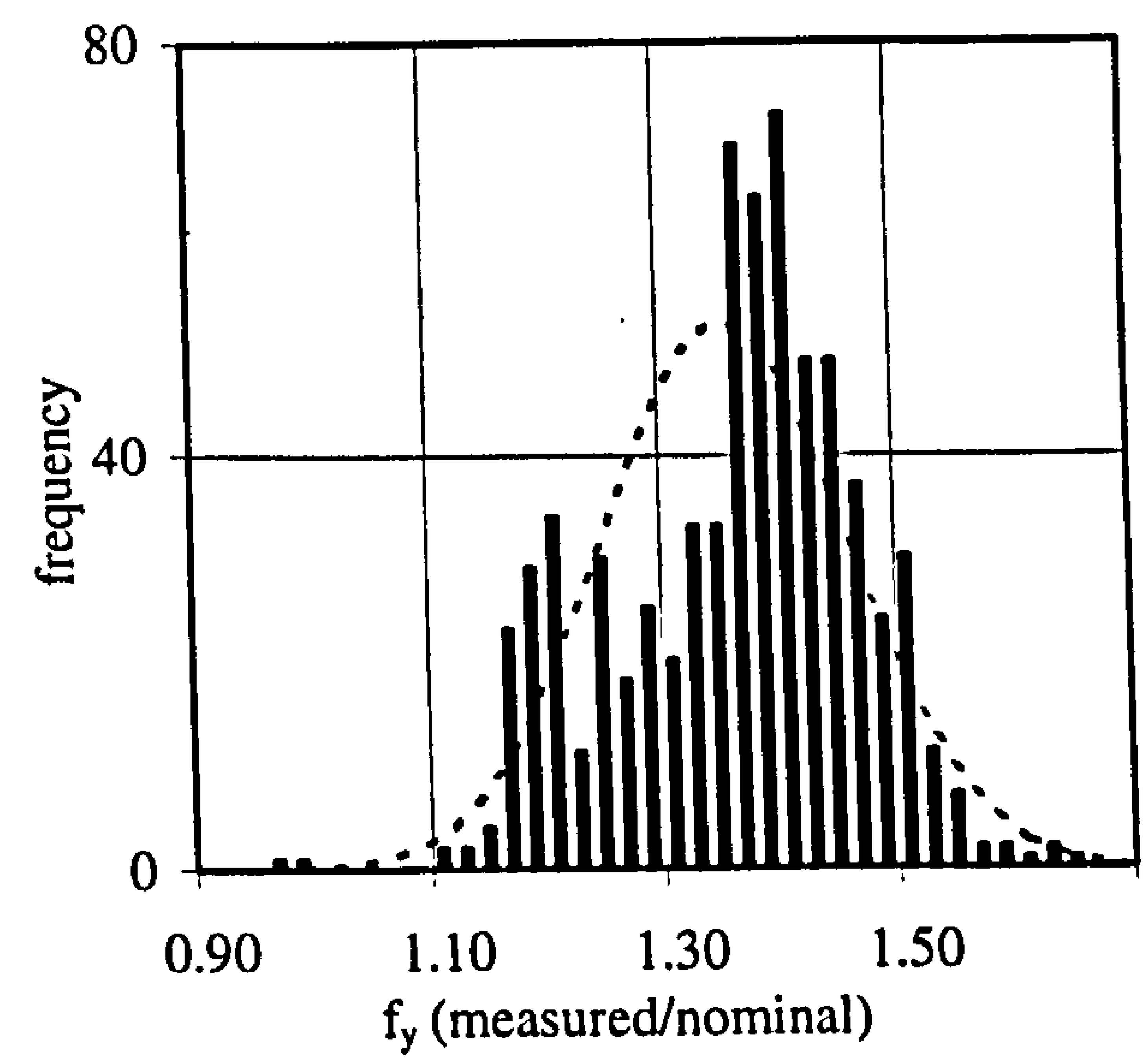


Fig. 6.7: The frequency distribution S235-B-ROS-689

Assumption (2): *the variability of material properties approximates to the log-normal distribution.* Fig. 6.7 shows the probability distribution based on mill tests from producer B; also plotted is the log-normal p.d.f. (probability distribution function). The mill tests exhibit a probability distribution with two peaks. This is characteristic for mill tests and originates from the incorporation of higher grade steel into the sample. Higher grade steels that fail to meet the required strength are typically re-assigned. Whilst the two profiles are quite different, for calibration purposes only the lower tail is of interest. In this case the log-normal distribution provides a satisfactory model of the lower tail. Thus the assumption that the probability distribution of yield stress approximates to the log-normal p.d.f is valid for this sample of steel.

Assumption (3): V_{fy} = *0.07*. The results from this survey are summarised in Table 6.4. If the sample analysed is representative of delivered structural steel, then V_{fy} can be safely be reduced to 0.05.

t_f (mm)	sample size	V_{fy}	f_y mean/nom	95% conf. limit of f_y (mean/nom.)
less than10mm	829	0.053	1.37	1.25
greater than 10mm	6831	0.046	1.16	1.07

Table 6.4: The variability of mill tests

Assumption (4): *the nominal value of yield stress is a characteristic value.* The results from this survey indicate that this assumption is slightly conservative. The results from this survey summarised in Table 6.4 show that the mean yield stress was 1.16 x nominal yield stress, where t_f>10mm. Equation (6. 1) is the equation used for determining the location of mean yield stress according to assumptions (2) and (4). Given that V_{fy} = 0.05, then mean yield stress = 1.09 x nominal yield stress, see equation (6. 2). Therefore, the assumption that nominal yield stress is a characteristic value is slightly conservative.

$$f_{y.mean} = \frac{f_{y.nom}}{\exp(-1.645V_{fy} - 0.5V_{fy}^2)} \tag{6. 1}$$

$$f_{y.mean} = \frac{f_{y.nom}}{\exp(-1.645x0.05 - 0.5x0.05^2)} = 1.09f_{y.nom} \tag{6. 2}$$

6.3 THE VARIABILITY OF GEOMETRIC PROPERTIES

6.3.1 ASSUMED GEOMETRIC VARIABILITY

The following assumptions have been made about the geometric variability of steel during the calibration of EC3 γ_R^*-factors (CEN, 1993a):

1. all variables approximate to the log-normal distribution;
2. no correlation (statistical dependence) exists between the basic variables of the strength function;
3. geometric properties have a mean value equal to the nominal value specified for the purpose of design;
4. the coefficient of variation of geometric properties is approximately 0.03.

This section will attempt to gauge the appropriateness of these assumptions, given that they are based on work originally undertaken during the 1970's (Alpsten, 1977). It should be noted that basic geometric variables include section properties, not section dimensions. It can be argued that actual dimensions such as flange thickness are the true basic variables, since section properties are dependent upon several dimensions. In the background documentation to EC3 (Sedlacek *et al*, 1989), COV values for calibration purposes are only quoted for section properties. No information is provided on dimensional variability. For the sake of consistency this approach has been adopted in this study.

The Annex Z calibration method is based on the assumption that no correlation exists between basic variables. Due to BS4 (BSI, 1980) requirements about minimum section weight, it is likely that the variability of section dimensions will correlate to some extent; i.e. variability of flange thickness and section depth will be linked. It may be for this reason that the background documentation to EC3 provides no information on dimensional variability. This effect is negated to a large extent through the use of section properties. Therefore the development of a manageable statistical method for calibration purposes is made easier.

6.3.2 ANALYSIS OF DIMENSIONAL DATA

Measurements of the material and geometric properties of over 689 rolled open sections are listed in (Bureau, 1993). These measurements were analysed to quantify the geometric variability. Listed in Table 6.5 are the results of the analysis, whilst Fig. 6.8 to Fig. 6.18 show the frequency distribution profiles for basic variables. Also plotted on these figures is the log-normal p.d.f. Of particular interest is the degree of fit of the lower tail of the log-normal distribution compared to the observed distribution and the location of the normalised mean value of the sample in relation to unity.

Basic variable	mean measured / nominal	COV measured / nominal
h	1.01	0.010
b	1.00	0.010
t_w	1.01	0.044
t_f	0.98	0.017
A	0.99	0.022
I_y	1.00	0.025
I_z	0.98	0.037
$W_{el.y}$	1.00	0.019
$W_{el.z}$	0.98	0.029
$W_{pl.y}$	1.00	0.020
$W_{pl.z}$	0.98	0.029
I_w	1.00	0.039
I_t	0.97	0.056

Table 6.5: Results from the analysis of 689 measurements originating from Producer B

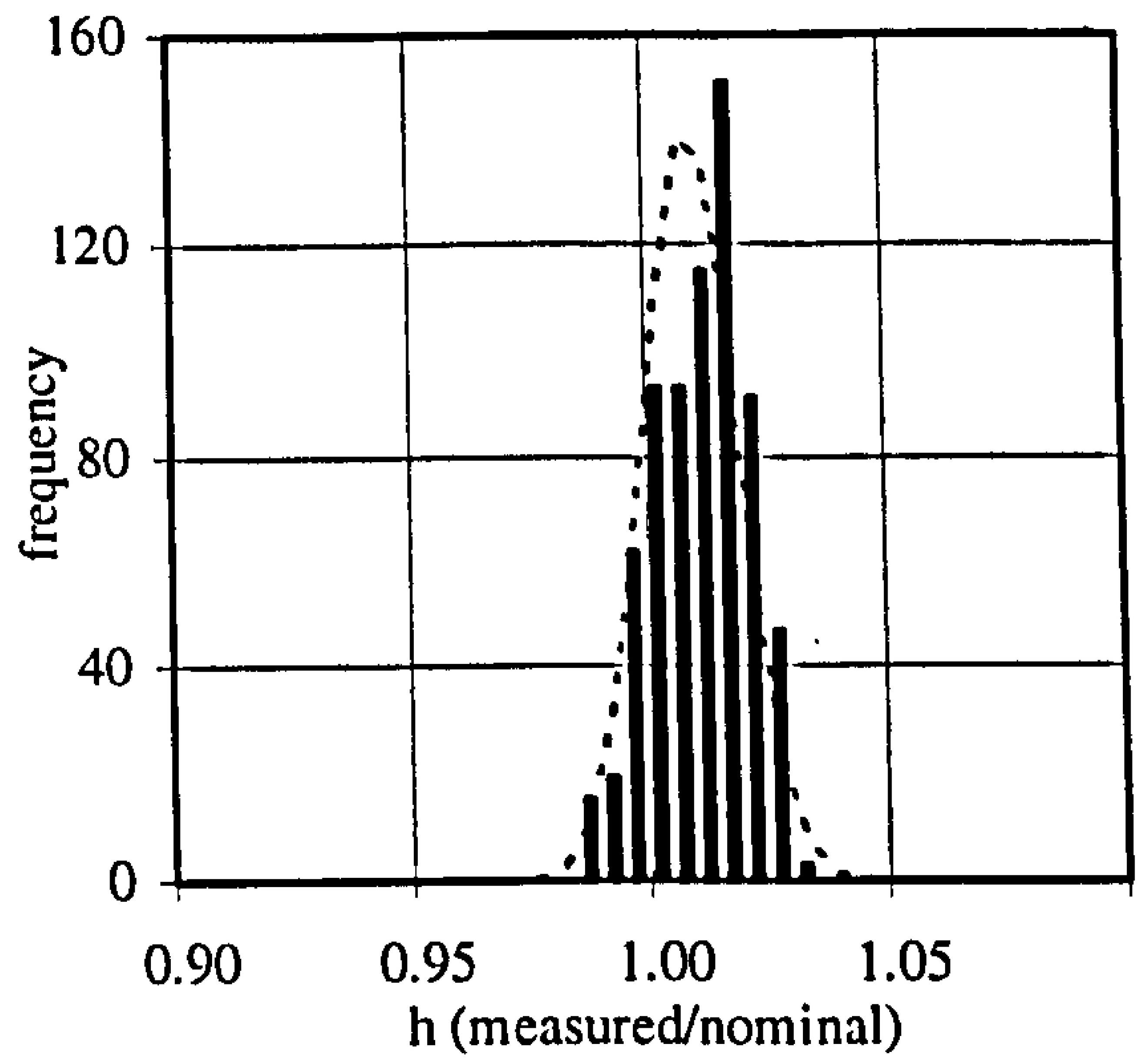


Fig. 6.8: S235-B-ROS-689

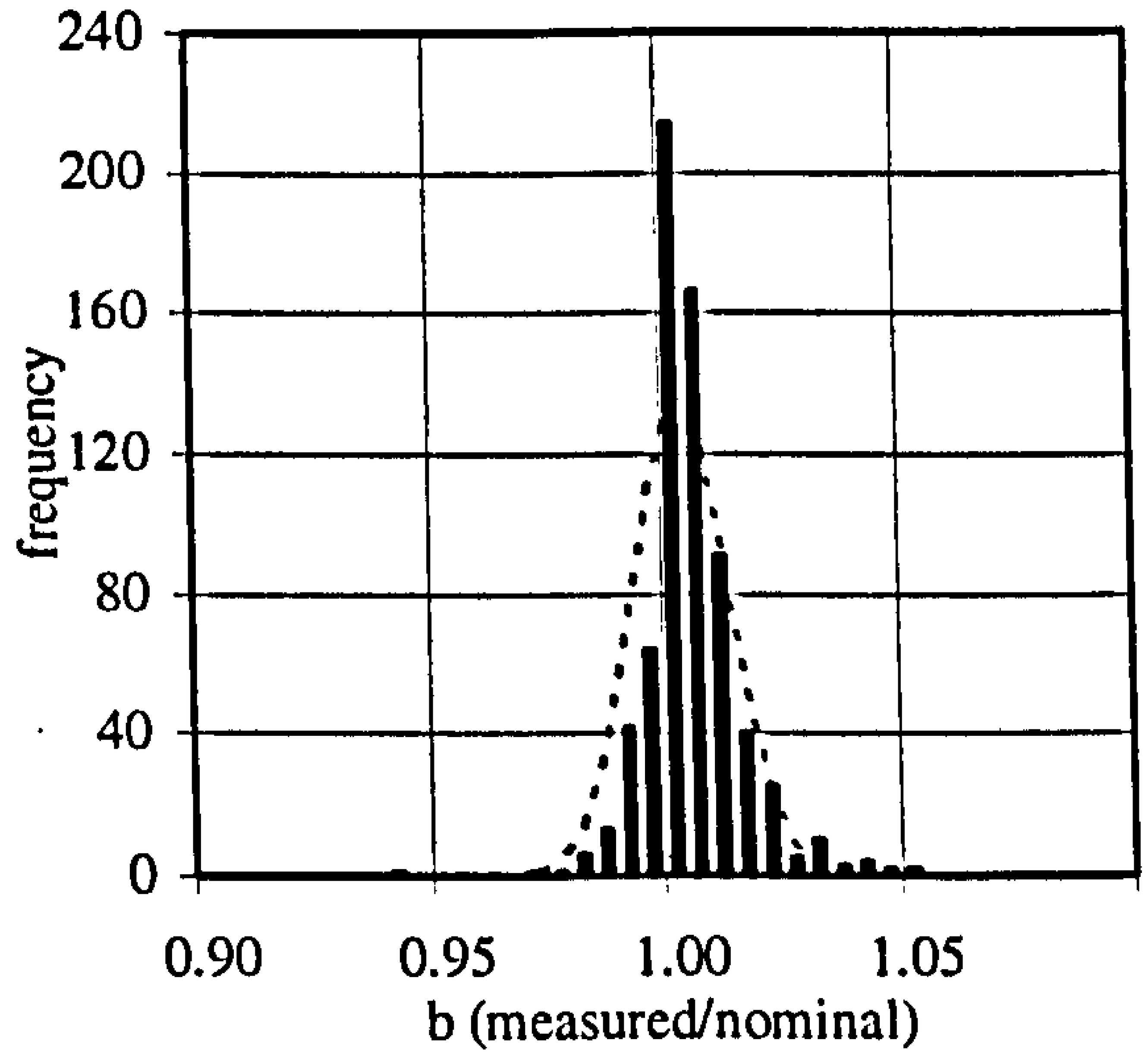


Fig. 6.9: S235-B-ROS-689

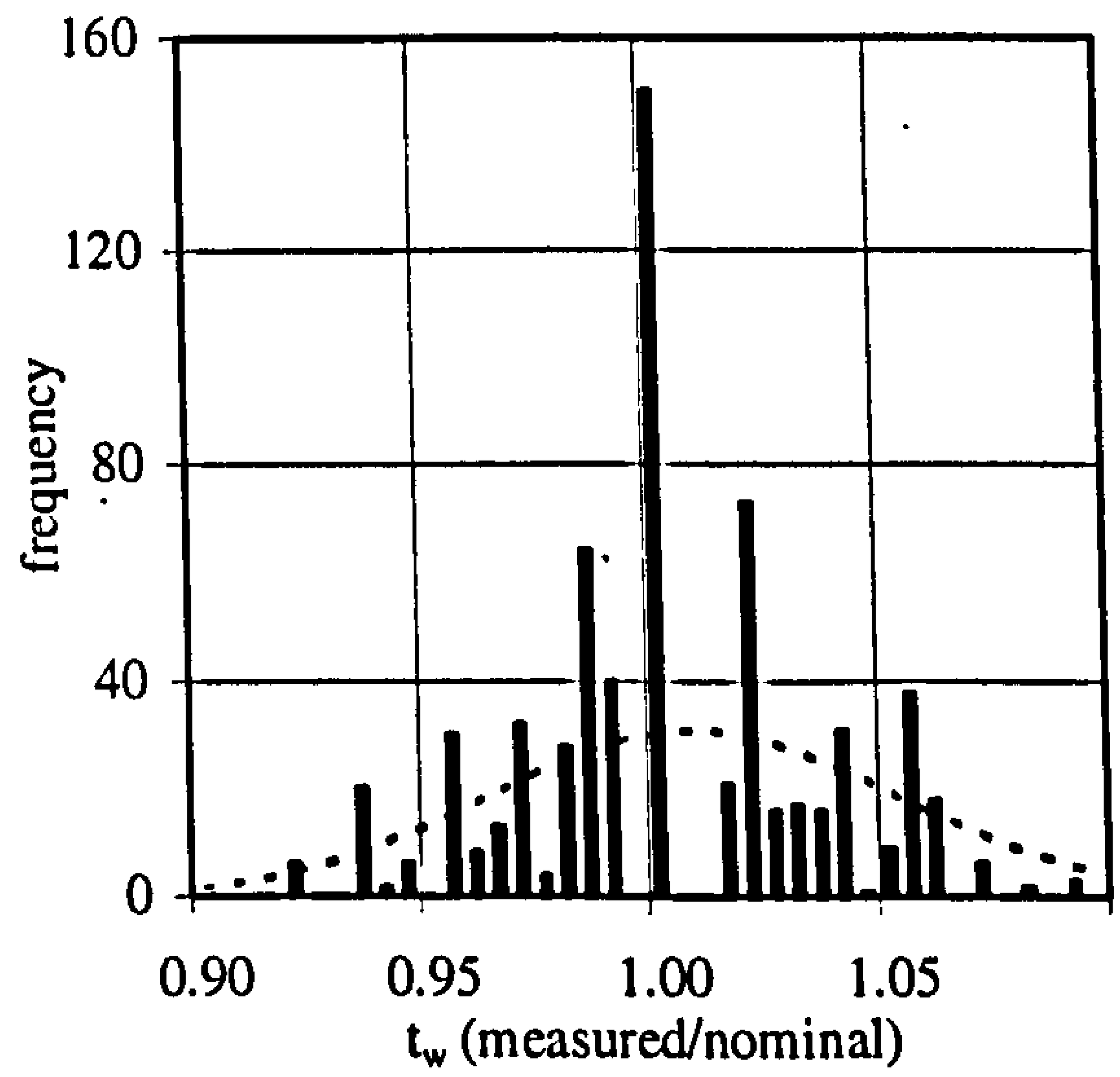


Fig. 6.10: S235-B-ROS-689

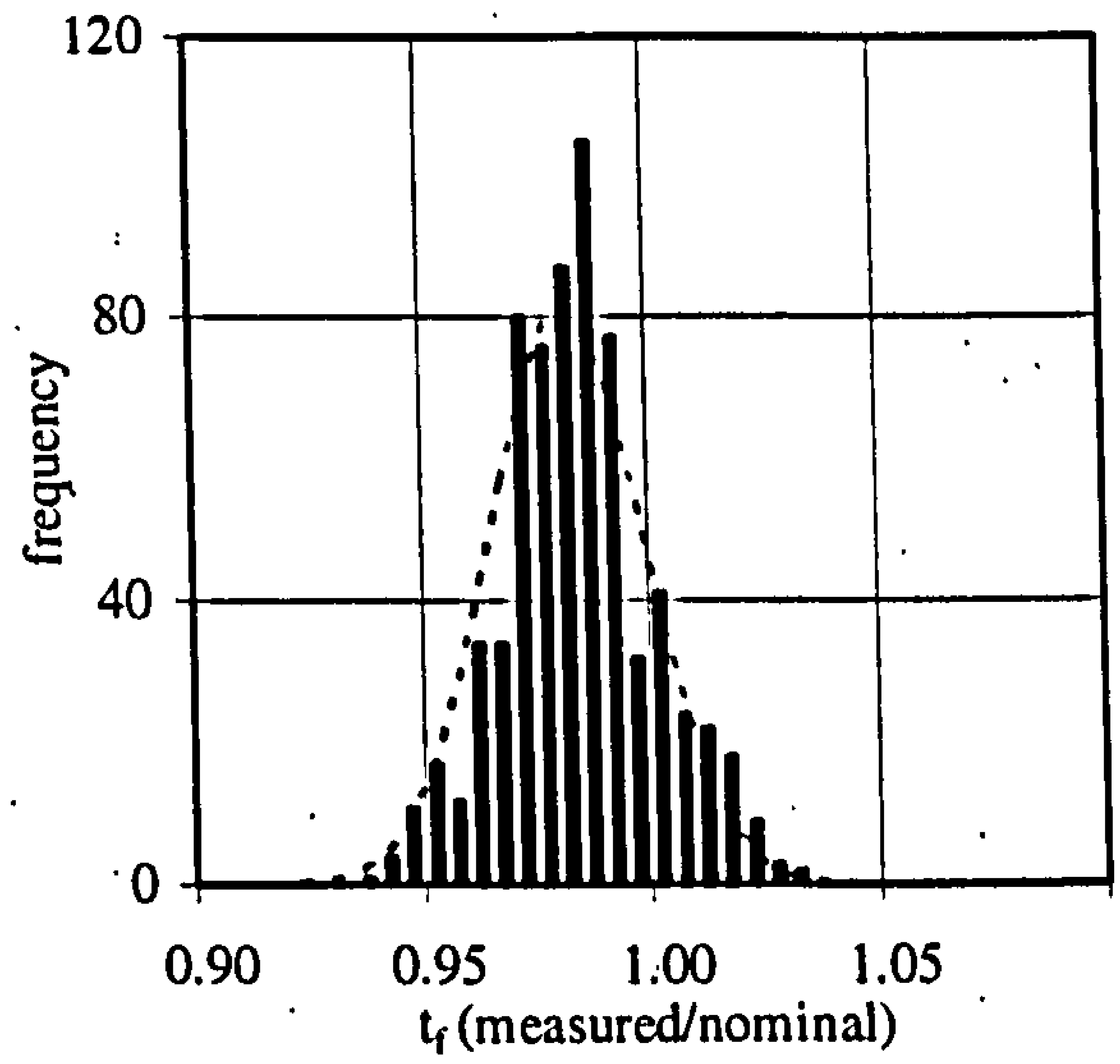


Fig. 6.11: S235-B-ROS-689

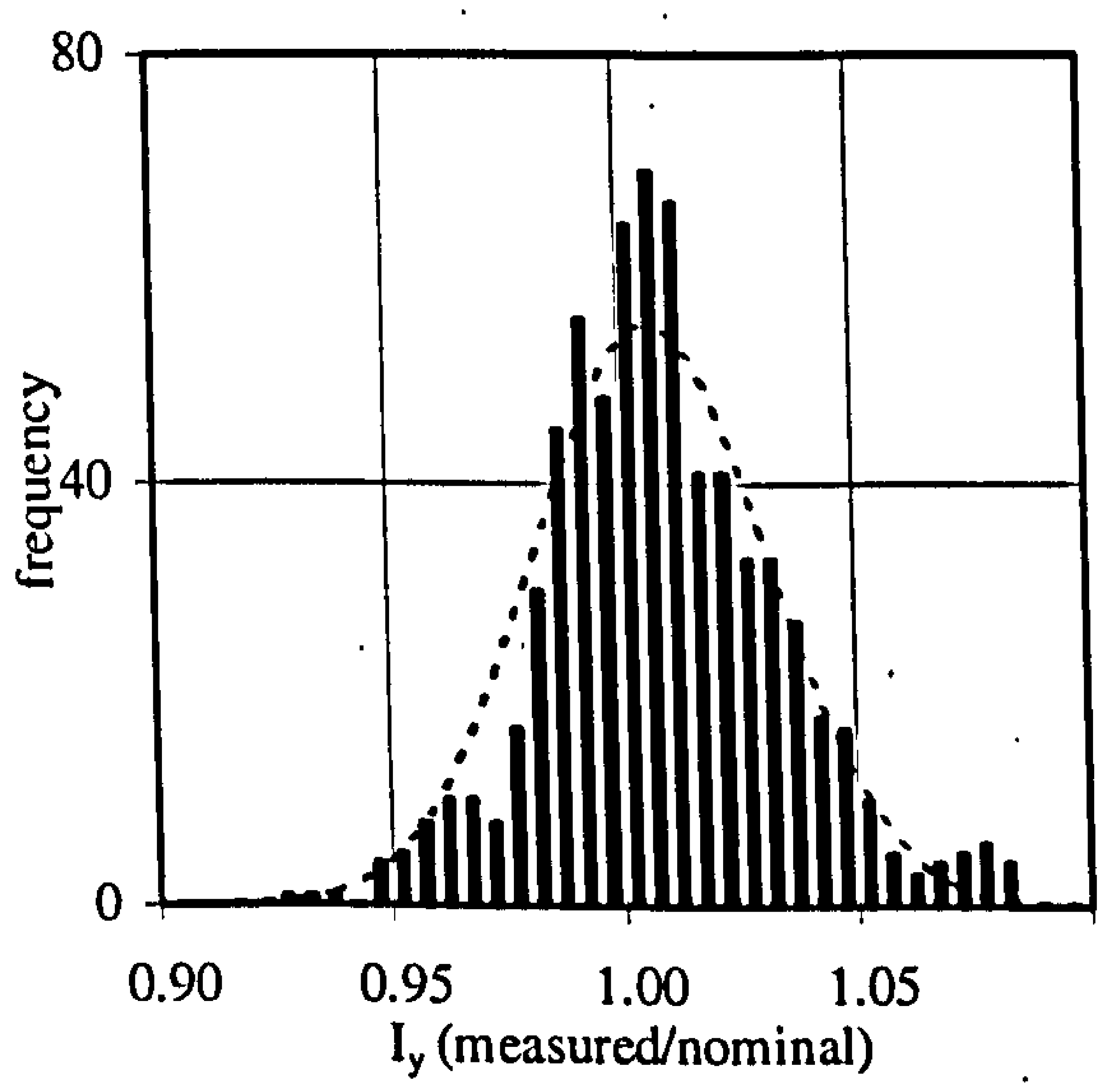


Fig. 6.12: S235-B-ROS-689

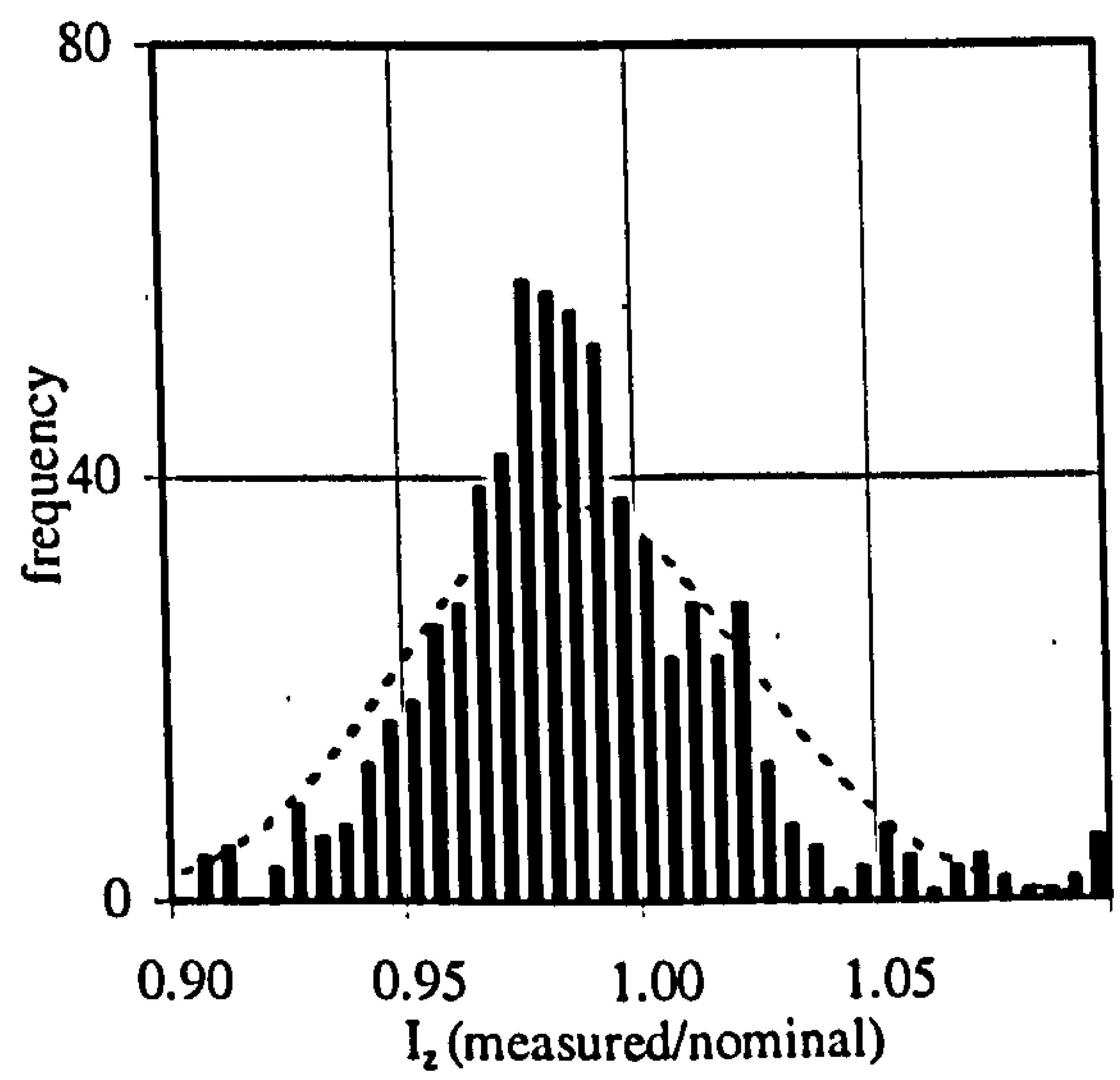


Fig. 6.13: S235-B-ROS-689

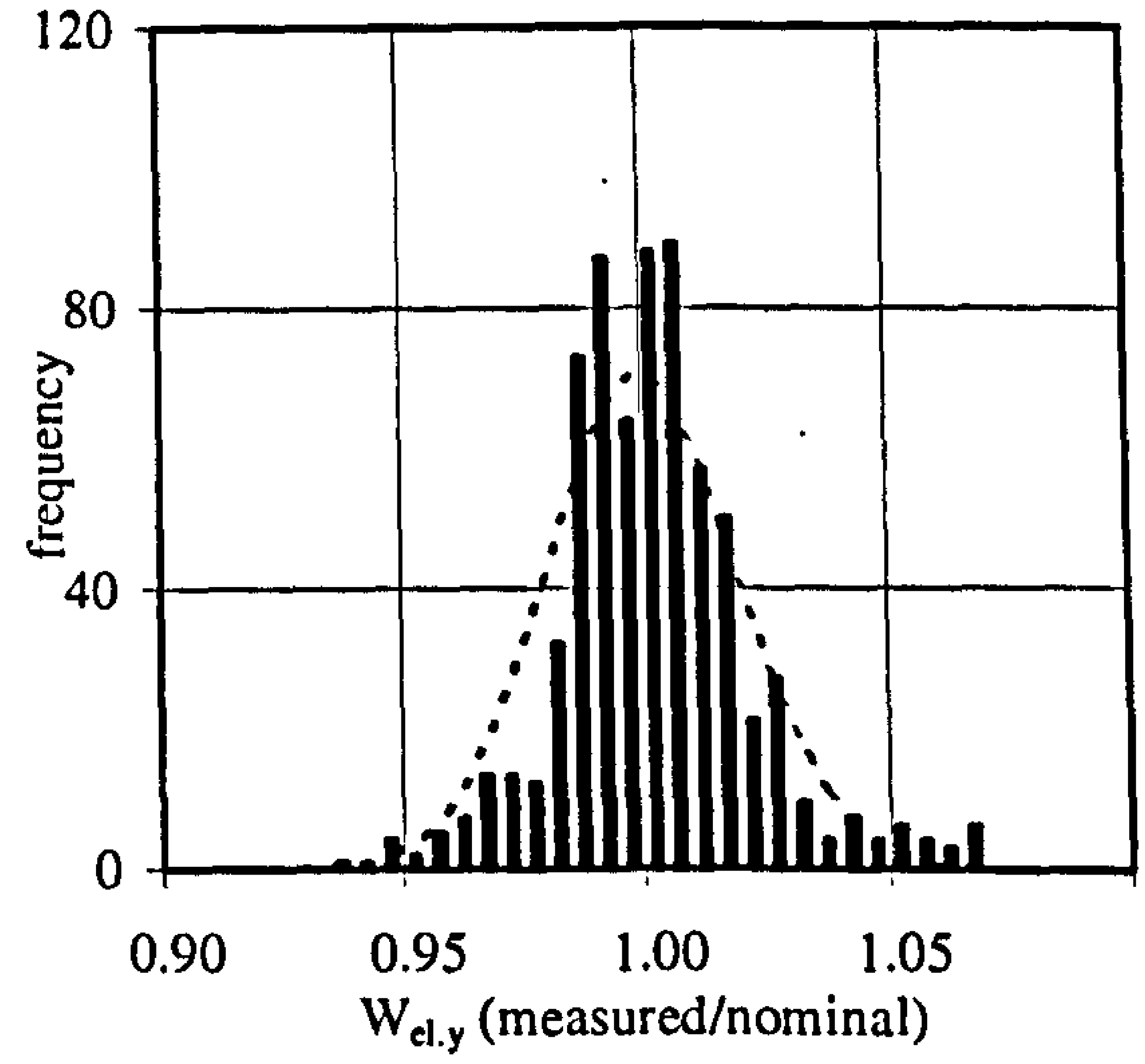


Fig. 6.14: S235-B-ROS-689

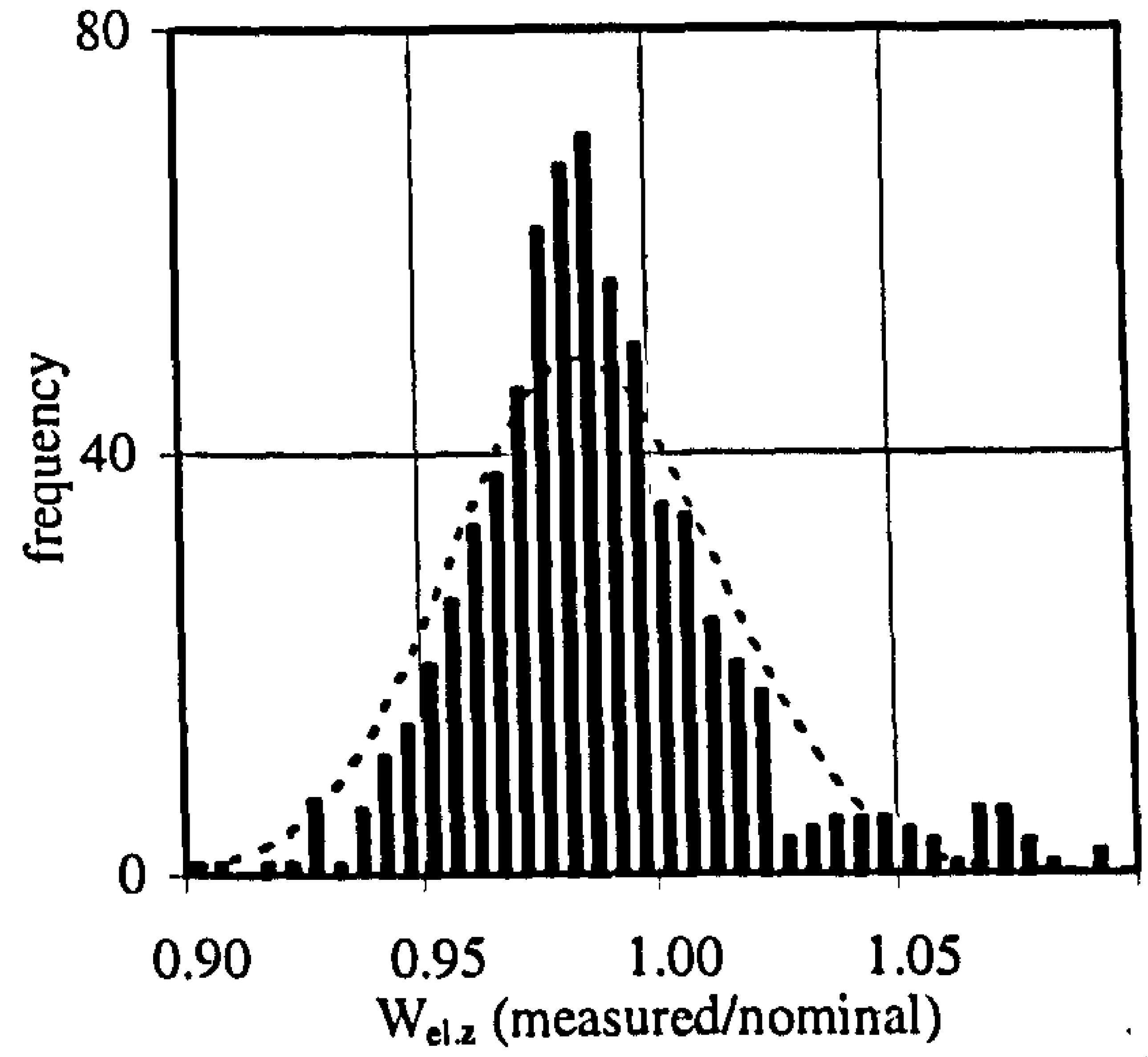


Fig. 6.15: S235-B-ROS-689

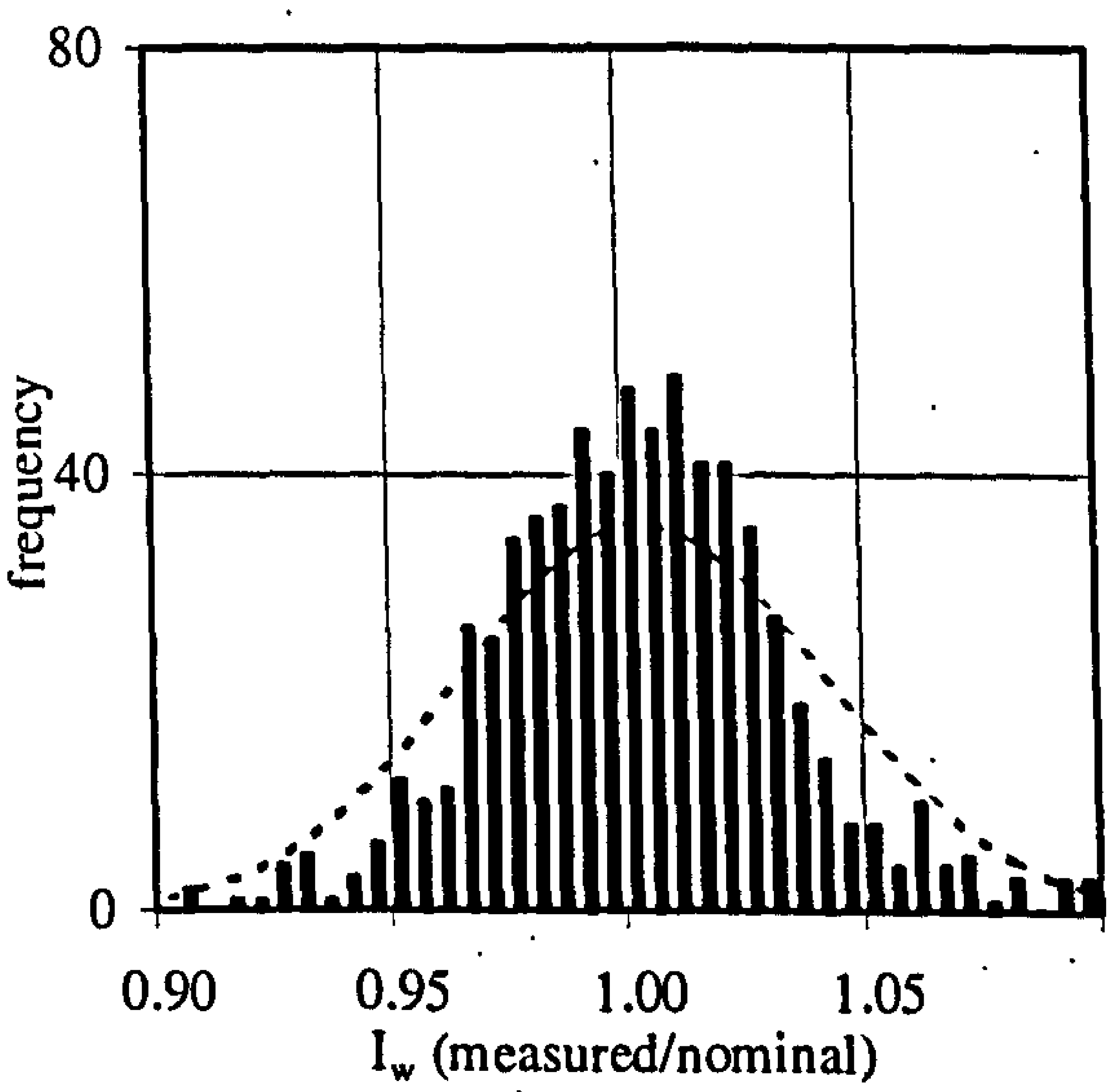


Fig. 6.16: S235-B-ROS-689

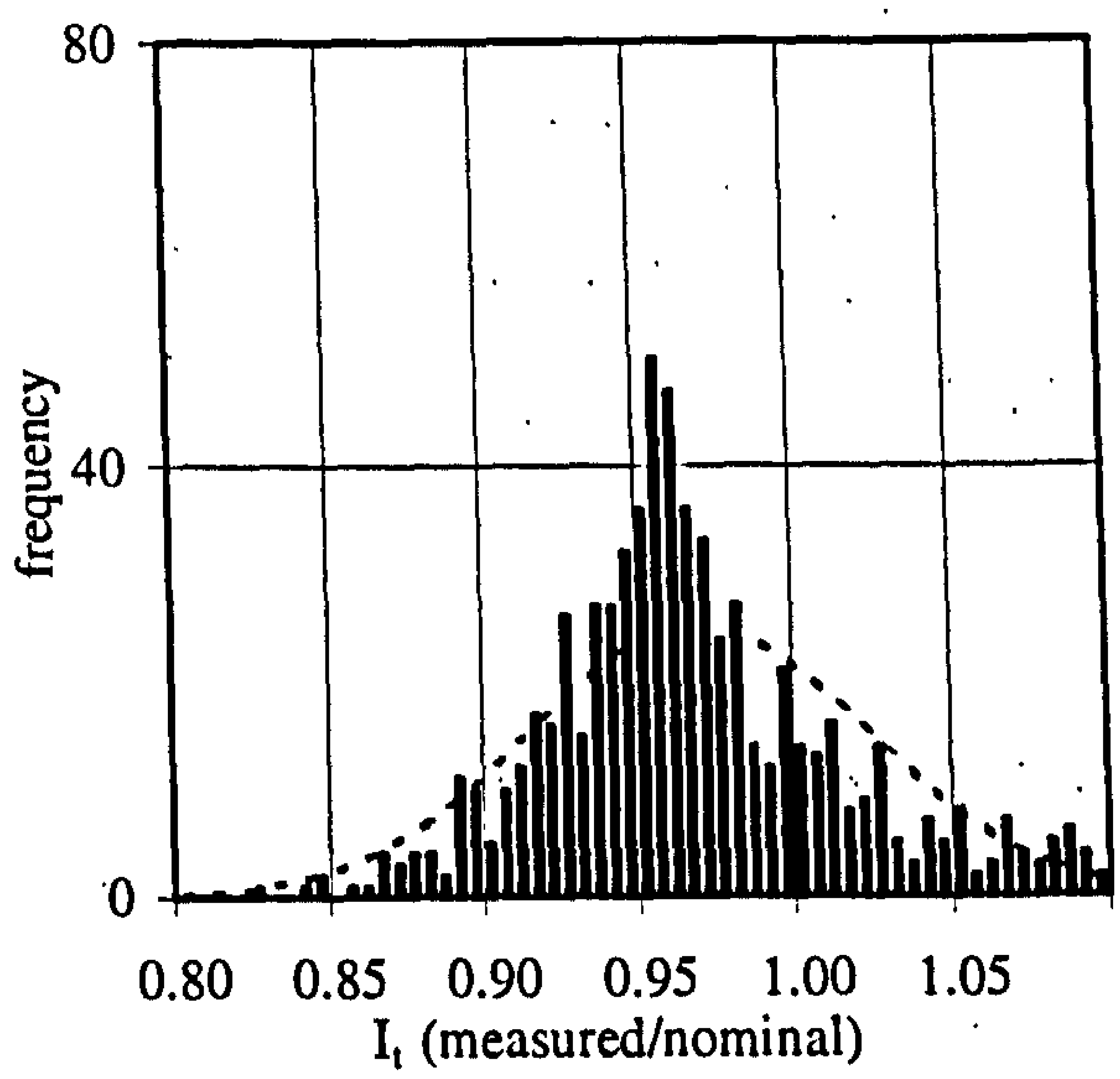


Fig. 6.17: S235-B-ROS-689

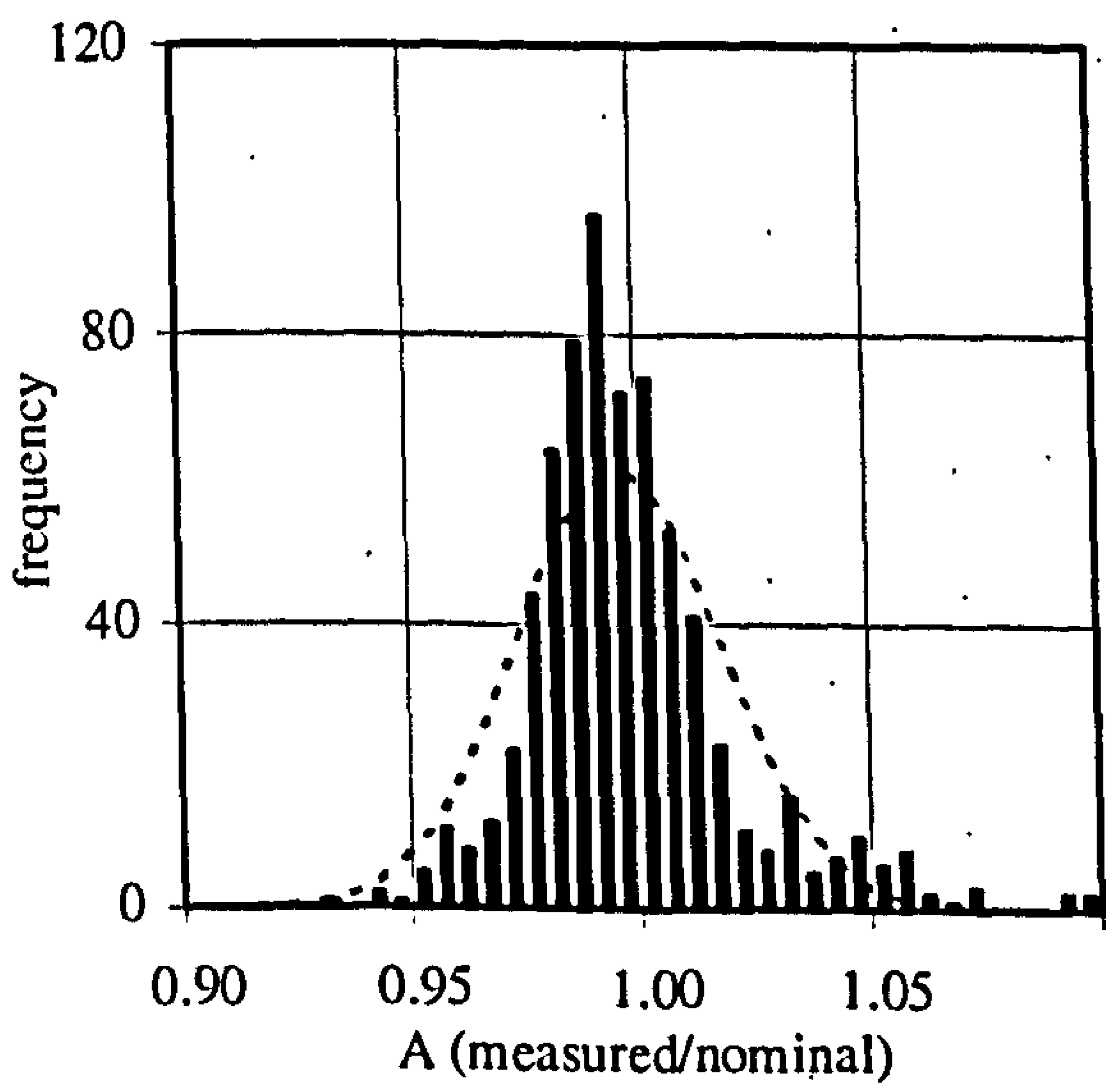


Fig. 6.18: S235-B-ROS-689

6.3.3 DISCUSSION

The dimensions h, b, t_w and t_f all exhibit classic bell shaped probability distributions; a variability accurately modelled by the log-normal p.d.f. In addition, the manufactured mean is approximately equal to the nominal values specified for designers. Section depth and width show little variation. Mean flange thickness is less than the nominal, though the negative effect of this on section properties is offset by both the mean width and depth being slightly greater than the nominal. In comparison to h, b and t_f, t_w exhibits a high degree of variability. Thus, section properties dependent on web thickness exhibit a correspondingly high variability; V_{Iw} =0.039 and V_{It}=0.056 in comparison to V_{Iy}=0.025, a quantity largely unaffected by web thickness..

The Annex Z approach to γ_R^* calibration assumes that geometric variability approximates to the log-normal p.d.f.. Fig. 6.8 to Fig. 6.18 confirm this assumption. The log-normal p.d.f. provides an extremely good approximation of the variability observed in this data. It should be noted that the normal p.d.f. will provide almost identical results in this situation.

The background documentation to EC3 (Sedlacek *et al.*, 1989) specifies the value of coefficient of variation for basic geometric variables as equal to 0.03, see Table 6.6. The variability observed in the studied data confirms that this seemingly over simplified approximation does in fact provide a realistic measure of observed variability. Almost all the geometric variables calculated from this sample lie close to this figure. The exception is I_t which is greatly influenced by the high variability of web thickness.

The final assumption about the variability of geometric variables is that mean values of geometric variables correspond to the nominal value. Once again the observed variability supports this assumption. Table 6.5 lists the detailed variability found in the analysed samples. These values are compared with the EC3 background document values in Table 6.6.

There is some evidence to show that geometric variability reduces with increased section size, see Fig. 6. 6. However, as the data analysed in this study are based mainly on the smaller section sizes, they may have produced relatively high values of COV. It could be argued that geometric variability should be based on the analysis of larger section sizes since the higher geometric variability associated with the light weight sections is offset by a rapid increase in yield stress (due mainly to the higher degree of work hardening and reduced cooling time associated with the manufacture of light weight sections).

Basic variable	EC3 COV	Observed COV	EC3 mean/ nom.	Observed mean/ nom.
A	0.03	0.02	1.0	1.0
I_y	0.03	0.03	1.0	1.0
I_z	0.03	0.04	1.0	1.0
$W_{pl.y}$	0.03	0.02	1.0	1.0
$W_{pl.z}$	0.03	0.03	1.0	1.0
I_t	0.03	0.06	1.0	1.0
I_w	0.03	0.04	1.0	1.0

Table 6.6. Coefficients of variation as used for calibrating Eurocode 3: Part 1.1 (Sedlacek et al, 1989) compared with observed variability.

6.4 THE VARIABILITY OF RESISTANCE

6.4.1 ASSUMED RESISTANCE VARIABILITY

The major and minor axis plastic moments of resistance have been calculated for the sections analysed in section 6.3. Of particular interest is the type of p.d.f. that best models resistance; since resistance is assumed to be a log-normal variable in the Annex Z method.

6.4.2 ANALYSIS OF DATA

Fig. 6.19 and Fig. 6.20 show the frequency distributions of the predicted moments of resistance, calculated using the measured material and geometric properties listed in (Bureau, 1993). Fig. 6.12 shows frequency distribution profile exhibited by the major axis second moment of area. This approximates well to the normal or log-normal p.d.f.s. Fig. 6.7 shows the frequency distribution profile of yield stress, which exhibits the characteristic twin peak distribution associated with yield stress. Both these effects tend to merge with the distributions for calculated plastic moment of resistance.

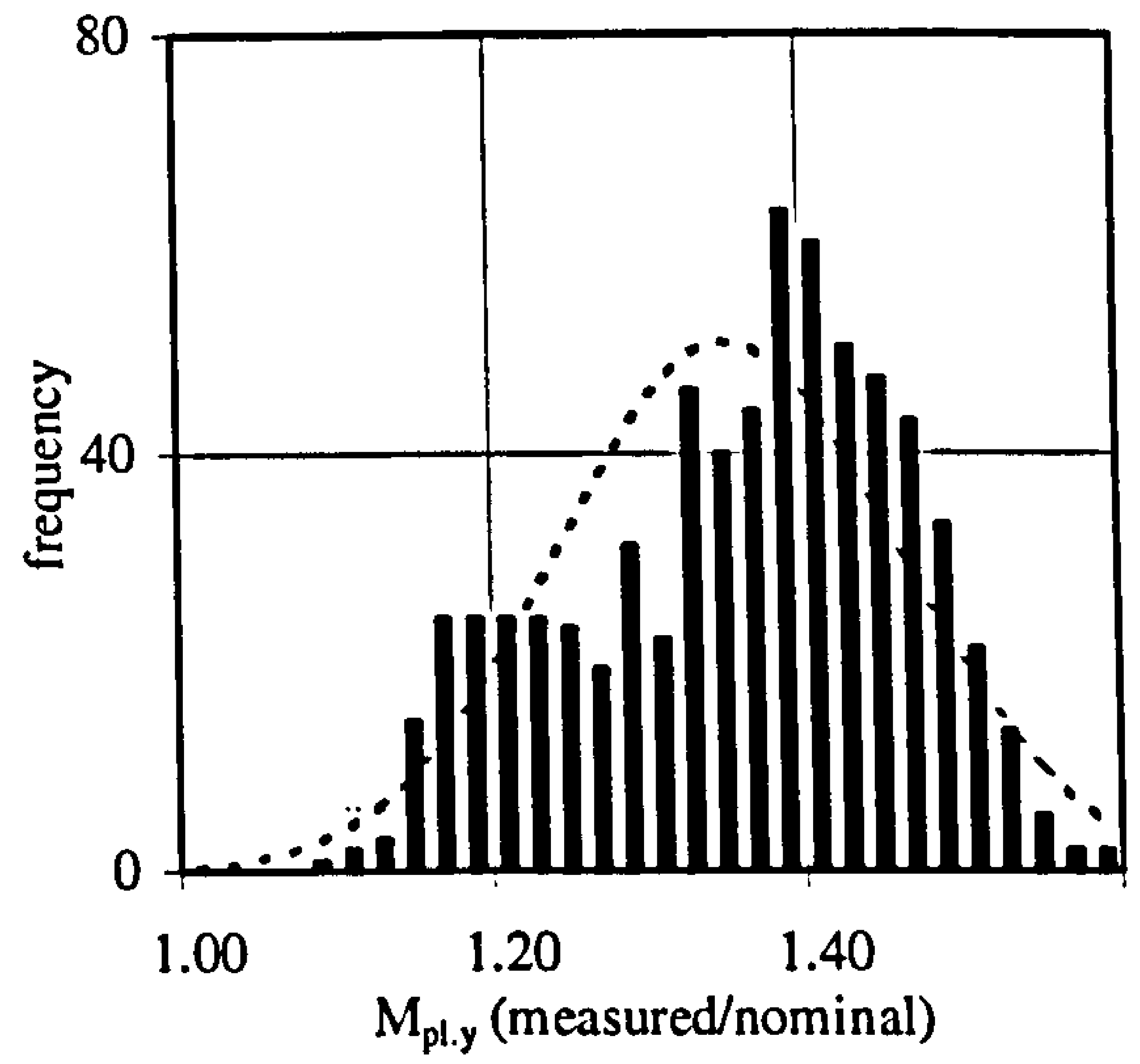


Fig. 6.19: S235-B-ROS-689

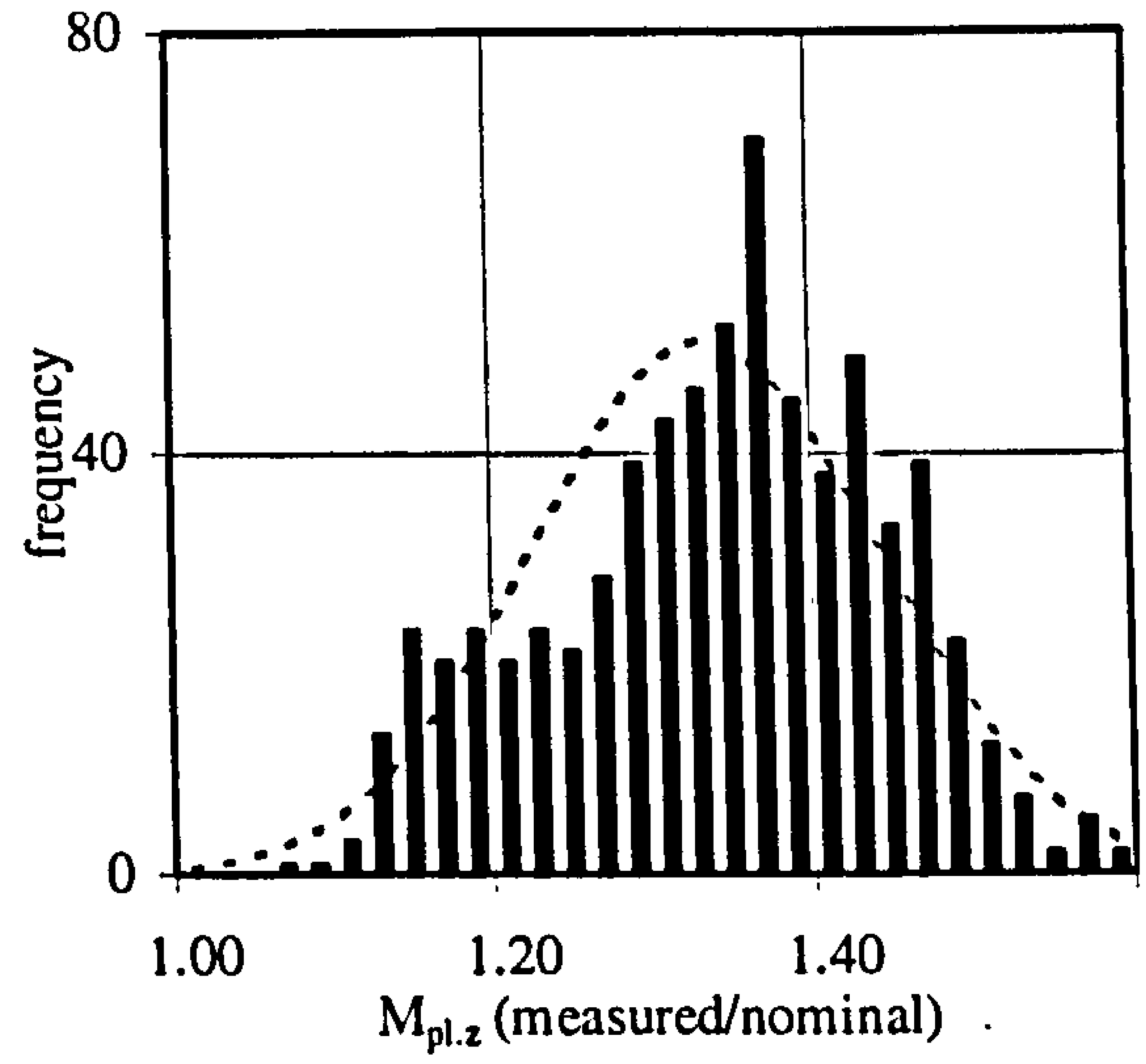


Fig. 6.20: S235-B-ROS-689

Basic variable	mean measured / nominal	COV measured / nominal
$M_{pl.y}$	1.34	0.081
$M_{pl.z}$	1.32	0.082

Table 6. 7: Results from the analysis of 689 measurements originating from Producer B

6.4.3 DISCUSSION

Whilst at first glance the log-normal p.d.f. provides a poor reflection of the observed variability, for the purposes of calibration the log-normal p.d.f. must have the ability to accurately model the extreme low end of the distribution tail. At this point the log-normal distribution provides a realistic approximation of the observed variability, though a slightly conservative answer may result. Thus the assumption that resistance is a log-normally distributed variable would appear to be justified, given the need for a workable statistical method of γ_R^* calibration.

6.5 CONCLUSIONS

The assumptions made about the variability of geometric and material properties during the calibration of EC3 affect both the reliability and economics of steel design. From an inspection of the EC3 background documentation, it would appear that many of these assumptions are based on work undertaken about 20 years ago (Alpsten, 1977). This study has examined the appropriateness of the key assumptions, with the following findings:

Assumption 1: All variables approximate to the log-normal distribution. This study demonstrates that the log-normal p.d.f. does provide a reasonable model of the observed distribution of material and geometric properties. In particular the log-normal p.d.f. provides an accurate, though conservative, model of the lower tail of the observed variability.

Assumption 2: No correlation (statistical dependence) exists between the basic variables of the strength function. This work demonstrates that the nominal yield stress levels specified in product standards (referenced in the UK NAD to EC3) provide a sufficiently accurate model of the relationship between material thickness and yield stress, that the effect of the correlation between them is negated. By contrast, the nominal yield stress levels specified in Table 3.1 of EC3 (see Table 6.2), provide an insufficiently accurate model of this relationship to negate the correlation; with the result that reliability is adversely affected.

Assumption 3: Geometric properties have a mean value equal to the nominal value specified for the purpose of design. Assumption verified.

Assumption 4: The nominal value of yield strength is a characteristic value, i.e. a 95% confidence limit. Assumption conservative, though not inappropriate.

Assumption 5: The coefficients of variation for basic variables approximate to the values listed in Table 6.1. The COV of yield stress can safely be reduced from 0.07 to 0.05. The COV of 0.03 for geometric properties is a reasonable approximation of the variability found in this study.

Chapter Seven

PLATE GIRDER RELIABILITY

7.1 INTRODUCTION

Plate girder design is arguably one of the most complex design tasks considered by Part 1.1 of Eurocode 3. The work reported in this chapter has been undertaken in order to gauge the validity of the γ^*_R-factors applied to the formulae used for determining the shear buckling resistance of plate girders. This was investigated in an attempt to understand whether a link exists between the reliability and complexity of the structural phenomenon considered during design. In chapter 8 the reliability levels achieved during the design of laterally restrained beams is considered. Since this is a relatively simple failure mechanism it can be used for comparison purposes with the work reported in this chapter.

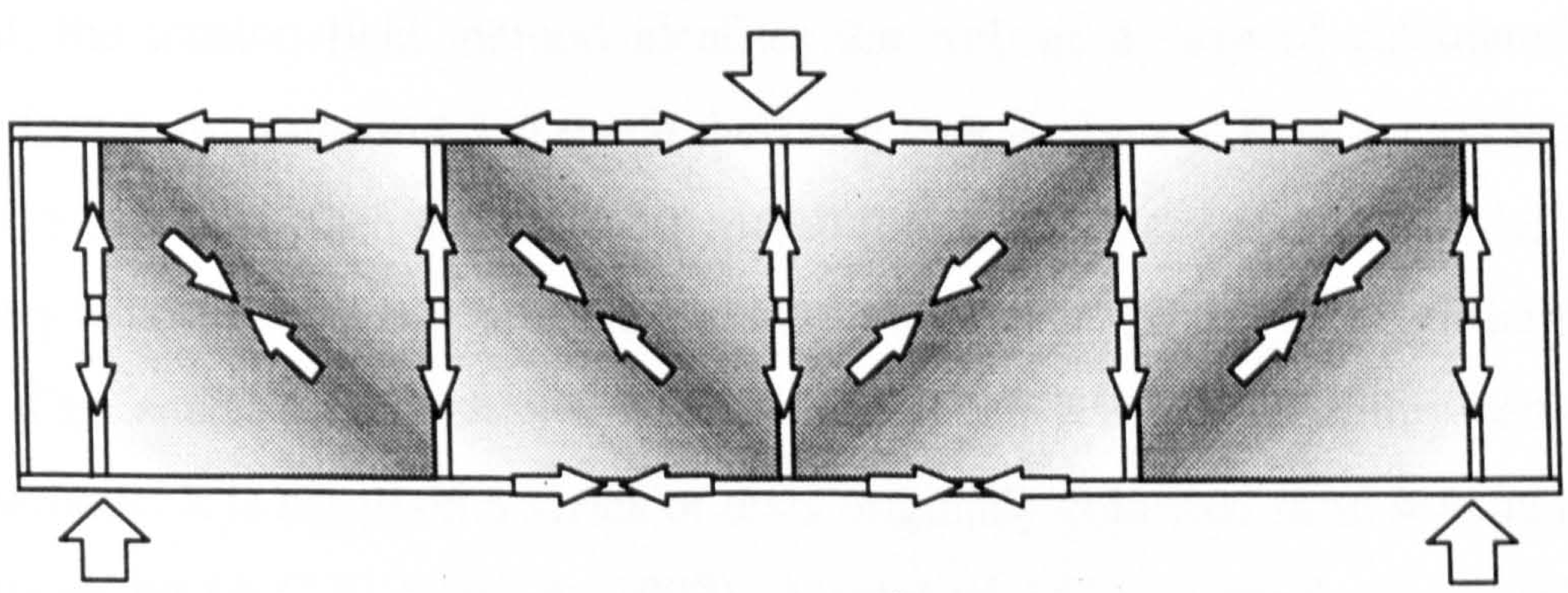


Fig. 7. 1: Tension field action

The Eurocode 3 design guidance for plate girders contains two alternative methods for determining the shear buckling resistance. The first method is intended as a simplified approach that produces a less economical result, although it is substantially easier to apply. This is known as the simple post-critical method and it is applicable to both stiffened and unstiffened girders. The second method involves greater complexity. It is intended to produce a more economical design, since it utilises what is known as the post-buckling reserve of strength that characterises the failure of plate girder test specimens. This method is known as the tension field method.

It has long been known that girders possess a considerable degree of post buckling strength. Early tests showed that plate girders with slender webs are capable of resisting considerably greater shear forces than are predicted from simple web buckling theory. This post-buckling reserve of strength was explained in 1916 by Rode, who developed the theory of tension field action. The theory is based on the concept that a plate girder with transverse stiffeners will behave in a truss-like fashion after initial web buckling. This concept is illustrated by Fig. 7. 1 showing a buckled web transferring tensile forces via a membrane-type action. Tension field theory was initially developed for the aeronautical industry, where the stiffening effect that the aluminium shell has on the aircraft frame is utilised.

Both the simple post-critical method and the tension field method take account of the post-buckling reserve of strength, although the design concepts used are radically different. The simple post-critical method utilises a simply supported plate model of the web. Buckling strength calculations are based on web slenderness, whilst attempting to account for the post-buckling strength for the most slender webs. By contrast, the tension field method idealises the web as a strip of calculated width. Tensile forces are transmitted in much the same way as the ties in an equivalent lattice girder. In addition, the tension field method makes an allowance for the loading necessary to cause a collapse mechanism involving plastic hinges in the girder flanges.

This analysis has attempted to determine the level of reliability achieved by both methods. It is based on a series of tests originally collected from several different laboratories by (A.C.B. Newark, 1993). A total of 143 test results are available; of which only 67 are suitable for calibration purposes. Test results were considered unsuitable either because of the incomplete recording of material properties and/or geometric properties, or because the ratio between the applied moment and the theoretical moment of resistance was greater than unity (i.e, $M_{exp}/M_{f.Rd}$ >=1.0). This limit was chosen since EC3 states that the design shear resistance need not be reduced to allow for the moment in the member, provided that the flanges are capable of resisting the design values of bending moment. By applying this limit the interaction between shear force and bending moment could be ignored.

The measures of material strength variability reported in Chapter 6 have been used for the purposes of calibration in which:

- mean f_y / nominal f_y = 1.16;
- V_{fy} = 0.05.

The following values taken from CEN (1991) have been used for the COV of geometric variables:

- V_d = 0.005;
- V_{tw} and V_{tf} = 0.05.

These values for geometric variability may well be conservative because it would seem likely that the variability of plate thickness would be less than the variability of the web thickness of hot rolled open sections (V_{tw}=0.044, chapter 5). However, no alternative data were available for calibration purposes.

7.2 THE SIMPLE POST-CRITICAL METHOD

7.2.1 DESIGN METHOD

According to the EC3 simple post-critical method, the design shear buckling resistance $V_{ba.Rd}$ is calculated directly as:

$$V_{ba.Rd} = d\, t_w \tau_{ba} / \gamma_{M1} \qquad (7.\ 1)$$

The simple post-critical shear strength (τ_{ba}) is dependent on the web slenderness ($\bar{\lambda}_w$) given as:

$$\bar{\lambda}_w = [\ (f_{yw} / \sqrt{3})/\tau_{cr}\ ]^{0,5} = \frac{d/\ t_w}{37{,}4\ \ \varepsilon\sqrt{k_\tau}} \qquad (7.\ 2)$$

where τ_{cr} is the elastic critical shear strength

and k_τ is the buckling factor for shear given by elastic buckling theory for simply supported plates. k_τ is equal to either:

(a) for webs with transverse stiffeners at the supports but no intermediate transverse stiffeners;

$$k_\tau = 5{,}34 \tag{7.3}$$

(b) for webs with transverse stiffeners at the supports and intermediate transverse stiffeners with a/d < 1;

$$k_\tau = 4 + 5{,}34 / (a/d)^2 \tag{7.4}$$

(c) for webs with transverse stiffeners at the supports and intermediate transverse stiffeners with a/d ≥ 1;

$$k_\tau = 5{,}34 + 4/(a/d)^2 \tag{7.5}$$

The simple post-critical shear strength τ_{ba} is determined as follows:

a) for stocky webs ($\bar{\lambda}_w \leq 0{,}8$) the shear strength at failure will equal the material shear strength, therefore;

$$\tau_{ba} = (f_{yw} / \sqrt{3}) \tag{7.6}$$

b) for webs of intermediate slenderness ($0{,}8 < \bar{\lambda}_w < 1{,}2$) failure is by a combination of yielding and buckling, in this case τ_{ba} is defined empirically as;

$$\tau_{ba} = [1 - 0{,}625\,(\bar{\lambda}_w - 0{,}8)\,]\,(f_{yw} / \sqrt{3}) \tag{7.7}$$

c) for slender webs ($\bar{\lambda}_w \geq 1{,}2$) buckling will occur prior to yielding, the post-buckling strength reserve is partially accounted for by the following expression;

$$\tau_{ba} = [0{,}9 / \bar{\lambda}_w\,]\,(f_{yw} / \sqrt{3}) \tag{7.8}$$

7.2.2 COMPARISON BETWEEN THE PREDICTED AND EXPERIMENTAL RESISTANCE

Listed below are the basic statistical parameters obtained from the comparison of experimental resistance with the predicted resistance determined using the simple post-critical design method. Calculations were based on the measured material and geometric properties. The test results used to determine this information are listed in detail in Appendix C.

$n = 67$

$V_b = 0.592$

$\overline{b} = 2.27$

$b_{max} = 5.66$

$b_{min} = 0.95$

where $b = V_{exp} / V_{ba.Rd}$

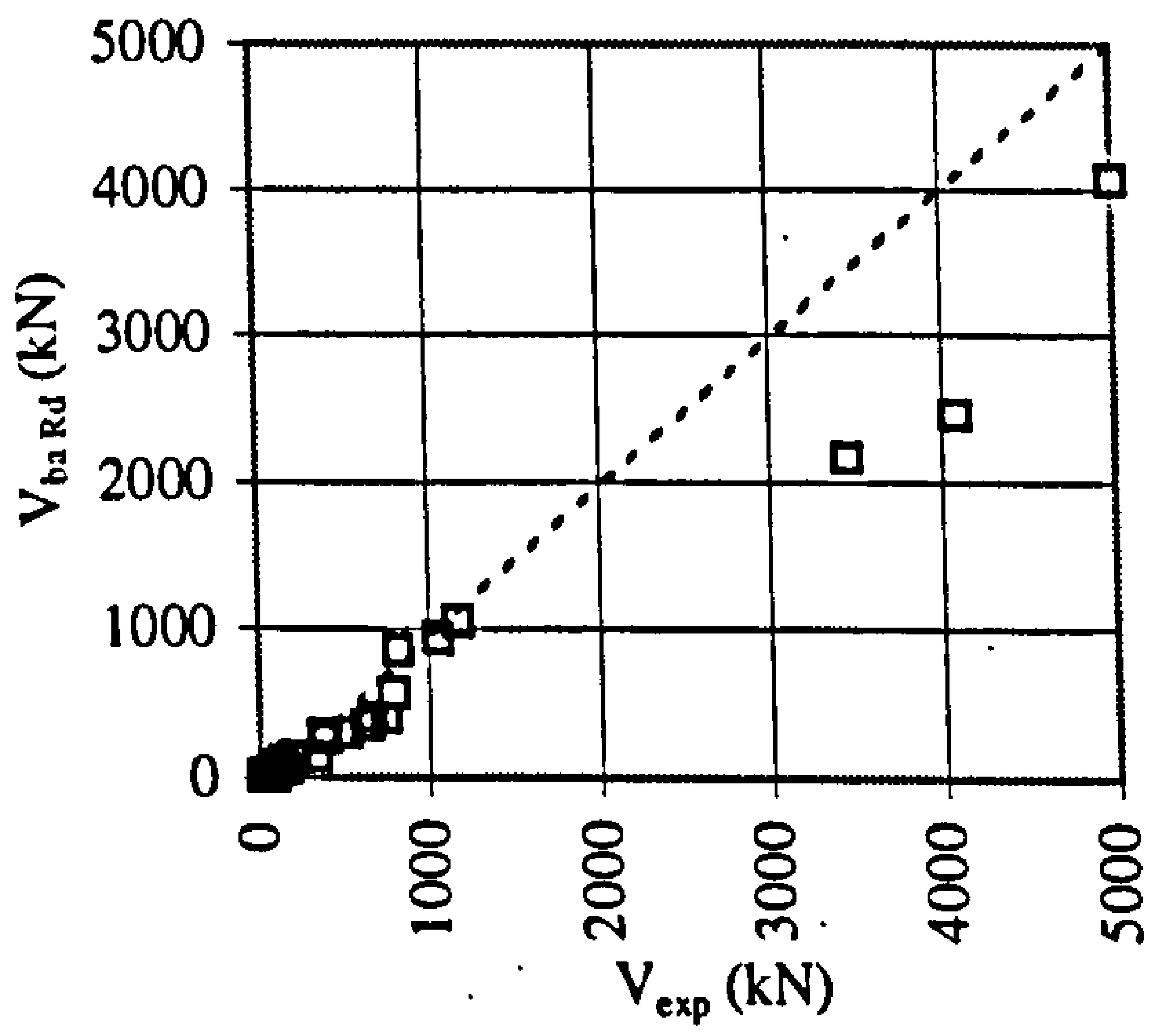


Fig. 7. 2: Predicted vs. experimental resistance

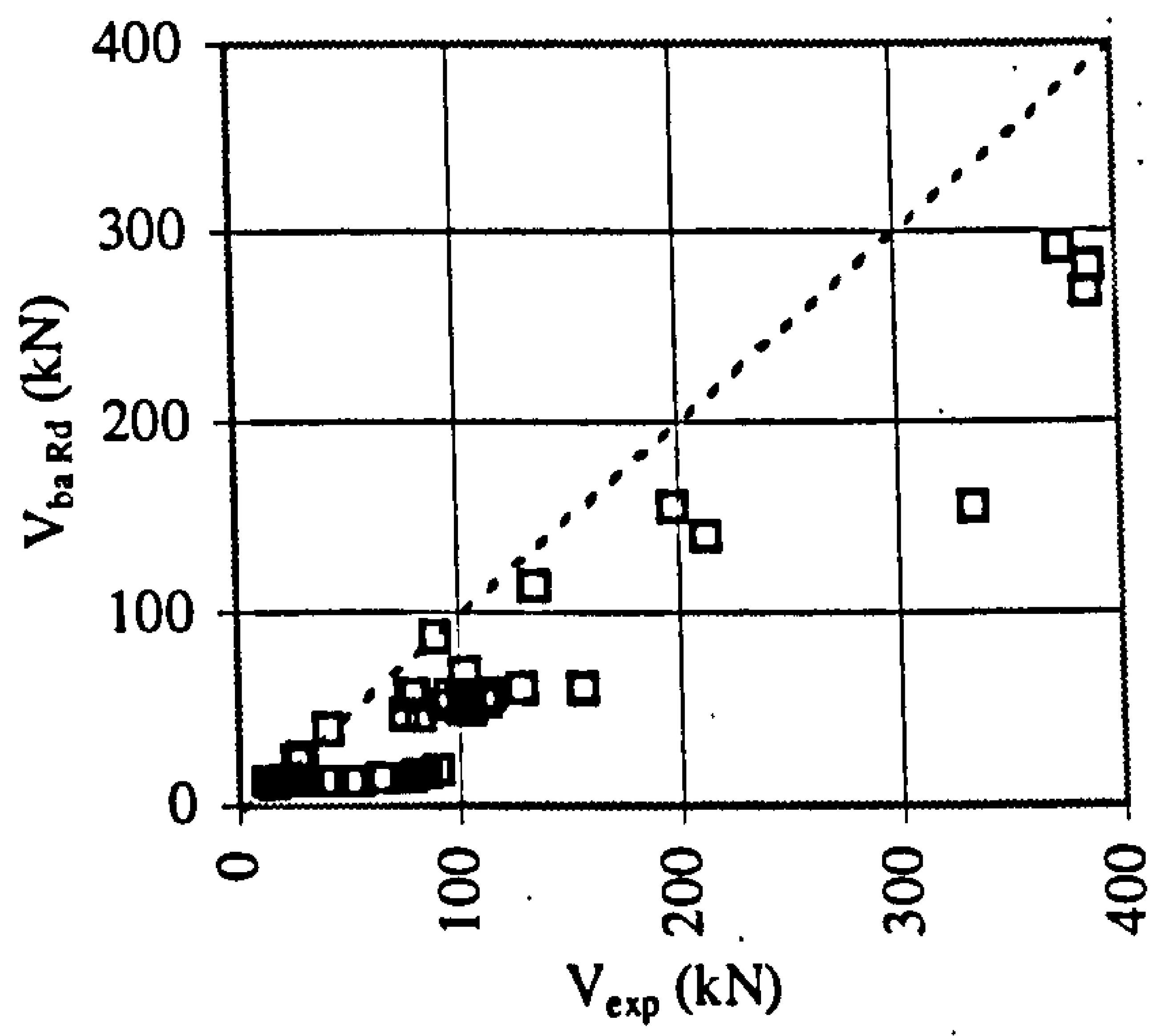


Fig. 7. 3: Predicted vs. experimental resistance

Fig. 7. 2 and Fig. 7. 3 show the graphs of predicted vs. experimental resistance; Fig. 7. 3 is a magnified section from Fig. 7. 2. If the design formulae provided a perfect model for resistance and the laboratory tests were ideal, then the points shown on the graphs would all lie on the bisector between the predicted and experimental resistances. Both the graphs and the data show the simple post-critical method is, under certain circumstances, unduly conservative ($\overline{b}$ = 2.27, b_{max} = 5.66). This high degree of conservatism results in a very high value of V_b (0.592) (a measure of the scatter between predicted and experimental resistance). This factor has a key influence on the numerical value of γ_R^*.

Most of the tests have been carried out to investigate tension field action. Since this phenomenon is most pronounced in girders with slender webs, most of the test specimens fall into the most slender category. Fig. 7. 4 illustrates that the simple post-critical method is capable of quite accurate predictions of resistance, when the web d/t ratio is less than 250. Where web slenderness is above this range, the over-conservatism of the method becomes important. Thus the method is incapable of predicting the ultimate resistance of girders with extremely slender webs and closely spaced transverse stiffeners (Fig. 7. 5), i.e. girders that lend themselves to tension field action.

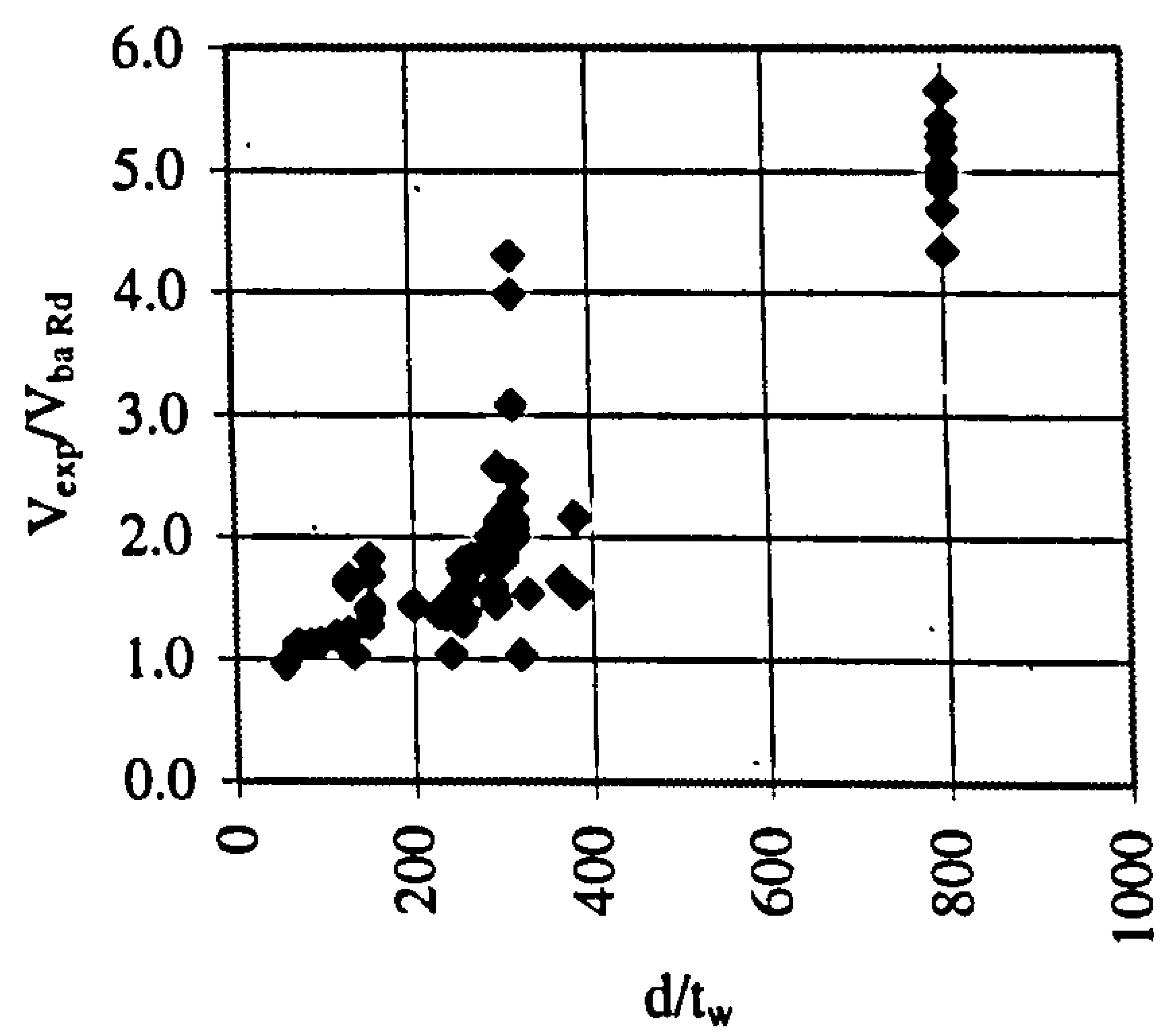


Fig. 7. 4: The effect web slenderness has on the accuracy of the simple post-critical method

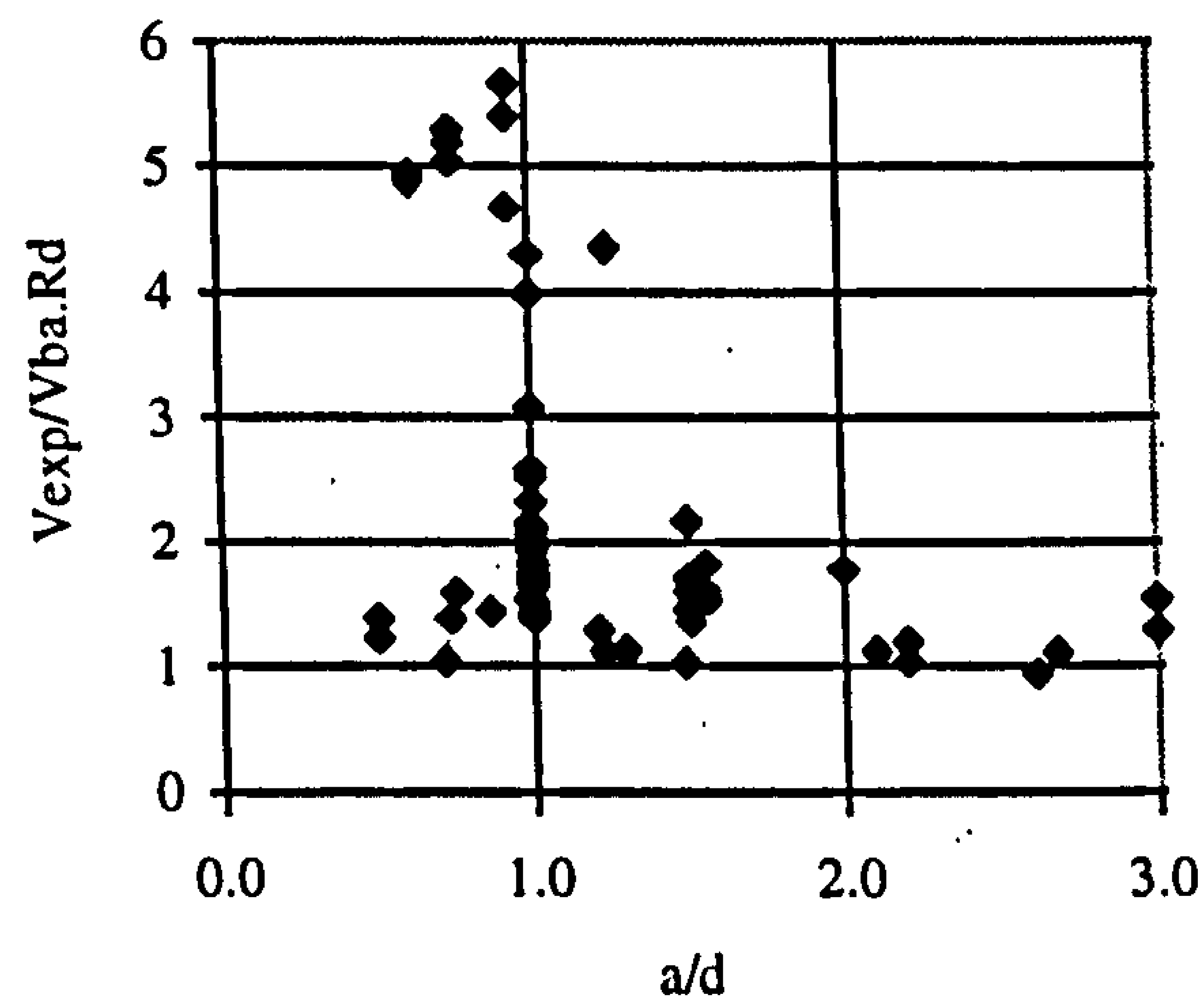


Fig. 7. 5: The effect aspect ratio has on the accuracy of the simple post-critical design method

At first sight the extreme conservatism of the method under certain conditions ought to be reflected in a low γ_R^* value necessary to achieve the target reliability. On average the method underestimated resistance by 127% and of the 67 tests only 1 had an experimental resistance falling below the predicted resistance. However the conservatism of the method results in a very high value of V_b. This increased V_b results in a unduly high value of γ_R^* despite the apparent conservatism of the method.

Fig. 7. 6 shows the actual frequency distribution calculated for the set of 67 test results. Plotted alongside is the log-normal p.d.f. calculated using the observed mean and standard deviation of b; this is the distribution used for determining γ_R^* according to the Annex Z method. For an accurate and representative value of γ_R^* to be calculated, it is essential that the lower tail of the log-normal p.d.f. provides a reasonable reflection of the observed variability of b. Fig. 7. 6 shows that the lower tail of the log-normal distribution provides a poor and unduly conservative model of

the lower tail. This conservatism will be reflected in a high value of γ_R^* (above 2.0) if calibration is carried out using the entire set of 67 test results. Clearly some sorting of the data is necessary so that a realistic γ_R^* value can be established.

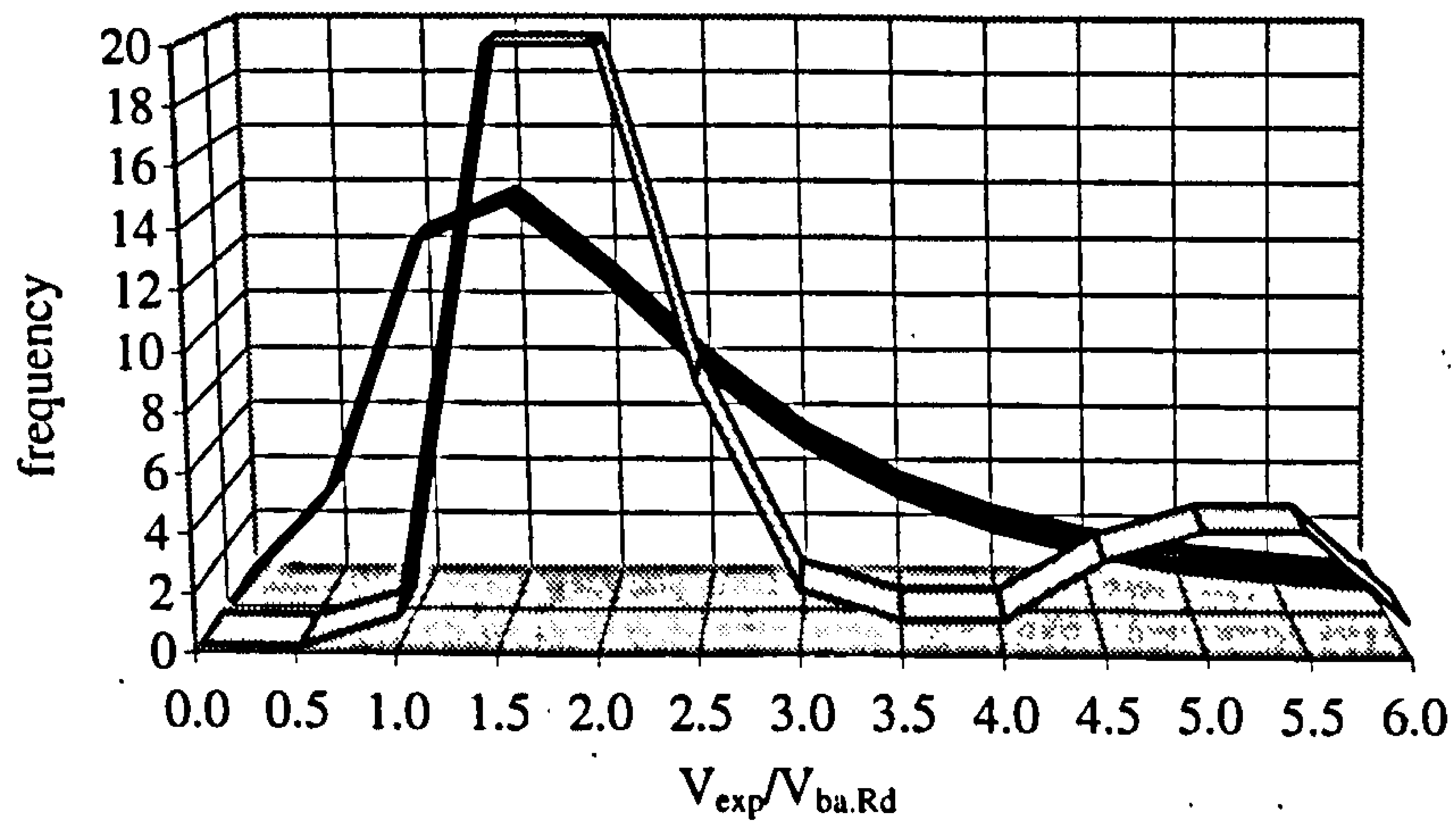


Fig. 7. 6: The actual probability distribution (in front) compared with the log-normal distribution (behind) based on all test results ($\overline{b}$ = 2.27, V_b =0.592).

The lower tail of the observed distribution can be more accurately modelled using the log-normal p.d.f. by cutting out data from the set where b is greater than a specified value. The effect is to reduce both $\overline{b}$ and V_b. Reducing V_b via this method more than outweighs the conservative effect of reducing $\overline{b}$. This is the method by which the distributions shown in Fig. 7. 7 have been determined.

Fig. 7. 7 illustrates that the higher the cut-off point the better the fit on the lower tail of the distribution. Therefore, by omitting the high values of b an improved model of the lower tail of the observed distribution is obtained. Ideally this type of data selection would be unnecessary. However, the method excludes only the most conservative test results that result in a high γ_R^* value, not because the design method is unsafe, but because of the limitations of the log-normal p.d.f.. Thus, by omitting high values of b an improved model is achieved for the lower tail of the observed distribution. In addition, the introduction of a cut-off point does have a physical basis. Fig. 7. 4 shows that the method produced unduly conservative predictions of strength for girders with the most slender webs, girders that designers are unlikely to specify but which have been tested in order to investigate tension field action.

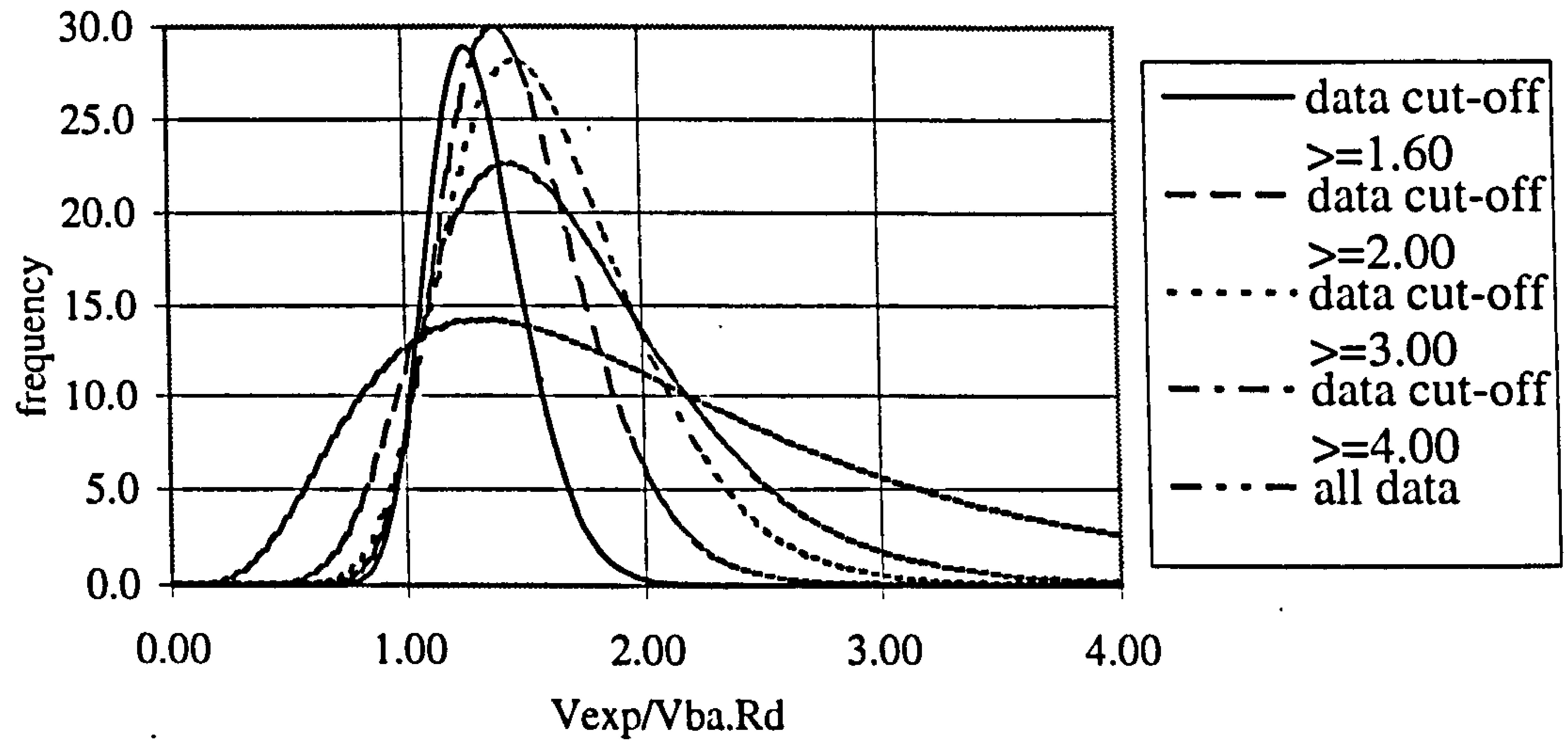


Fig. 7. 7: Log-normal models of resistance distribution

Introducing a cut-off point reduces the size of the sample, with a resulting increase in the t-factor used in the calculation of γ_R^*. Therefore, a cut-off point needs to be chosen that will produce the optimum value for γ_R^*. Table 7. 1 shows the effect that the cut-off point has on the 95% confidence limit of b; a factor that provides a good indication of the effect on γ_R^*. According to this analysis the most appropriate cut-off point is 1.90.

data cut-off greater than	n	$\overline{b}$	b max.	b min.	σ_b	V_b	95% conf. limit
1.1	4.00	1.02	1.04	0.95	0.04	0.04	0.95
1.2	10.00	1.09	1.20	0.95	0.07	0.06	0.97
1.6	28.00	1.31	1.60	0.95	0.20	0.15	0.99
1.8	35.00	1.39	1.80	0.95	0.24	0.17	1.00
2.0	43.00	1.49	2.00	0.95	0.30	0.20	1.00
2.5	51.00	1.59	2.32	0.95	0.36	0.23	0.99
3.0	53.00	1.63	2.58	0.95	0.40	0.25	0.97
4.0	55.00	1.69	3.98	0.95	0.54	0.32	0.81
5.0	63.00	2.07	5.04	0.95	1.12	0.54	0.23
5.7	67.00	2.27	5.66	0.95	1.34	0.59	0.06

Table 7. 1: The effect the data cut-off point has on the correction factor b

7.2.3 THE CALCULATION OF γ_R^*

In terms of calibration, both the simple post-critical and the tension field methods are relatively complex because the design procedure differs according to the web slenderness $\overline{\lambda}_w$. Therefore, γ_R^* will need to be determined separately for girders with $\overline{\lambda}_w$ falling into each of the following categories, i.e. $\overline{\lambda}_w \leq 0.8$, $0.8<\overline{\lambda}_w<1.2$, $\overline{\lambda}_w \geq 1.2$.

Two separate girders (named A and B) have been calibrated in order to check that γ_R^* is not overly sensitive to the girder proportions. Web slenderness was varied for both girders. The basic data for the variables are listed in Table 7. 2 and Table 7. 3.

	a mm	d mm	b mm	t_w mm	t_f mm	f_y mm
Mean	2000	1000	200	*	35	319
Nom.	2000	1000	200	*	35	275
COV	0.005	0.005	0.005	0.005	0.050	0.05
std.	10.00	5.00	1.00	*	1.75	14.67

* dependent on $\overline{\lambda}_w$

Table 7. 2: The basic data for girder type A

	a mm	d mm	b mm	t_w mm	t_f mm	f_y mm
Mean	2800	1000	300	*	38	412
Nom.	2800	1000	300	*	38	355
COV	0.005	0.005	0.005	0.005	0.050	0.05
std.	10.00	5.00	1.00	*	1.75	14.67

* dependent on $\overline{\lambda}_w$

Table 7. 3: The basic data for girder type B

Using the data listed in the above tables, V_{rt} (a measure of the uncertainty due to variations in geometric and material properties) can be quantified (Bijlaard *et al,* 1988). Equation (7.9) is used for determining σ_{rt}. Large changes in certain basic variables may have relatively small effects on overall resistance. Likewise resistance may be highly sensitive to small changes in other basic variables. In the case of plate girders small changes in web thickness will have a large impact on the shear buckling resistance. Conversely, large variations in flange thickness have virtually no effect on shear buckling resistance. By slightly altering the value of each basic variable and determining the corresponding change in resistance, equation (7.9) weights the variability of each basic variable depending on corresponding sensitivity. Using this method a representative value for σ_{rt} can be determined. Equation (7.9) presents an established method for calculating σ_{rt}. Alternative methods are available such as those described in (Thoft-Christensen and Baker, 1982).

$$\sigma_{rt}^2 = \left(\frac{\delta R}{\delta a}\sigma_a\right)^2 + \left(\frac{\delta R}{\delta d}\sigma_d\right)^2 + \left(\frac{\delta R}{\delta b}\sigma_b\right)^2 + \left(\frac{\delta R}{\delta t_w}\sigma_{t_w}\right)^2 + \left(\frac{\delta R}{\delta t_f}\sigma_{t_f}\right)^2 + \left(\frac{\delta R}{\delta f_y}\sigma_{f_y}\right)^2 \quad (7.9)$$

Applying the basic variables contained in Table 7. 2 with $\overline{\lambda}_w$ set at 1.15 and altering each basic variable by 5% gives equation (7.10) below. Thus, γ_R^* is determined as follows:

$$\sigma_{rt}^2 = \left(\frac{1555-1545}{0.05\text{x}2000}\text{x}0.005\right)^2 + \left(\frac{1555-1571}{0.05\text{x}1000}\text{x}0.005\right)^2 + \left(\frac{1555-1555}{0.05\text{x}200}\text{x}0.005\right)^2 +$$

$$\left(\frac{1555-1704}{0.05\text{x}10.8}\text{x}0.05\right)^2 + \left(\frac{1555-1555}{0.05\text{x}35.0}\text{x}0.05\right)^2 + \left(\frac{1555-1596}{0.05\text{x}319}\text{x}0.05\right)^2 \quad (7.10)$$

σ_{rt} = 153.6 kn.

$$V_{rt} = \frac{\sigma_{rt}}{r_m} = \frac{153.6}{1555} = 0.099 \quad (7.11)$$

$$V_R = \sqrt{V_b^2 + V_{rt}^2} = \sqrt{0.17^2 + 0.099^2} = 0.197 \quad (7.12)$$

$$\gamma_R^* = \frac{1428\text{x}\exp(0.5\text{x}0.197^2 + 3.04\text{x}0.197)}{1.39\text{x}1555} = 1.28 \quad (7.14)$$

Using this technique, γ_R^* has been calculated for girders A and B with a variety of different web slendernesses. The results of this analysis are shown in Fig. 7. 8.

The reliability of the simple post-critical method is closely linked to web slenderness, although it appears unaffected by the overall girder configuration.

Arguably, γ_R^* should be calibrated for the worst possible design situation. Fig. 7. 8 shows that reliability falls for girders with a web slenderness exceeding 1.2. Thus, the γ_R^* factor calculated for girders falling in this category should be applied to the γ_{M1} factor applied to the simple post-critical method. According to this analysis γ_{M1} should equal 1.3 which is significantly above the value of γ_{M1} specified in EC3. The UK NAD sets γ_{M1} equal to 1.05, whilst the EC3 boxed value for γ_{M1} is equal to 1.10.

The calibration process is expected to achieve a target reliability of resistance being less than design resistance of 1 in 845. Fig. 7. 9 shows the probability of resistance being less than design resistance for both the UK NAD and EC3 Boxed values of γ_R^*. It should be remembered that these values are realistic since the log-normal distribution of resistance provides an accurate model of the lower tail of the observed distribution of resistance due to the introduction of the data cut off point discussed earlier in this section.

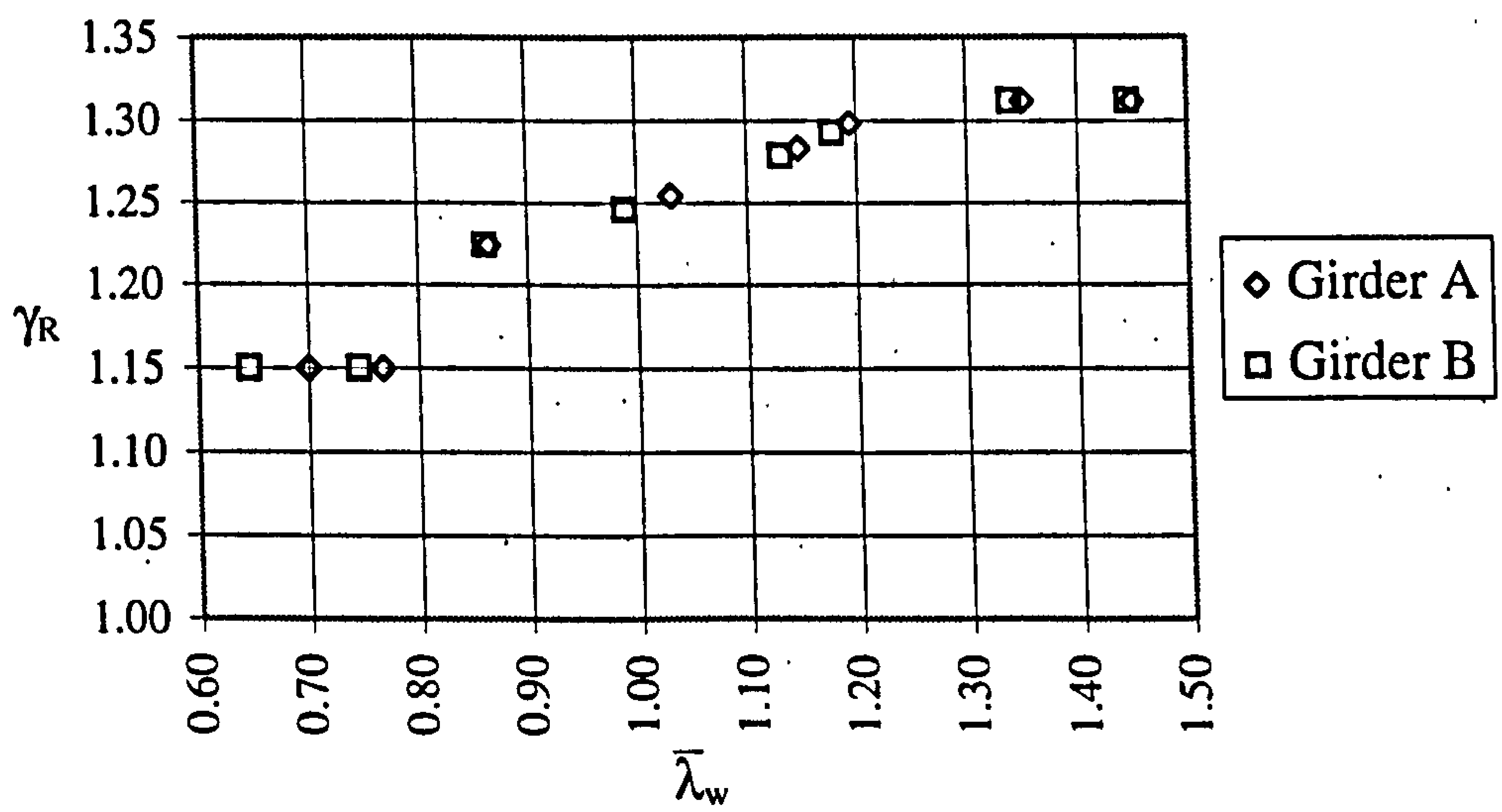


Fig. 7. 8: The value of γ_R^* necessary to achieve the target reliability for girders with differing web slenderness $\bar{\lambda}_w$

Fig. 7. 9 illustrates the negative effects that low values of γ_{M1} have on plate girder reliability. According to the UK NAD value of γ_{M1} the simple post-critical method achieves a probability of resistance being less than design resistance of approximately 1 in 50, when web slenderness exceeds 1.2. Since the target reliability is 1 in 845, the achieved reliability level is well below that required.

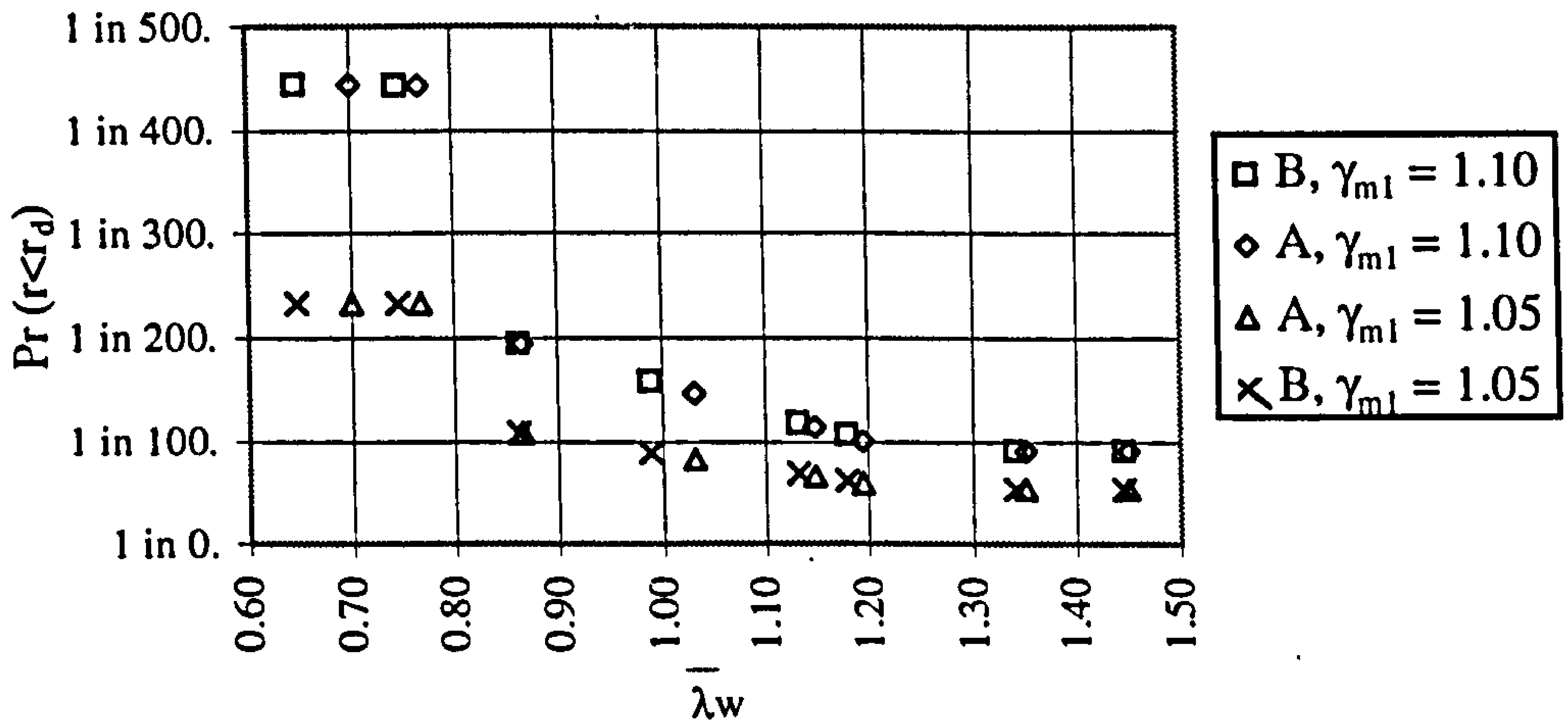


Fig. 7. 9: The probability of resistance being less than design resistance for girders with differing values of $\bar{\lambda}_w$ (target probability of $r<r_d$ is 1 in 845)

7.3 THE TENSION FIELD METHOD

Unlike the simple-post critical method, the tension field method can only be used for plate girders with transverse stiffeners. The tension field method, also known as the Cardiff Method was substantially developed in the 1970's by Rockey, Evans and Porter. Basler (1960) first developed design formulae utilising post-buckling strength, although he did not account for the contribution of flange rigidity to the ultimate load capacity - a factor that was later included by Rockey and Skaloud (1968). Although the tension field method appears complex in comparison to the simple post-critical method, it does represent the most accurate method for utilising the substantial post-buckling reserve of strength that characterises plate girder failure.

The ultimate shear buckling resistance is determined by the addition of three distinct modes through which the girder resists the applied load. These components of response to loading are:

1. Beam action prior to buckling, where $\tau<\tau_{cr}$, see Fig. 7. 10a.
2. Tension field action post-buckling, see Fig. 7. 10b.
3. Frame action through the development of four plastic hinges in the compression and tension flanges, Fig. 7. 10c.

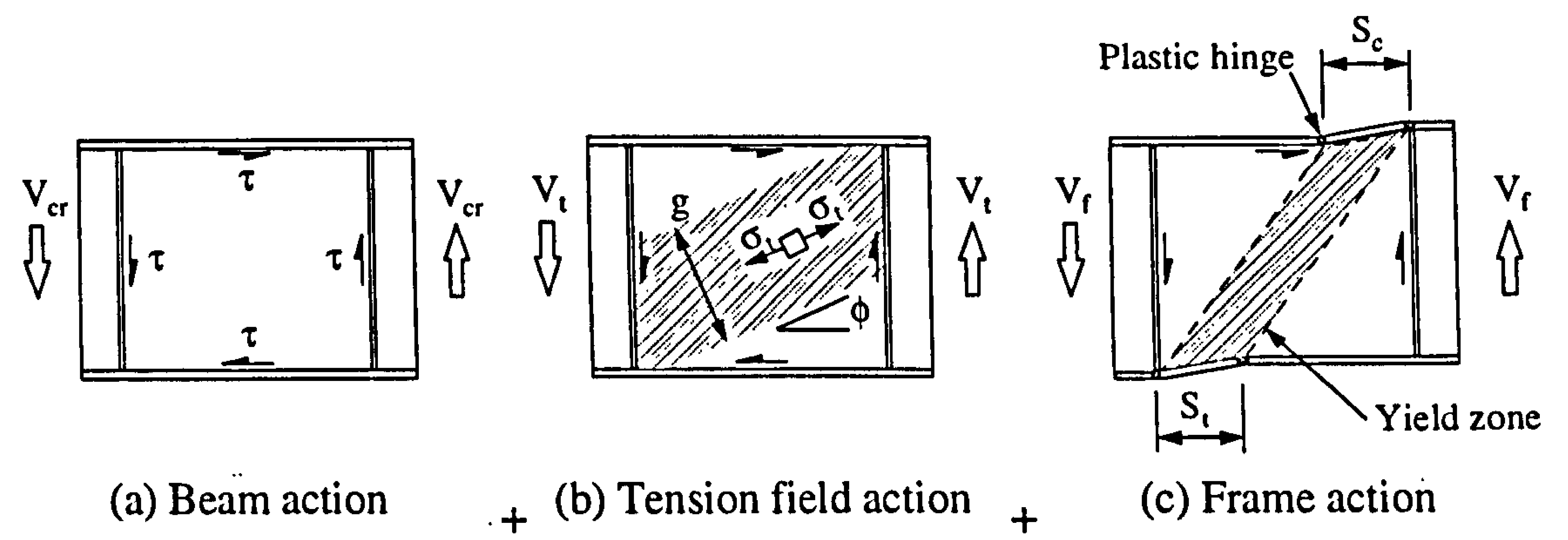


Fig. 7. 10: Contributions to shear resistance accounted for in the tension field method

7.3.1 DESIGN METHOD

According to the EC3 tension field method the design shear buckling resistance $V_{bb.Rd}$ should be obtained from:

$$V_{bb.Rd} = [(d\ t_w\ \tau_{bb}) + 0{,}9\ (g\ t_w\ \sigma_{bb} \sin\phi)]/\gamma_{M1} \qquad (7.\ 15)$$

The strength of the tension field (σ_{bb}) is obtained from:

$$\sigma_{bb} = [f_{yw}^2 - 3\tau_{bb}^2 + \psi^2]^{0,5} - \psi \qquad (7.\ 16)$$

in which

$$\psi = 1{,}5\tau_{bb} \sin 2\phi \qquad (7.\ 17)$$

where ϕ is the inclination of the tension field

g is the width of the tension field, see figure 5.6.1

and τ_{bb} is the initial shear buckling strength.

The initial shear buckling strength τ_{bb} should be determined as follows:

a) when $\bar{\lambda}_w \leq 0{,}8$;

$$\tau_{bb} = (f_{yw} / \sqrt{3}) \qquad (7.\ 18)$$

b) when $0{,}8 < \bar{\lambda}_w < 1{,}25$;

$$\tau_{bb} = [1 - 0{,}8\ (\bar{\lambda}_w - 0{,}8)\ ]\ (f_{yw} / \sqrt{3}) \qquad (7.\ 19)$$

c) when $\bar{\lambda}_w \geq 1{,}25$;

$$\tau_{bb} = [1 / \bar{\lambda}_w^{\,2}]\,(f_{yw} / \sqrt{3}) \qquad (7.20)$$

The width of the tension field g is given by:

$$g = d \cos\phi - (a - s_c - s_t)\sin\phi \qquad (7.21)$$

s_c and s_t are the anchorage lengths of the tension field along the compression and tension flanges respectively and are obtained from:

$$s = \frac{2}{\sin\phi}\left[\frac{M_{Nf.Rk}}{t_w\ \sigma_{bb}}\right]^{0,5} \quad \text{but } s \leq a \qquad (7.22)$$

$M_{Nf.Rk}$ is the reduced plastic resistance moment of the flange.

7.3.2 COMPARISON BETWEEN THE PREDICTED AND EXPERIMENTAL RESISTANCE

Eurocode 3 lists a series of criteria that limit the range of girder configurations within which the tension field method can be applied. For example the method cannot be used if a/d (spacing of web stiffeners / depth of web) is <1.0 or >3.0. Of the 86 available plate girder test results, only 44 satisfy the code requirements. The basic statistical parameters derived from the comparison of experimental resistance with the predicted resistance are listed below.

$n = 44$

$V_b = 0.111$

$\bar{b} = 1.160$

$b_{max} = 1.304$

$b_{min} = 0.835$

where $b = V_{exp} / V_{ba.Rd}$

These provide the numerical measure of the accuracy of the method for use in calibration. Fig. 7. 11 and Fig. 7. 12 illustrate graphically the accuracy of the method. The detailed experimental and predicted resistances are tabulated in Appendix C.

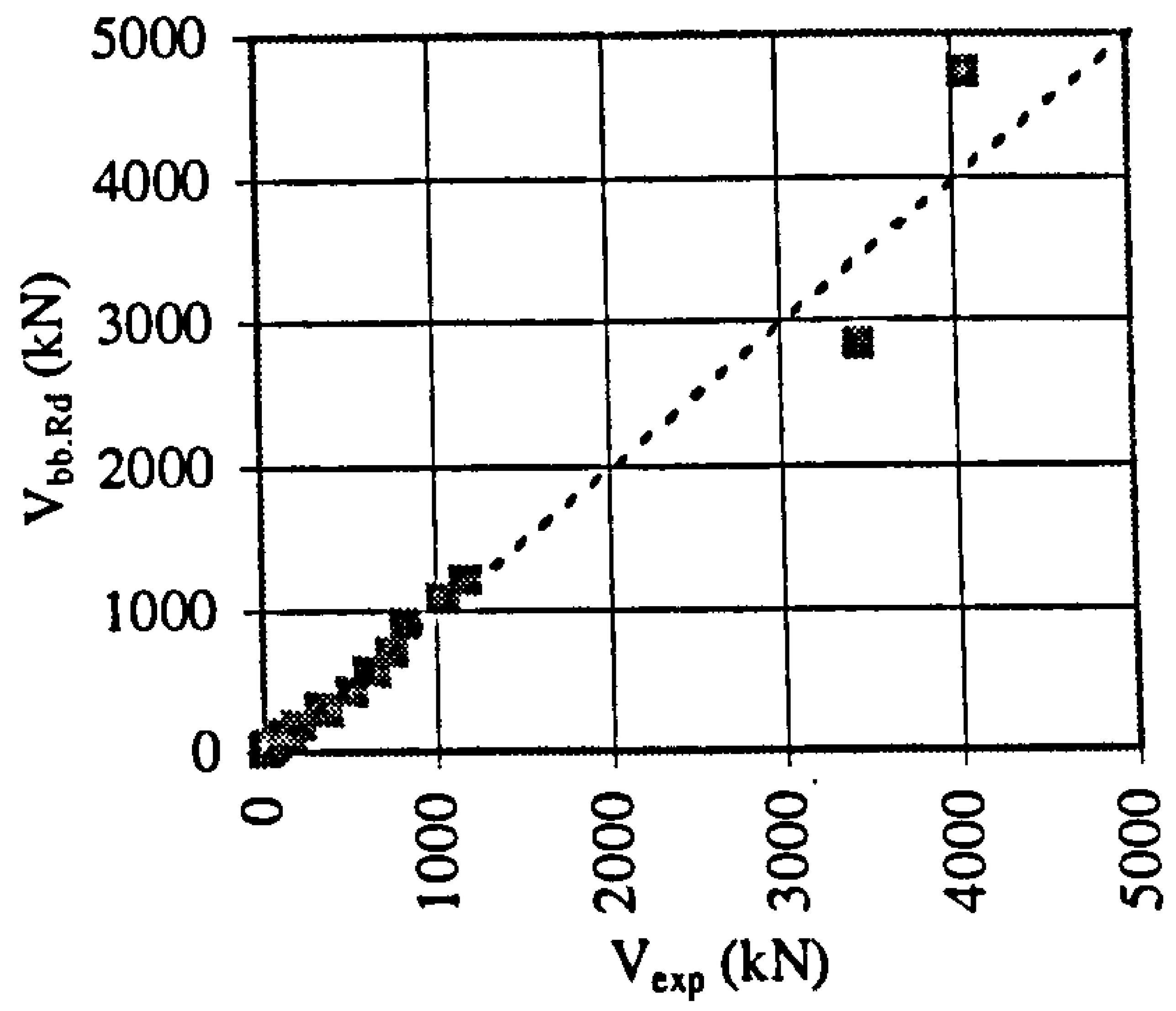


Fig. 7. 11 Predicted vs. experimental resistance

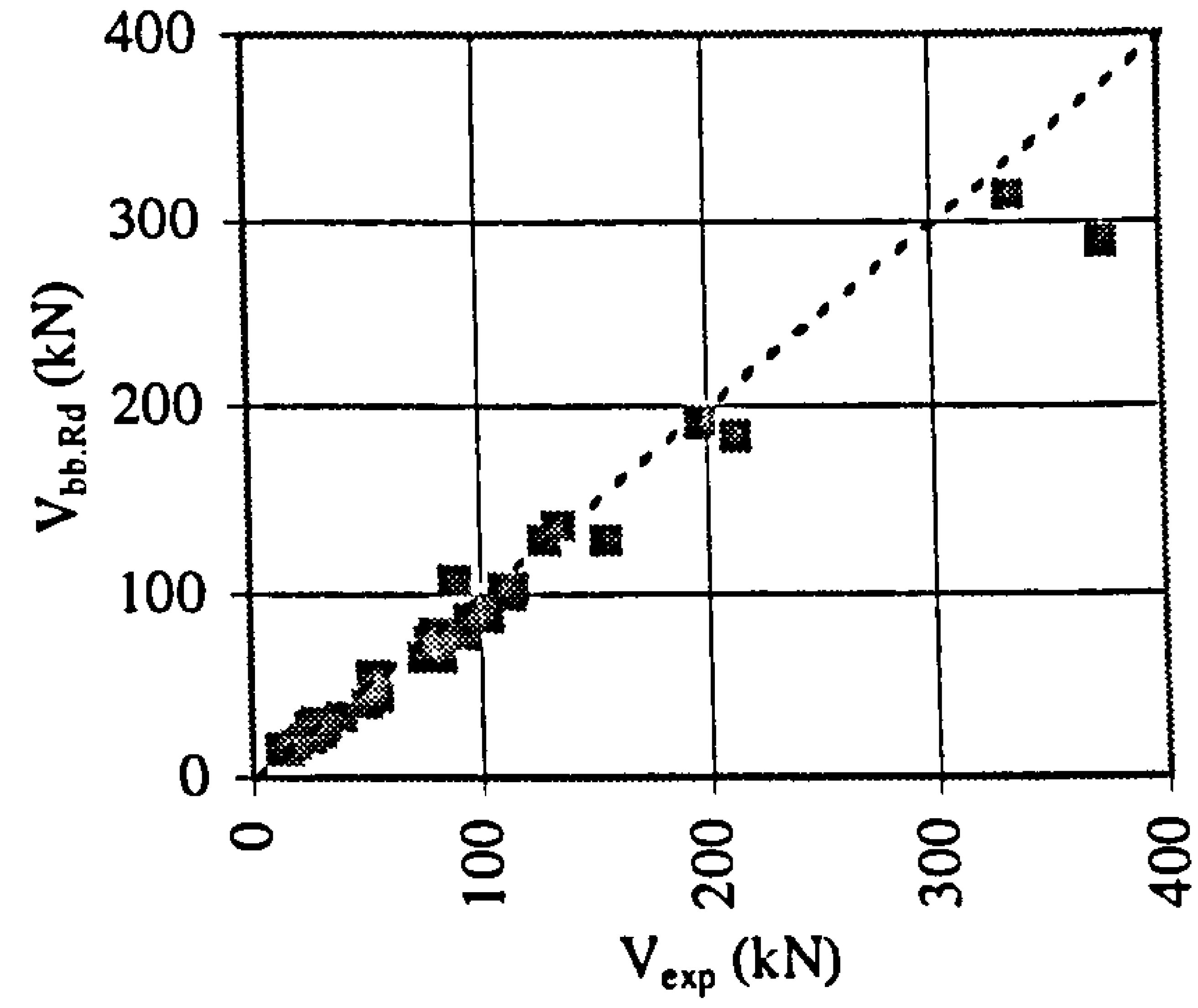


Fig. 7. 12 Predicted vs. experimental resistance

Compared with the simple post-critical method, the tension field method shows an improved degree of accuracy according to these test results. Factor V_b is comparatively low (0.111), and on average the method underestimated resistance by 16%. The simple post-critical method was incapable of predicting the true post-buckling reserve of strength for girders with slender webs. Since the tension field method directly utilises the post-buckling reserve of strength, in theory it should be capable of accurately predicting the resistance of girders with slender webs. Fig. 7. 14 confirms that resistance function accuracy is unaffected by web slenderness. Thus, the method is capable of predicting the post-buckling reserve of strength for the girders that were poorly modelled by the simple post-critical method. It is of interest to see if the aspect ratio (a/d) has an effect on the accuracy of the tension field method, since it is only applicable to girders with a limited range of aspect ratios (1.0<a/d<3.0). This analysis confirms that this range of aspect ratios is reasonable, as no change in the accuracy of the model is observed within that range.

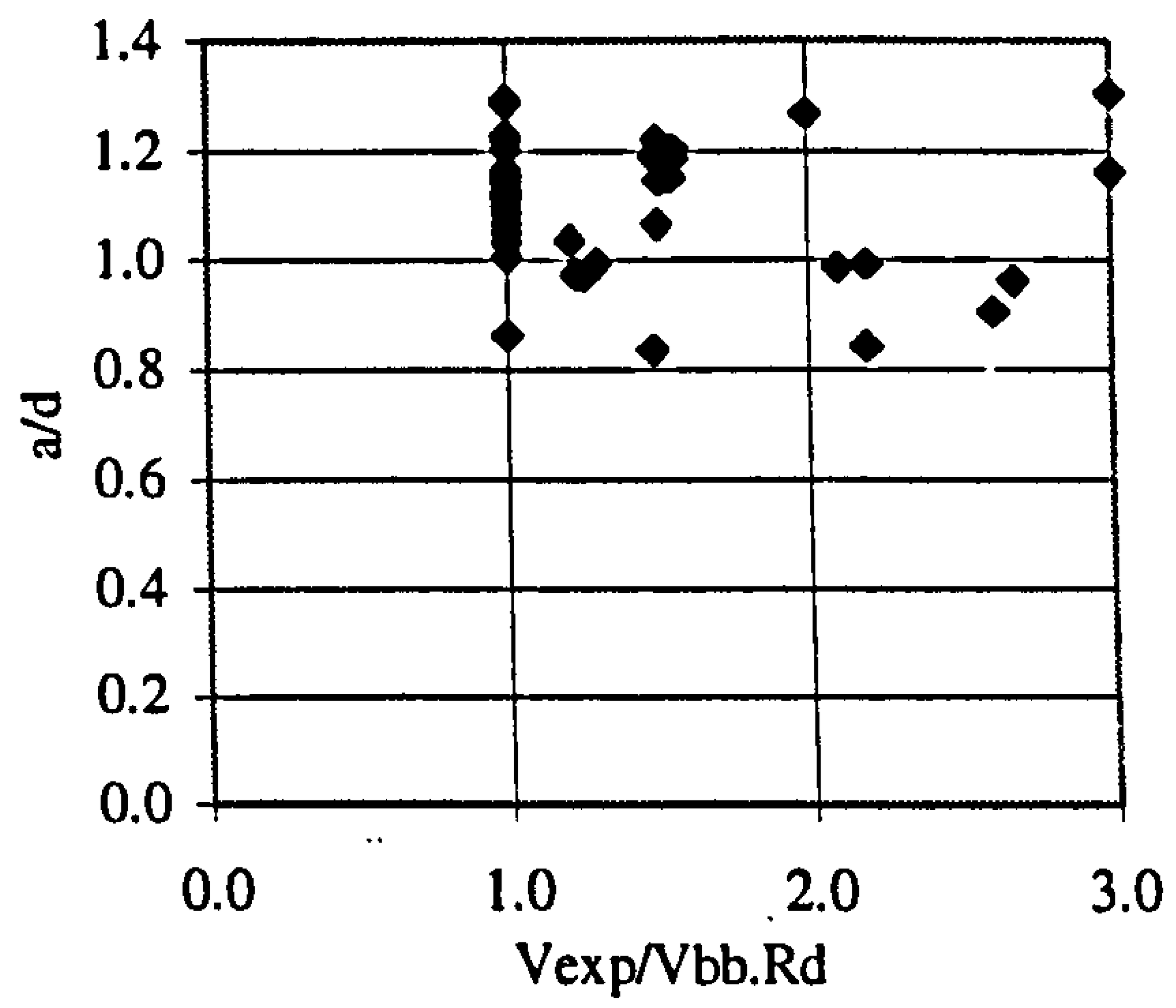


Fig. 7. 13: The influence aspect ratio has on the accuracy of the tension field method

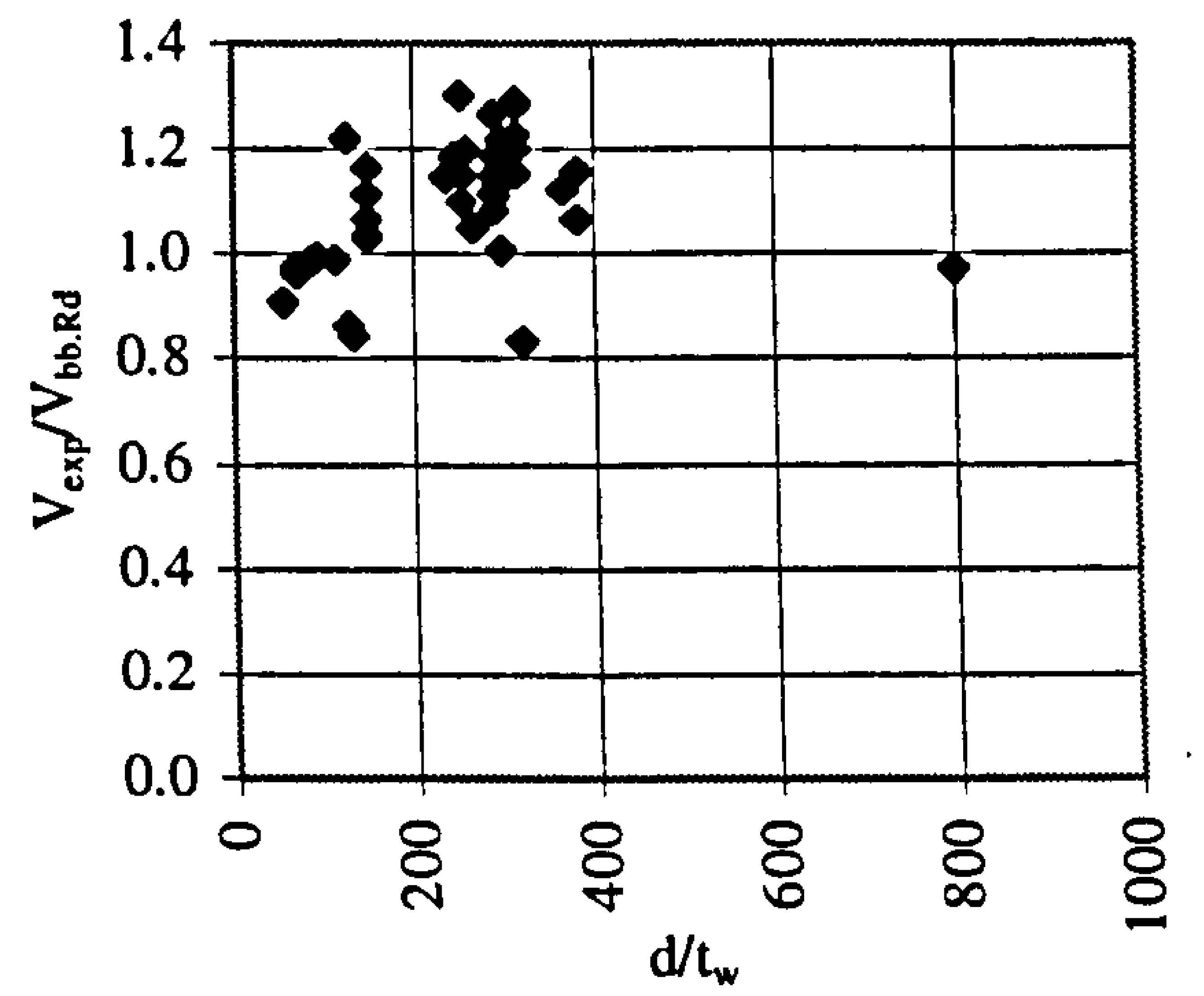


Fig. 7. 14: The influence web slenderness has on the accuracy of the tension field method

The Annex Z method of calibrating γ^*_R uses the log-normal p.d.f. to model the probability distribution of resistance. Fig. 7. 15 shows that the log-normal p.d.f. fits the observed variability of resistance of these test results with a high degree of accuracy. In particular, the lower tails of the observed and log-normal distributions match well. This is an important factor since it affects the Annex Z method's ability to calibrate a γ^*_R-factor to achieve the desired reliability level.

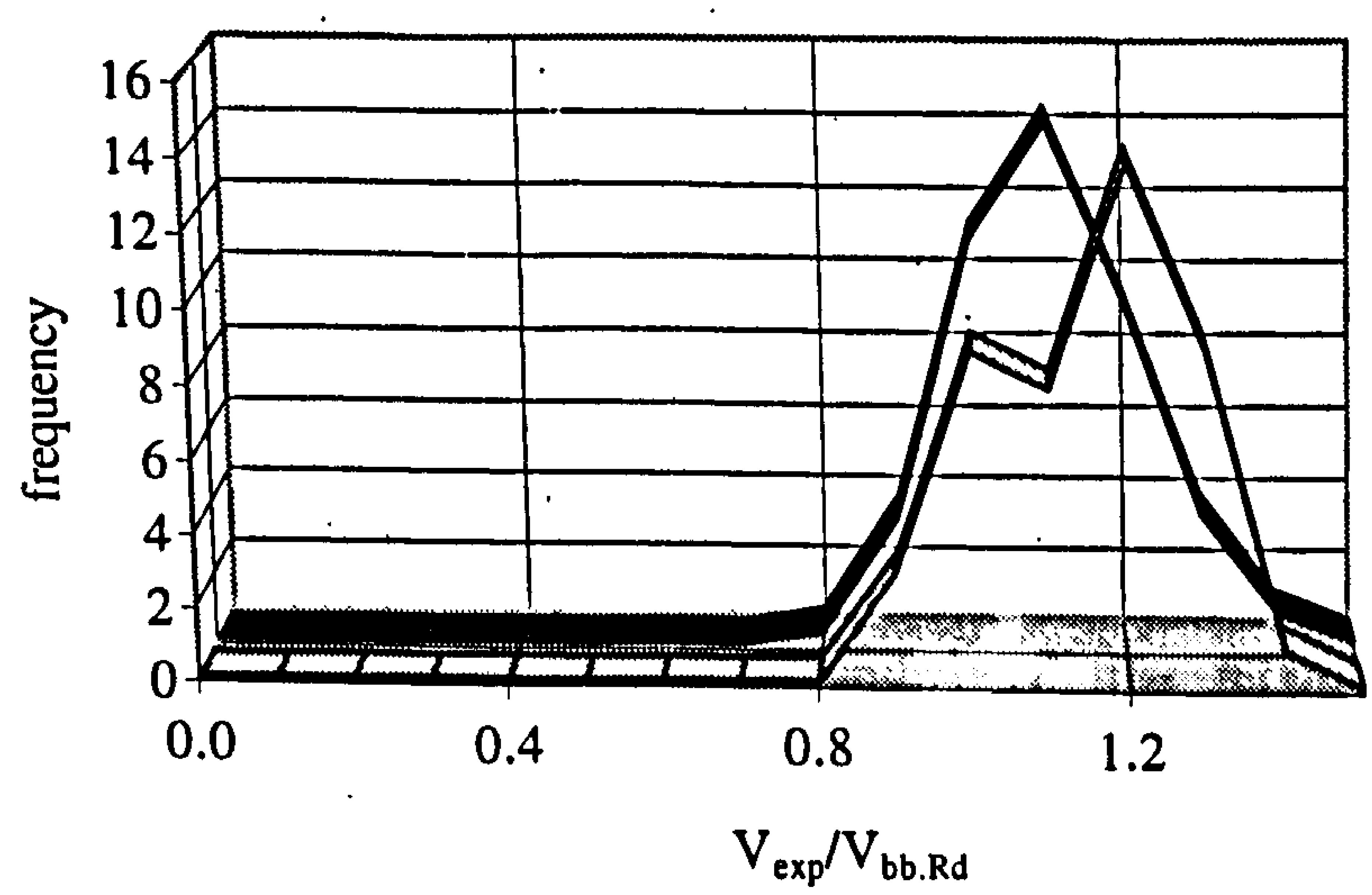


Fig. 7. 15: Comparison of the log-normal p.d.f. (behind) and the observed variability of $V_{exp}/V_{bb.Rd}$ (in front)

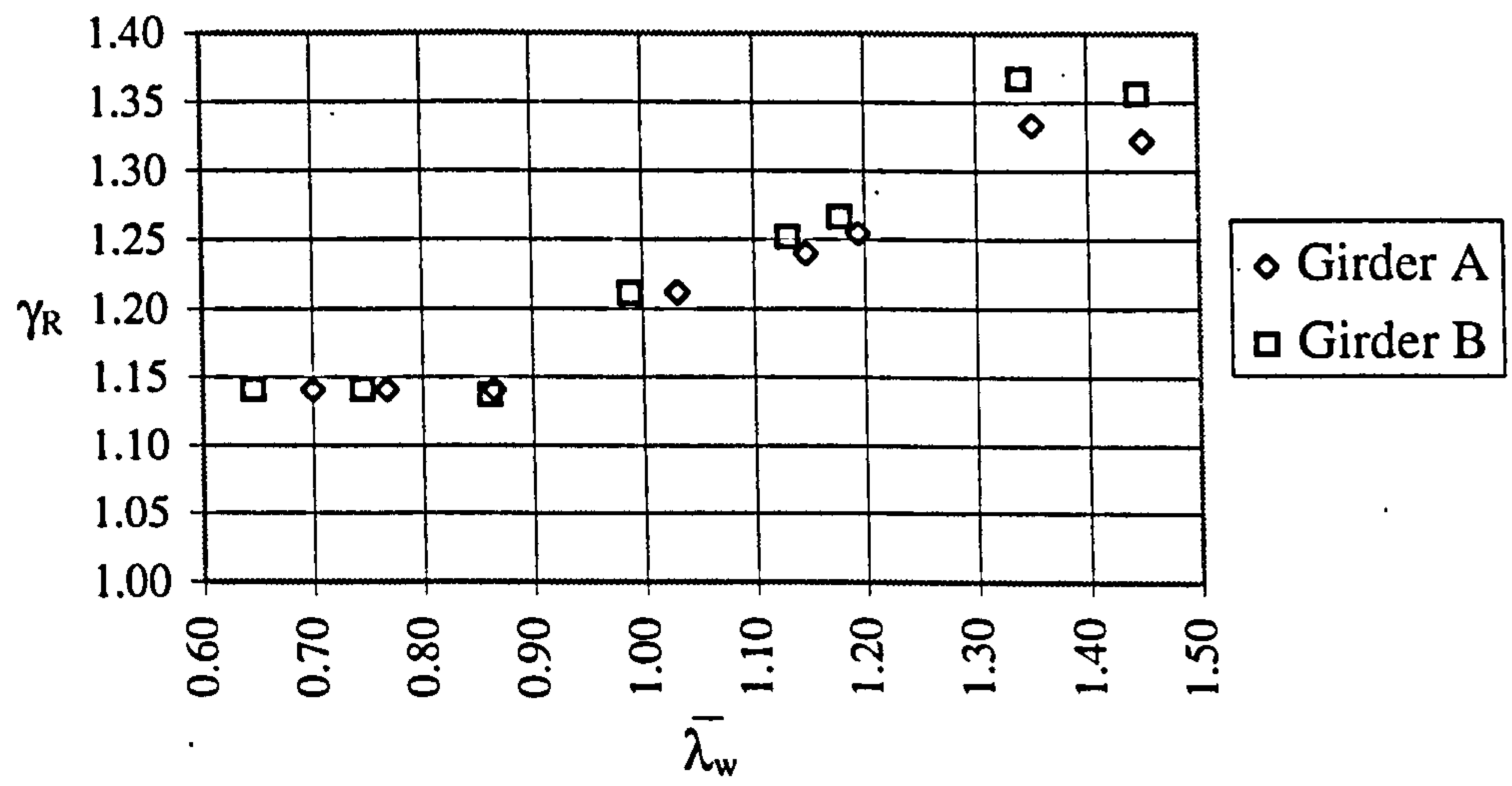


Fig. 7. 16: The value of γ_R^* necessary to achieve the target reliability for girders with differing web slenderness $\bar{\lambda}_w$

7.3.3 THE CALCULATION OF γ_R^*

The calibration procedure of the tension field method is identical to that of the simple post-critical method. Fig. 7. 16 shows values of γ_R^* determined for girders A and B whose dimensions are listed in Table 7. 2 and Table 7. 3 respectively. The results of the calibration are illustrated in Fig. 7. 16 and Fig. 7. 17.

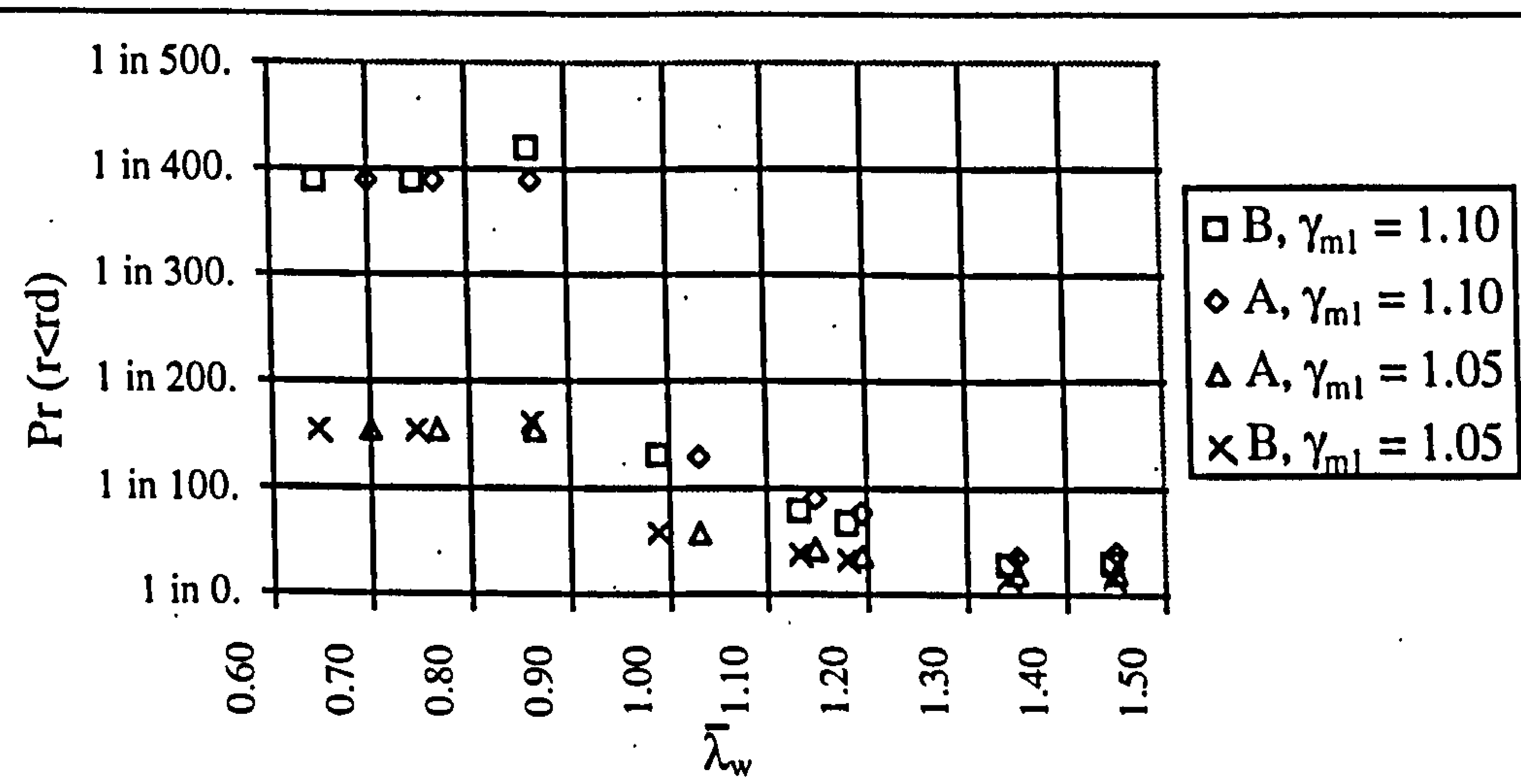


Fig. 7. 17: The probability of resistance being less than design resistance for girders with differing values of $\bar{\lambda}_w$ (target probability of $r<r_d$ is 1 in 845)

In common with the simple post-critical method, the reliability of plate girders designed in accordance with the tension field method is closely liked to web slenderness. According to this analysis, the value of 1.05 set by the UK NAD for γ_{M1} is too low. Rather, a γ_{M1}-factor of 1.35 is necessary to achieve the target reliability for the worst possible design situation, that where web slenderness exceeds 1.25.

Fig. 7. 17 shows that a γ_{M1} value of 1.05 sets the probability of resistance falling below design resistance of less than 1 in 50. Clearly this falls short of the target reliability of 1 in 845.

7.4 DISCUSSION

This exercise in calibration exposes a shortcoming in the boxed value approach to partial safety factors. Boxed values have been chosen mainly for political reasons, since they give member states the freedom to vary the design economy of the codes, in order to keep them in line with existing national standards. This does however create the situation where individual γ_m-factors are applied to a variety of different resistance functions.

Resistance functions vary in their ability to accurately predict resistance. Ideally, different resistance functions require different γ_m-factors in order to achieve the target reliability. This analysis has shown that shear buckling resistance is a relatively difficult structural phenomenon to predict. Increased uncertainty should be reflected by an increase in γ_{M1}, which would also affect all the other resistance functions to which it is applied.

A more rational method for applying the resistance functions contained in the codes would be to determine a γ_R^* factor for each resistance function. The factor would take the form of a numerical constant incorporated into the design expression, with the designer being largely unaware of the origin of the factor. In the case of a complex design procedure (such as the simple post-critical method), a separate factor could be applied to each part of the design procedure. This alternative method is illustrated below for the determination of simple post-buckling shear strength. The γ_{M1} factor is omitted; rather the partial safety factor (shown in bold) is included in the calculation of τ_{ba}.

$$V_{ba.Rd} = d\ t_w \tau_{ba} \qquad (7.23)$$

τ_{ba} is determined as follows:

a) if $\overline{\lambda}_w \leq 0,8$ $\qquad \tau_{ba} = (f_{yw} / \sqrt{3}) / \mathbf{1.15}$ $\qquad$ *(7. 24)*

b) if $0,8 < \overline{\lambda}_w < 1,2$ $\qquad \tau_{ba} = [1 - 0,625\,(\overline{\lambda}_w - 0,8)\,]\,(f_{yw} / \sqrt{3}) / \mathbf{1.30}$ $\qquad$ *(7. 25)*

c) if $\overline{\lambda}_w \geq 1,2$ $\qquad \tau_{ba} = [0,9 / \overline{\lambda}_w]\,(f_{yw} / \sqrt{3}) / \mathbf{1.30}$ $\qquad$ *(7. 26)*

It is appreciated that Nation States will be unwilling to give up the freedom provided by the boxed value system of safety factors. Given this, it is possible to tailor each resistance function with a hidden safety factor as illustrated above, whilst retaining the boxed value system of γ_M factors. Thus, resistance functions requiring relatively large γ_R^* factors to achieve the target reliability could have an additional safety factor applied within the current safety factor in order to adjust the reliability level. Likewise the hidden safety factor could take a value less than unity for design tasks that prove particularly reliable. The concept is illustrated as follows:

$$\gamma_R^* = \gamma_{M1} \cdot \gamma_P \qquad (7.\ 27)$$

γ_P takes the form of a constant contained within the resistance function. This system is applied below to illustrate the concept for the simple post-critical method:

$$V_{ba.Rd} = d\ t_w \tau_{ba} / \gamma_{M1} \qquad (7.\ 28)$$

where simple post-critical shear strength τ_{ba} is determined as follows; the γ_p factor is shown in bold type.

a) if $\overline{\lambda}_w \leq 0,8$ $\qquad \tau_{ba} = (f_{yw} / \sqrt{3}) / \mathbf{1.05}$ $\qquad$ (7. 29)

b) if $0,8 < \overline{\lambda}_w < 1,2$ $\qquad \tau_{ba} = [1 - 0,625\,(\overline{\lambda}_w - 0,8)\,]\,(f_{yw} / \sqrt{3}) / \mathbf{1.25}$ $\qquad$ (7. 30) *c) if* $\bar{\lambda}_w \geq 1,2$ $$\tau_{ba} = [0,9 / \bar{\lambda}_w] (f_{yw} / \sqrt{3}) / \mathbf{1.25} \qquad (7.31)$$

This system is relatively complex, although it does retain individual Nation States' freedom over the codes. A principal objective of the limit state design concept is that reliability should remain relatively uniform, regardless of the material type or design task considered. This chapter has demonstrated that the Eurocodes fall short of this objective. In fact, reliability shows considerable variability within even a single design task. This shortfall can be overcome through the tailoring of each individual resistance function to obtain the desired target reliability. Thus, the overall reliability levels will become more uniform, with certain resistance functions exhibiting a drop in the value of the safety factor applied to them, with a corresponding improvement in design efficiency.

The variation in reliability between different resistance functions is illustrated by Fig. 7. 18 and Fig. 7. 19. This shows an idealised view of the reliability levels achieved between different resistance functions for the system of blanket γ_M factors compared with one of the two alternative approaches proposed herein.

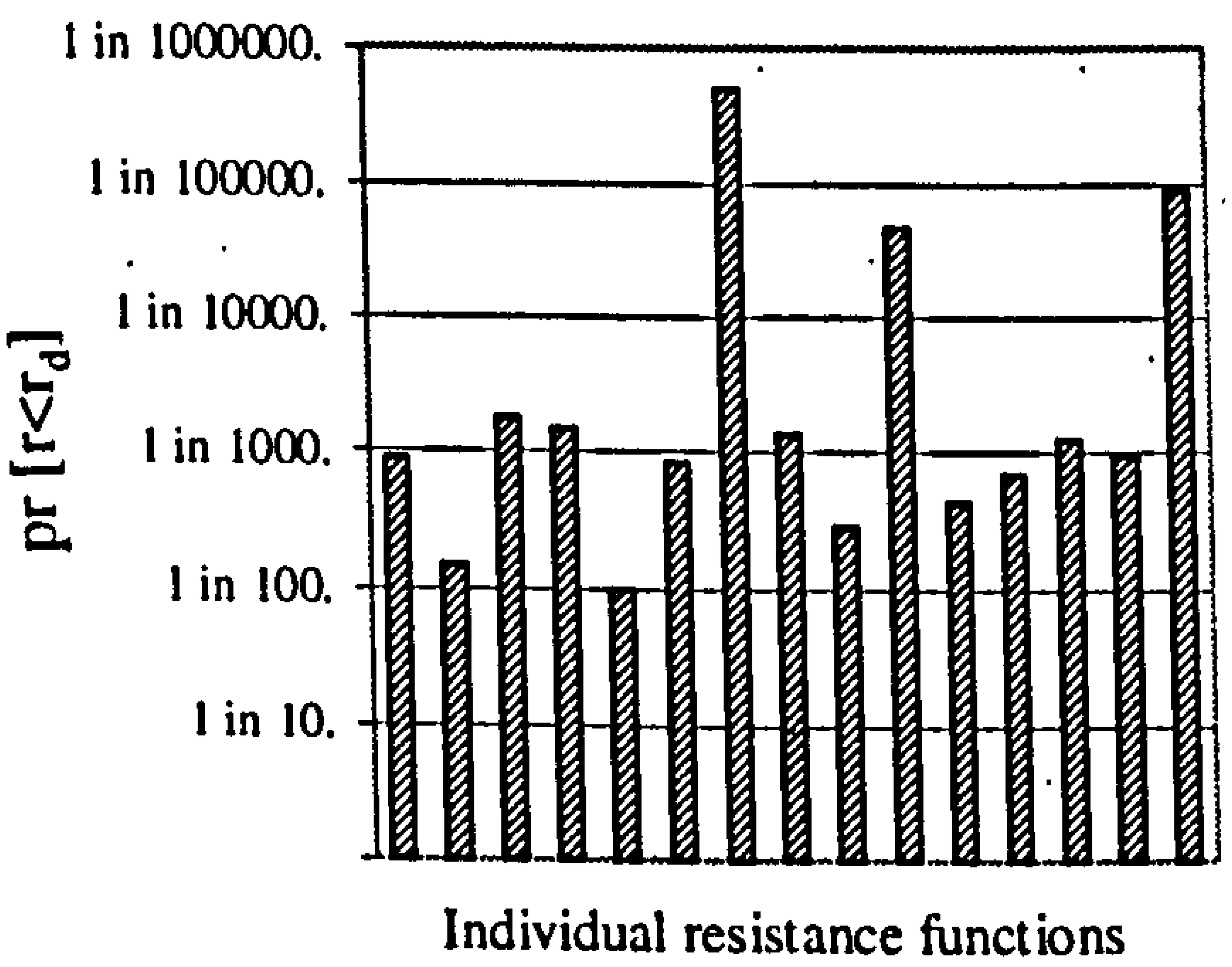


Fig. 7. 18: An idealised view of the variations in reliability, using the existing boxed value system for γ_M-factors

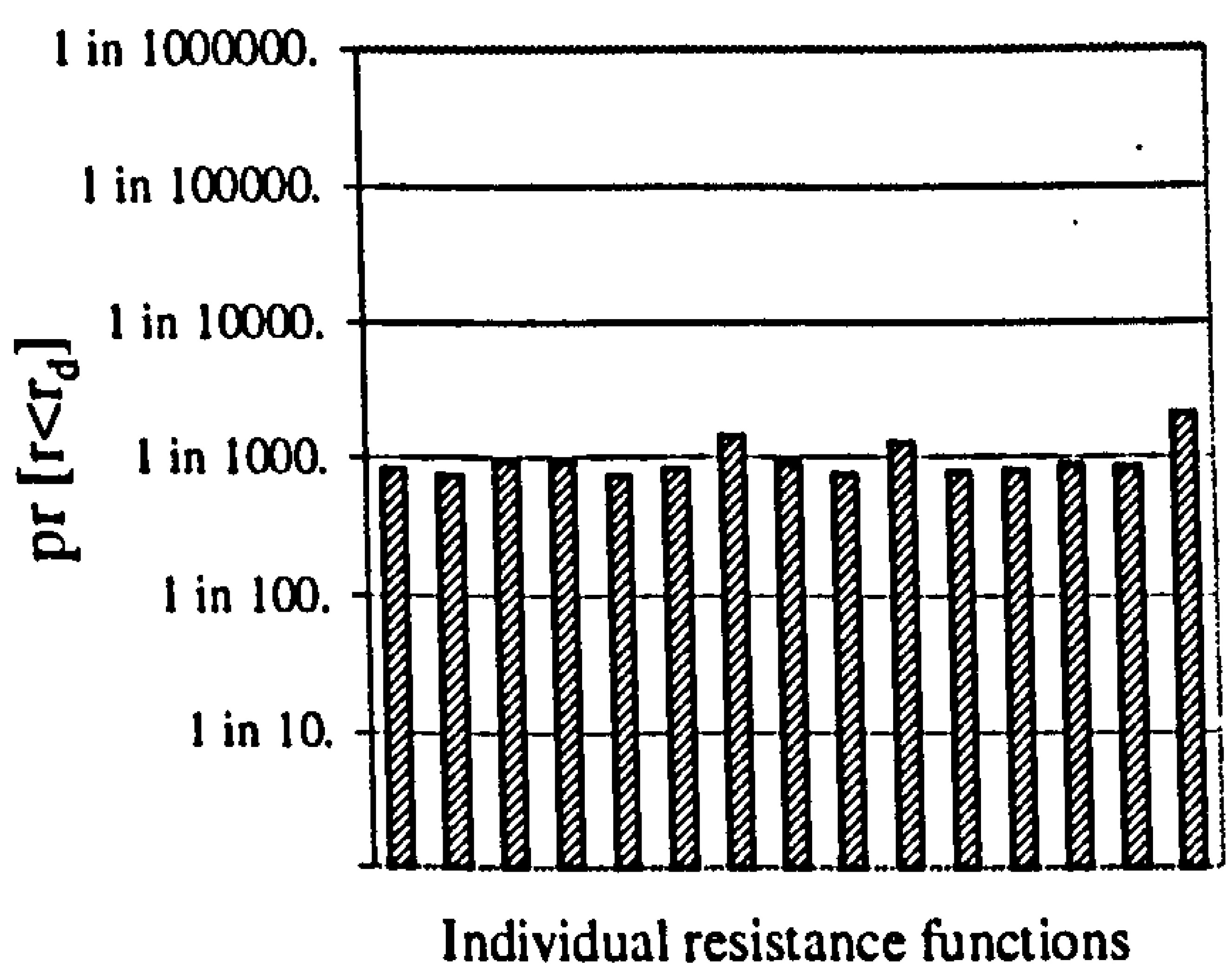


Fig. 7. 19: An idealised view of the variations in reliability, using either of the systems for γ_M-factors proposed herein

7.5 CONCLUSIONS

Plate girders designed using either the simple post-critical method or the tension field method fail to achieve the target reliability level specified by CEN. The reliability levels achieved are closely linked to web slenderness. Girders with stocky webs achieve a substantially greater degree of reliability than girders with slender webs. If the minimum target reliability is to be achieved across the range of plate girder configurations, then γ_{M1} should be increased from 1.05 (specified in the UK NAD to EC3) to 1.35. This very high safety factor would produce excessive levels of reliability for girders with relatively stocky webs, but it is necessary to prevent the minimum reliability level falling below the target reliability. It should be noted that reliability varies with web slenderness because of the increasing influence that web thickness variability has on the variability of the resistance. In the absence of more suitable data, this analysis was carried out using a relatively high value of V_{tw} (0.05). If a lower value of V_{tw} such as 0.01 could be justified the reliability levels would show a considerable improvement. For example the 1.35 value for γ_R^* could be reduced to 1.20.

It is impractical to increase γ_{M1} to a value of 1.35, since this safety factor is applied to a number of different resistance functions contained within the code. Therefore, it would damage the competitiveness of many aspects of steel construction. Thus a flaw is exposed in the CEN approach to limit state design. Boxed values of partial safety factors result in considerable variations in reliability between the various resistance functions. The objective of limit state design - that reliability levels should become uniform, regardless of the design task or material considered - is not achieved.

Statistical theory provides the tools for decision making in the face of uncertainty. Properly applied, it is possible to quantify the uncertainty associated with a particular resistance function and to determine a safety factor that sets the reliability to a certain specified level. Since the degree of uncertainty associated with different resistance functions and their related structural phenomena varies, the numerical values of the safety factors must also vary. Thus, a means of tailoring boxed values of safety factors to the requirements of individual resistance functions needs to be found if the objective of uniform reliability levels is to be achieved.

The reasoning behind the boxed value system of safety factors is largely political. They provide CEN member states with the freedom to adjust the design economies achieved by the codes to the levels achieved by existing national standards. This chapter proposes a system whereby resistance functions contain an additional numerical constant, that adjusts the functions to reach the specified target reliability. Resistance functions would contain two partial safety factors, the already familiar γ_M factor, whose value is set in the NAD of the code, and one that is simply a numerical constant tailored to account for the uncertainty associated with the particular resistance function - the basis of which would effectively be hidden from designers. Member states would retain the freedom facilitated by the boxed value system of safety factors, whilst reliability levels would become more consistent. Clearly this approach is complex, it may however be necessary in order to achieve important safety as well as political objectives.

Chapter Eight

THE RELIABILITY OF RESTRAINED BEAMS

8.1 INTRODUCTION

Laterally restrained beams are one of the most commonly occurring structural elements. During design it is normally assumed that their ultimate bending strength may be taken as their plastic moment capacity ($M_{pl.Rd}$), given as the product of their plastic section modulus and the yield strength of the material. In contrast to the case for many other structural elements such as laterally unrestrained beams, beam-columns, connections, plate girders etc., very little test data is available against which to check this basic assumption.

Bending tests reported in (Hasan and Hancock, 1988) show that the $M_{pl.Rd}$ resistance function substantially underestimates the strength of cold-formed rectangular hollow sections. In the present study a series of bending tests has been carried out to determine whether a similar degree of conservatism exists between the actual and predicted bending strengths of hot-rolled open sections.

After reviewing the Eurocode 3 background documentation (Sedlacek *et al*, 1989a) it would appear that the γ_R^*-factor relating to the plastic moment of resistance formula has been calibrated without the use of test results. Instead, material strengths and geometric properties were measured for a large sample of sections - the results of which were used to calculate resistance using the $M_{pl.Rd}$ formula. This method of calibration cannot account for the degree to which the formula underestimates resistance. It is likely therefore to have produced a high value for γ_R^*.

Providing full lateral restraint to the compression flange of test beams is somewhat difficult. In practice, compression flanges are frequently restrained along the length of the section by a supported slab. Previous bending tests (Bureau, 1993a) have omitted compression flange restraint with the result that premature, out-of-plane buckling occurs which prevents the development of full bending strength. The bending tests performed in this study have been devised to overcome this problem.

8.2 EXPERIMENTAL TESTING OF BEAMS

According to the design rules of Eurocode 3: Part 1.1, the full plastic moment capacity of members subjected to bending forces can be utilised where the non-dimensional slenderness ($\bar{\lambda}_{LT}$) is less than or equal to 0.4. Since load capacity reduces with increasing slenderness, these tests have been carried out close to the limiting slenderness, as this corresponds to the worst possible case scenario. The section designation and steel grade of the test specimens are listed in Table 8. 1.

Test numbers	Sample size	Grade	Section designation
V1 to V10	10	FE430A	203x102x23UB
W1 to W10	10	FE430A	152x152x30UC

Table 8. 1

8.2.1 THE TESTING APPARATUS

The apparatus for the testing programme was arranged in such a way that a plastic failure mechanism could be developed in the specimens without incurring significant frictional forces. This problem is made more acute because of the provision of lateral restraint to the compression flanges of the specimens.

There are a number of reasons why it was considered desirable to reduce frictional effects where possible. The first is that frictional effects tend to increase the degree of scatter between the predicted and experimental resistances - a quantity measured by the σ_b-factor (Bijlarrd *et al.*, 1988). Since the numerical value for γ_R^* determined during calibration is heavily influenced by σ_b, its reduction will result in an optimum value of γ_R^*.

Friction between the test specimen and the loading and reaction points was considered particularly undesirable since it would provide a restraint to the test section and result in an overestimate of the strength of the specimens. Quantifying the contribution of this frictional effect to the strength of the specimens would be complex, leading to additional assumptions when interpreting the results. In addition, it was considered desirable to limit the friction between the lateral restraints and the compression flanges. A significant force developing between the compression flange and the lateral restraint would lead to a destabilising torsional force. It was believed that this would reduce the load capacity of the sections.

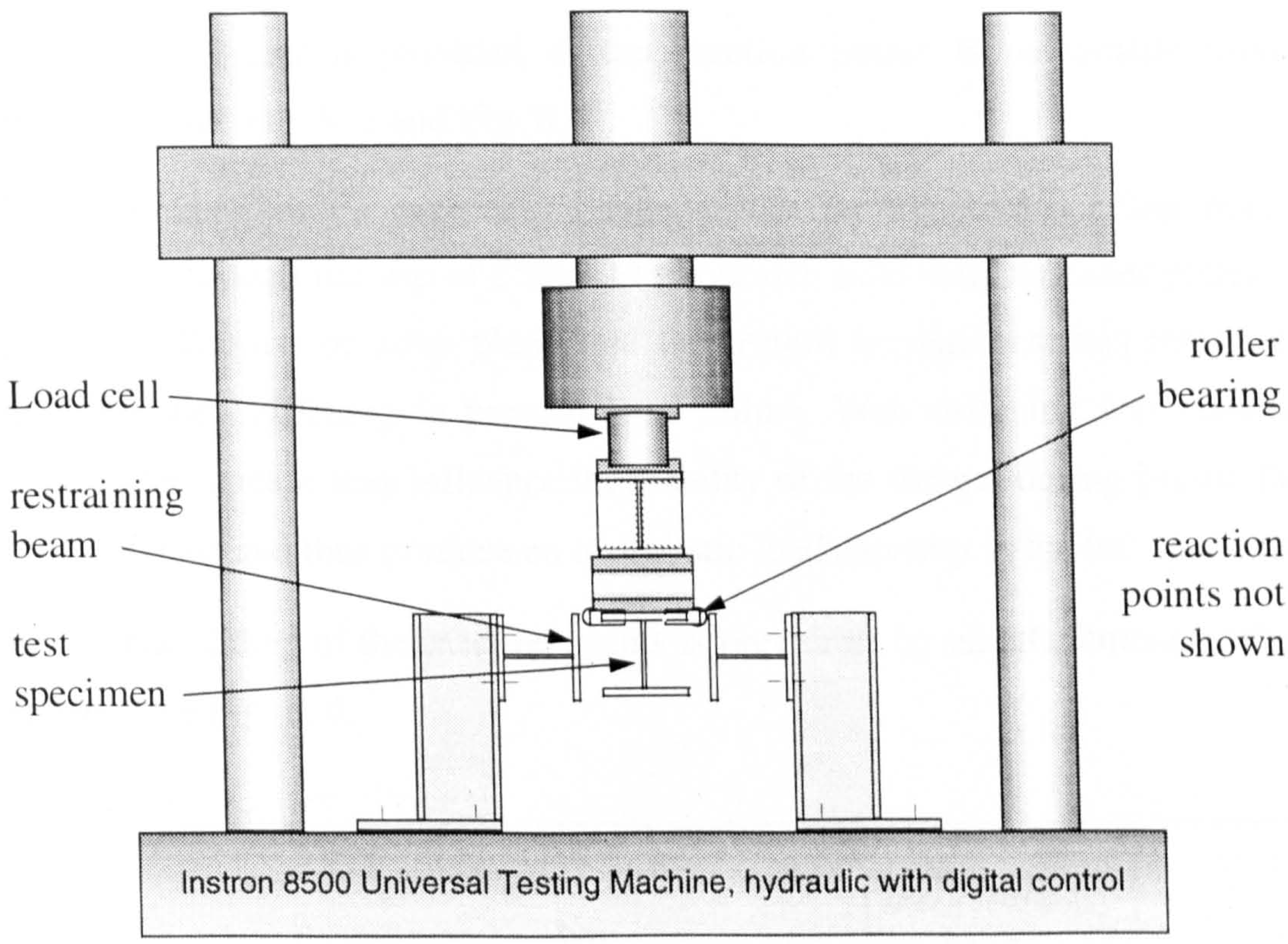


Fig. 8. 1: Section through test apparatus

The tests were of the 4-point bending type. Although this type of test involves extra complexity it was considered preferable to the 3-point bending test. The 4-point bending test has the advantage that the plastic hinge is isolated from the loading points (see Fig. 8. 2). In addition, 3-point bending tests restrain the compression flange against buckling at the location of the plastic hinge which would partially negate a source of instability that could lead to an early failure of the member.

The layout of the testing apparatus is sketched in Fig. 8. 1 to Fig. 8. 4. The main details of the arrangement may be summarised as follows:

- Roller bearings butt up to restraining beams to provide low friction lateral restraints - see Fig. 8. 1.

- Frictional forces developed at the loading and reaction points will have a restraining effect on the test specimens. These frictional forces have been reduced to a negligible level through the use of rollers at the reaction and loading points - see Fig. 8. 2. One of the rollers was fixed to the base of the Instron Universal Testing Machine for safety reasons. This fixity should not affect the test results.

- End rotation is measured by a spring loaded linear potentiometer connected to the end torsional restraint - see Fig. 8. 3.
- Torsional restraint is provided at the reaction points by adjustable torsional restraints - see Fig. 8. 3 and Fig. 8. 4.
- Local buckling of the web and flange at the loading and reaction points is prevented through the use of 25mm bright drawn mild steel spreader plates - see Fig. 8. 4. The use of these plates and the overall arrangement has removed the need for web stiffening to prevent local failure. Web stiffening was considered undesirable since it may influence the stability of the flanges during plastic failure of the section and thus produce an unrealistic load capacity in the test specimen.
- Horizontal sliding of the reaction points is prevented by an attachment to the end rollers - see Fig. 8. 4.

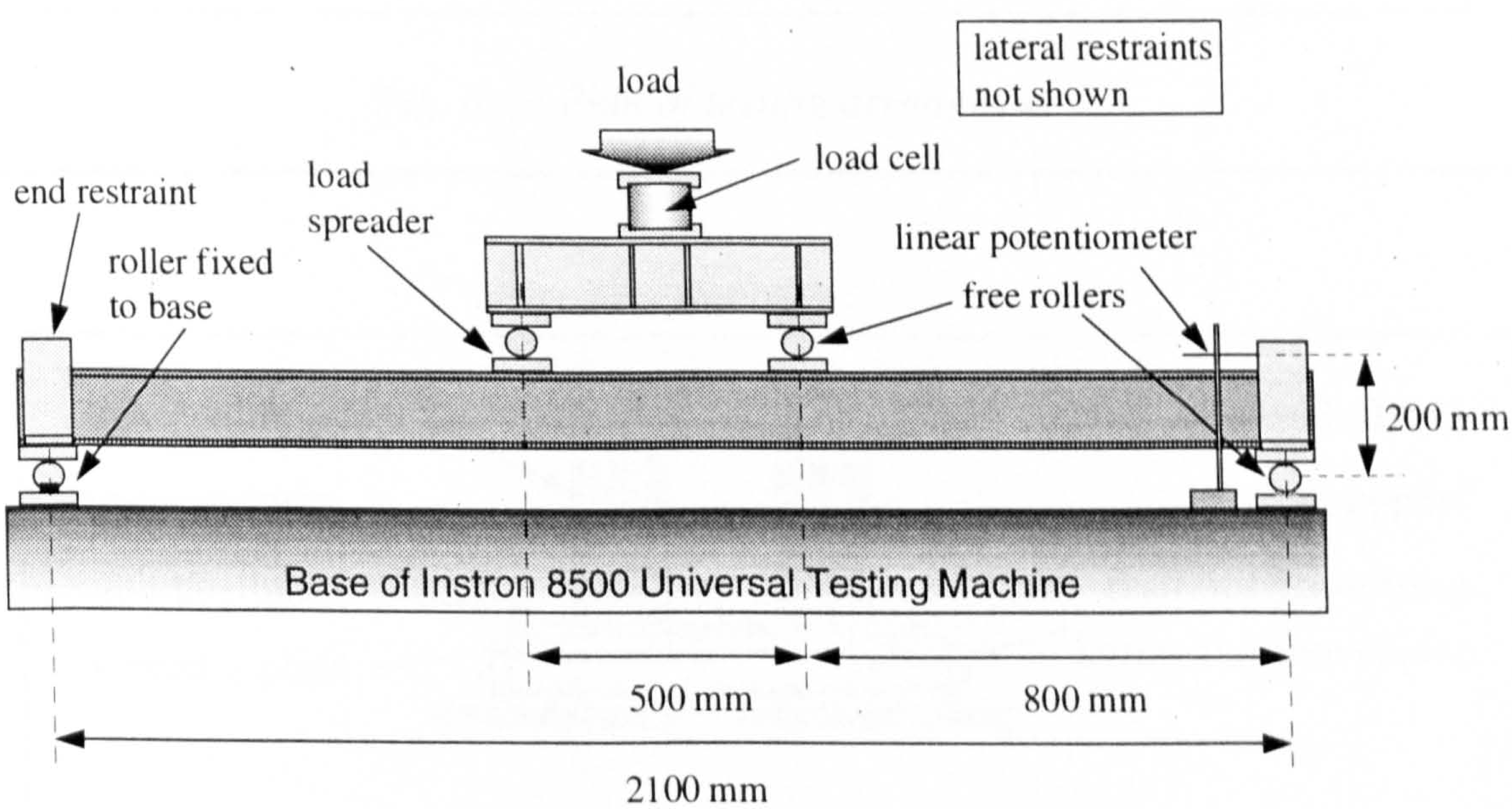


Fig. 8. 2: Elevation of test arrangement

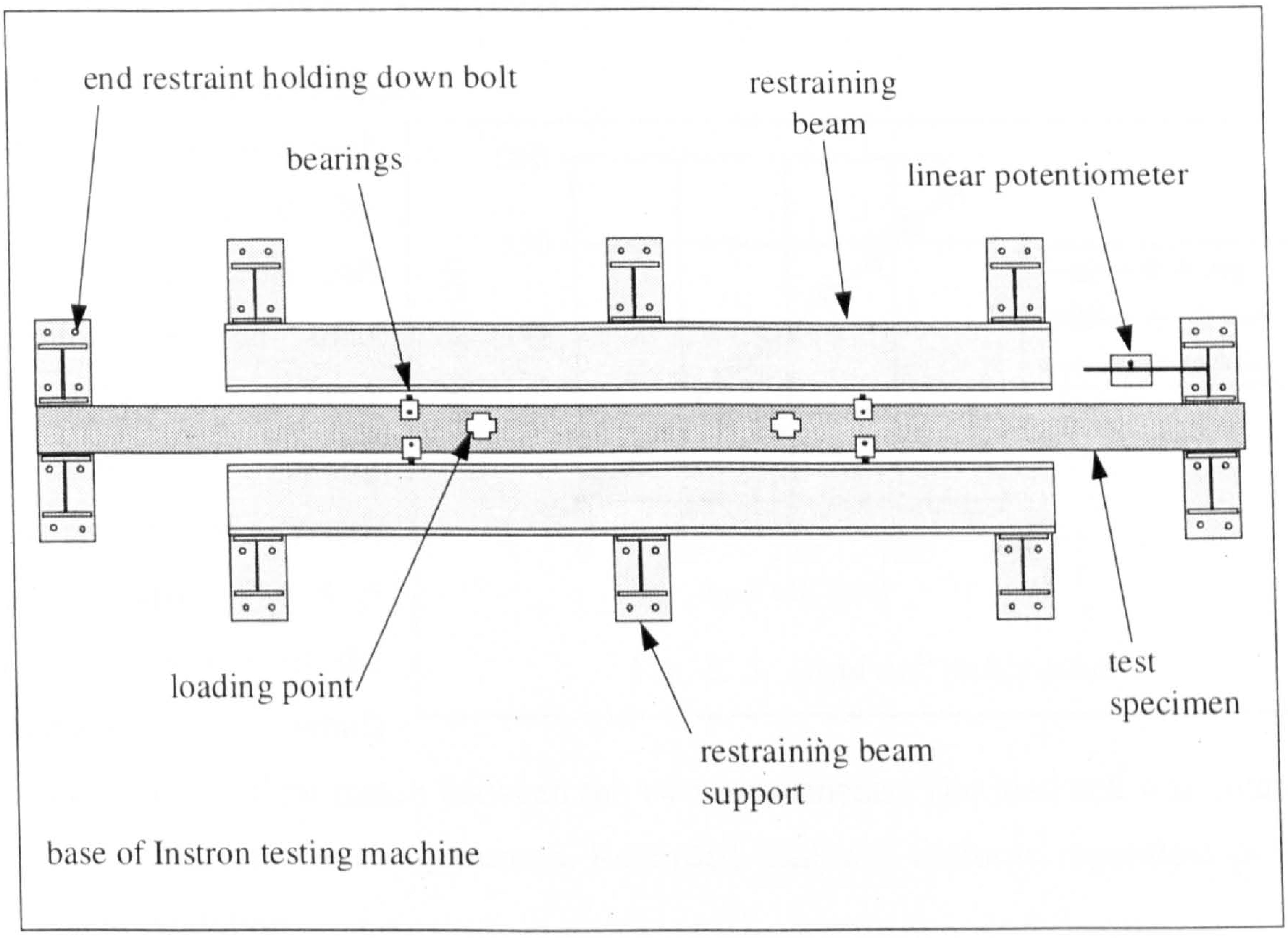


Fig. 8. 3: Plan of testing arrangement

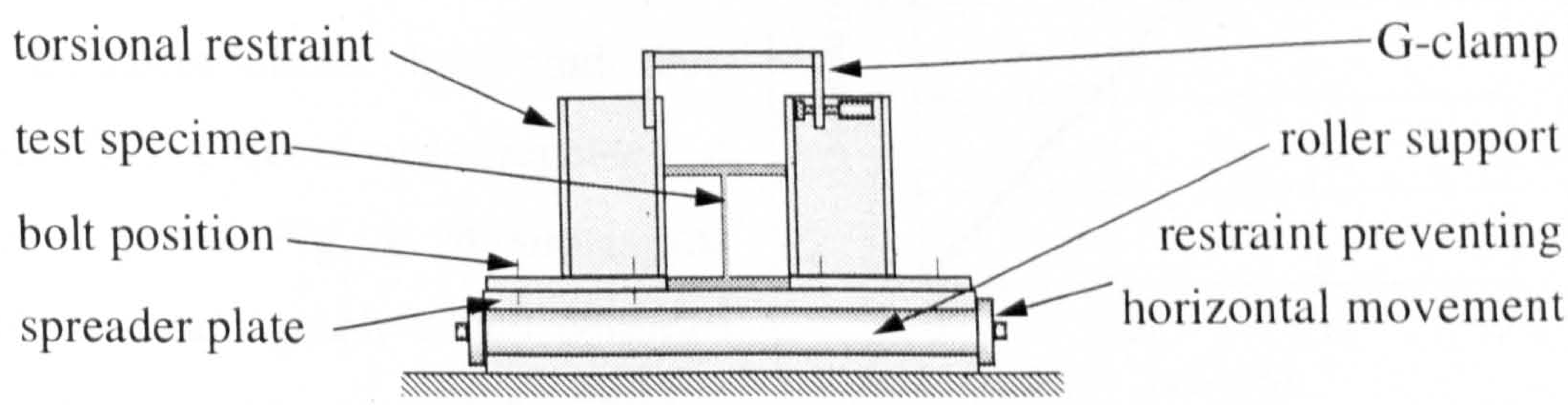


Fig. 8. 4: Detail of end torsional restraint

8.2.2 VALIDATION OF TESTING APPARATUS

8.2.2.1 LOAD CELL CHECK

The integrity of the load cell was checked by comparing the load cell output with the load applied by a Zwick 1484 Universal Testing Machine (screw driven, digital control). Fig. 8. 5 shows the results of the comparison which produced an excellent match between the two instruments. The load cell was rotated through 90 degrees for completeness. Recorded load was uniform, regardless of the load cell orientation.

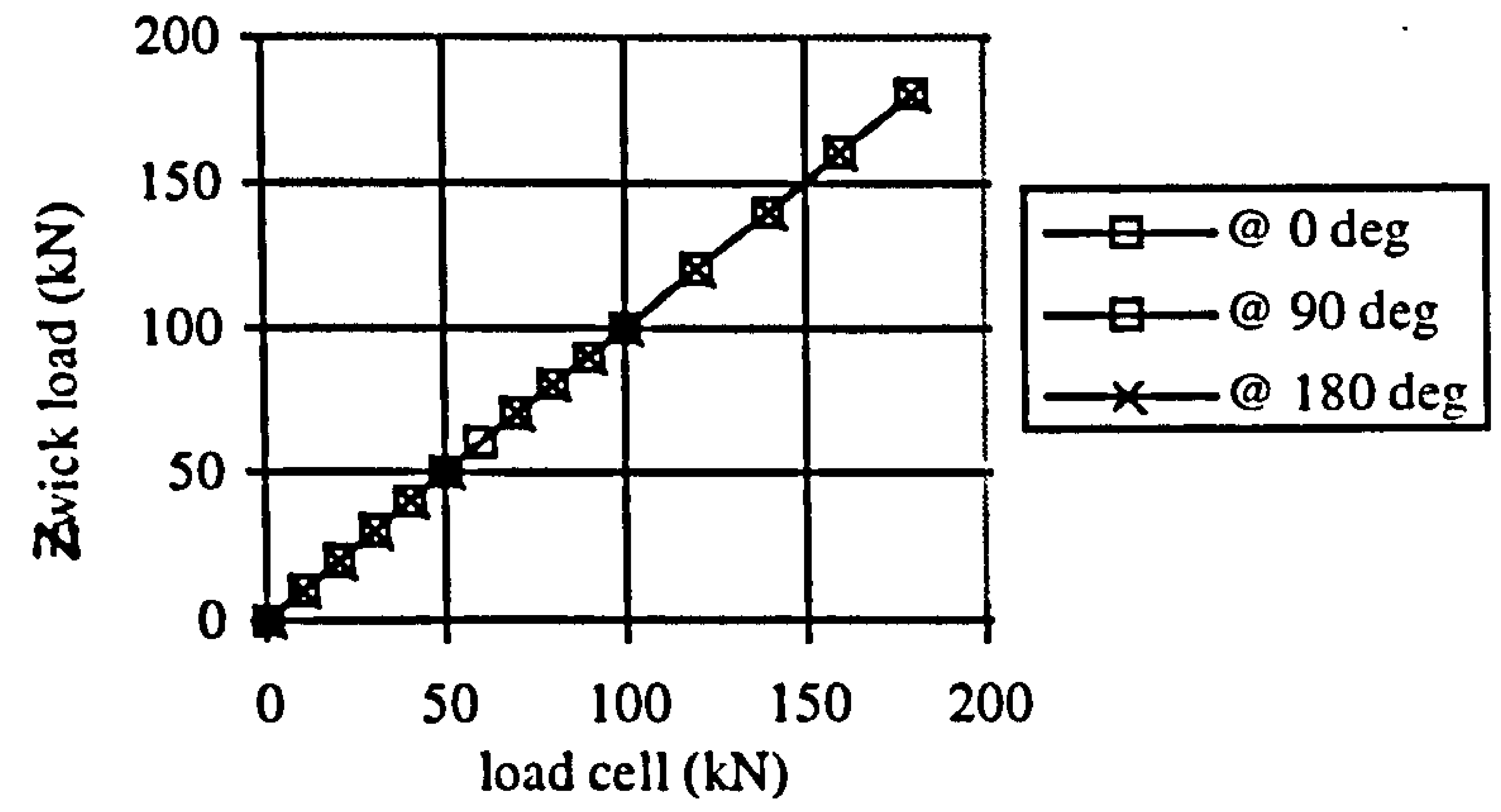


Fig. 8. 5: Load cell calibration

8.2.2.2 SIMPLE ELASTIC TESTS

Prior to destructive testing, elastic tests were carried out to identify any adverse frictional effects. Load was applied close to the members elastic limit and then released. This process was repeated a number of times. Fig. 8. 6 shows the moment vs. strain graph which confirms that the testing arrangement has largely overcome the adverse frictional effects.

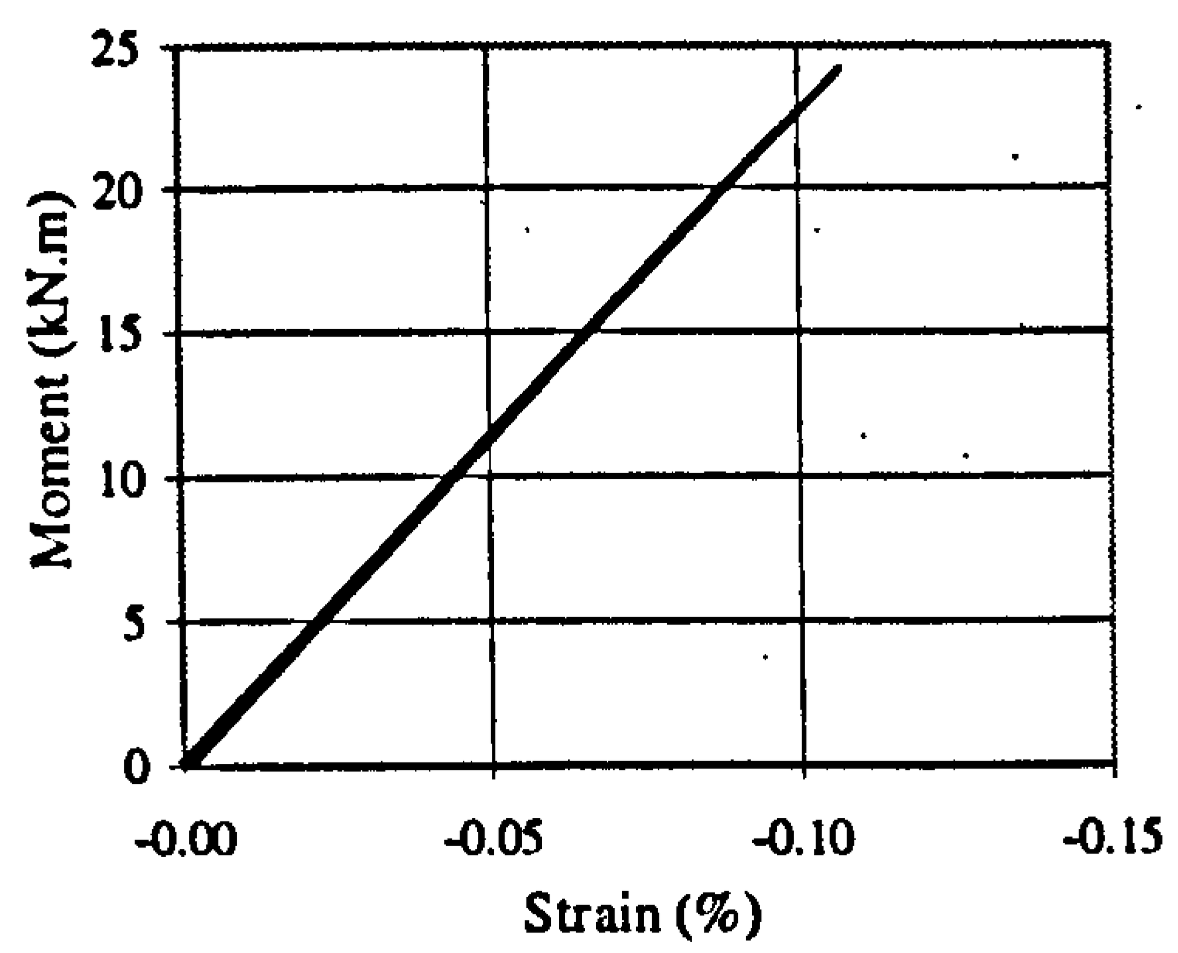


Fig. 8. 6: Elastic test

8.2.3 SLENDERNESS CALCULATIONS

As stated previously, the bending tests were carried out with the non-dimensional slenderness set close to but less than the limiting slenderness of 0.4. Thus, intermediate lateral restraints were appropriately spaced during testing so that the slenderness would be within this range. Both the effective length and $\overline{\lambda}_{LT}$ (calculated from the measured material and geometric properties of the test specimens) are listed in Table 8. 4. Briefly, these values of slenderness were calculated using the following assumptions about the behaviour of specimens during the tests:

- Any restraint occurring between the load and test specimen has been ignored. Loading is transferred to the test specimen through a free roller.
- The specimens are allowed to rotate freely on plan about the reaction points, i.e, no end fixity.
- Loading is transferred to the test specimen through the shear centre, $Z_g=0$, $C_2=0$.
- The restraining effect due to the unequal length between the restraining points and between these restraints and the reaction points is ignored (see Fig. 8. 7 and Fig. 8. 8); thus $k=1.0$, $k_w=1.0$.
- The bending moments between the restraining points have been assumed to be uniform, i.e, $C_1=1.0$.

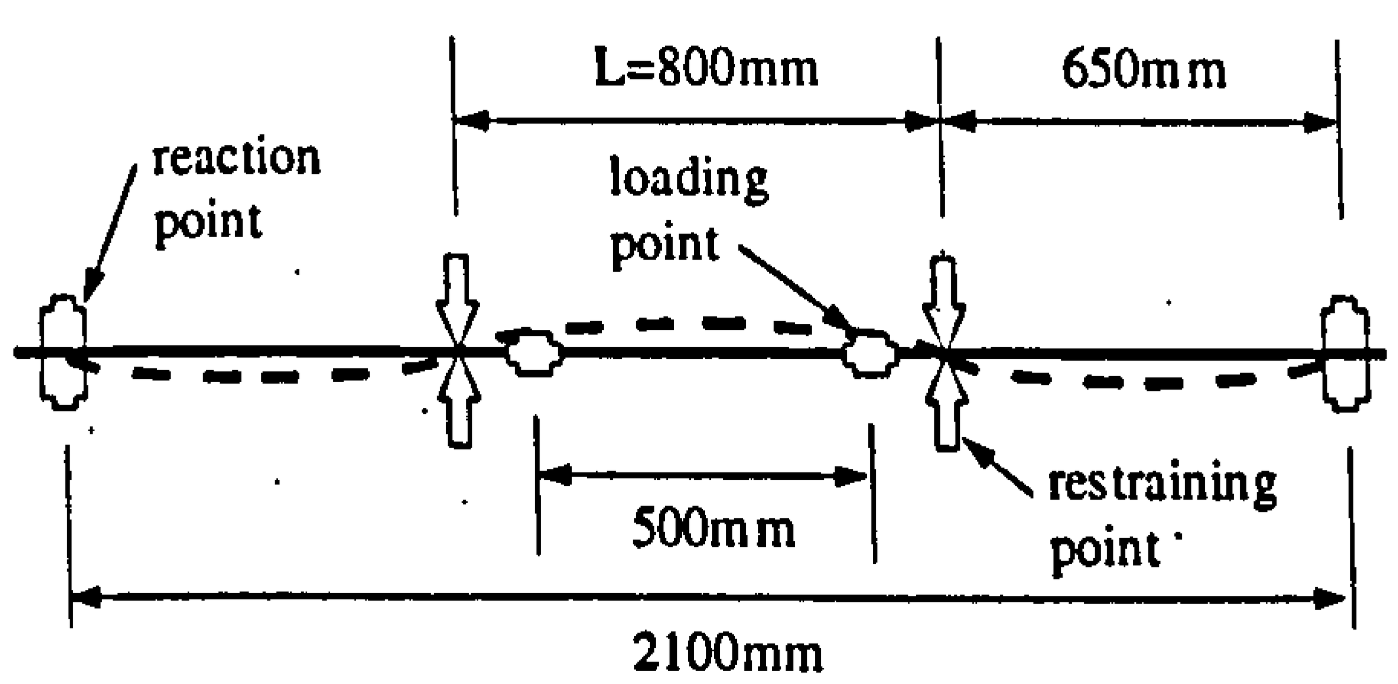


Fig. 8. 7: Loading arrangement test specimens V1 to V10

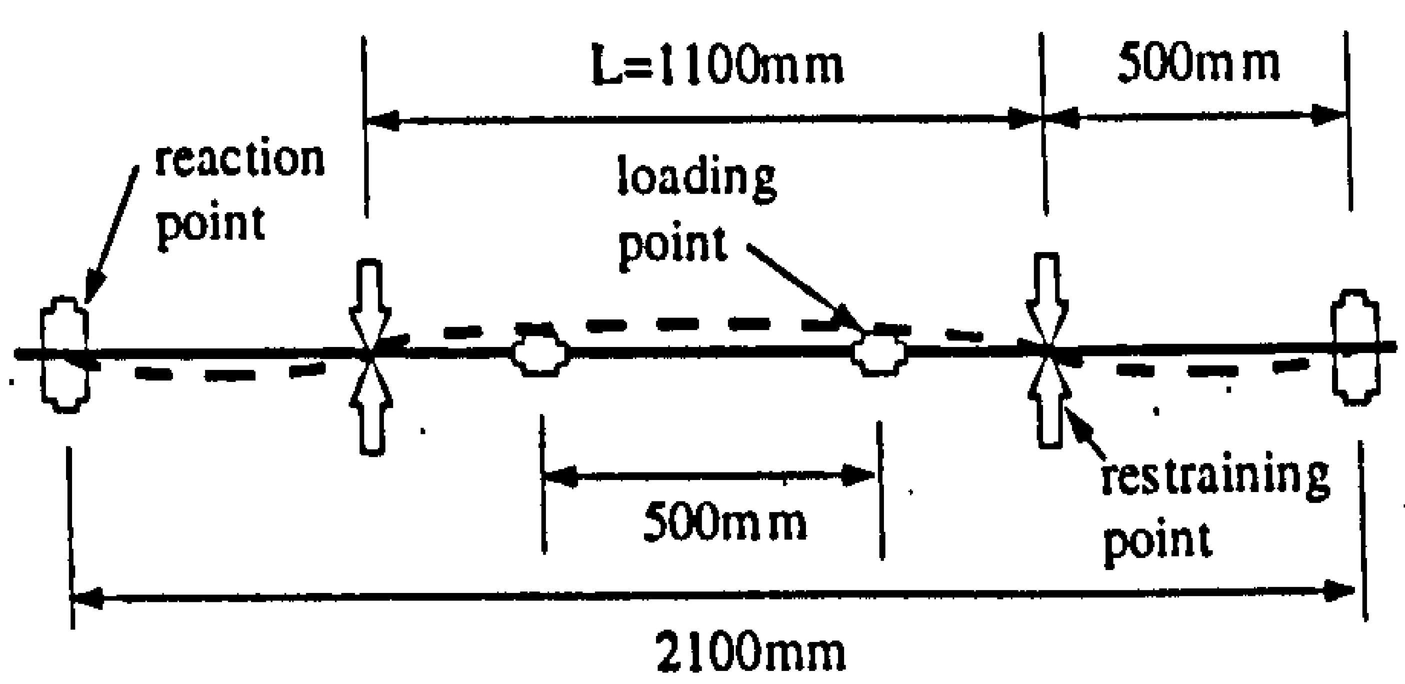


Fig. 8. 8: Loading arrangement test specimens W1 to W10

8.2.4 GEOMETRIC PROPERTIES

The geometric properties of each test specimen were recorded for the purposes of determining the theoretical moment capacity. These measurements are listed in Table 8. 2, together with the major axis plastic section modulus computed from these measurements. The notation used in Table 8. 2 is sketched in Fig. 8. 9. The nominal radius was assumed during calculations.

Test No.	Section type	B1 mm	B2 mm	D1 mm	D2 mm	t1 mm	t2 mm	T1 mm	T2 mm	T3 mm	T4 mm	$W_{pl.y}$ cm^3
V1	203x102x23UB	102.5	102.0	203.0	202.0	5.79	5.69	8.74	8.74	8.79	9.10	228.6
V2	203x102x23UB	102.5	103.0	203.0	202.0	5.76	5.65	8.78	8.77	9.06	8.80	229.3
V3	203x102x23UB	102.5	102.5	202.0	203.0	5.80	5.62	8.72	9.05	8.93	8.85	229.6
V4	203x102x23UB	102.0	102.0	203.0	203.0	5.79	5.64	8.80	8.73	9.04	8.75	228.5
V5	203x102x23UB	102.0	102.0	202.0	203.0	5.80	5.65	8.77	8.76	9.05	8.80	228.1
V6	203x102x23UB	102.5	102.0	202.0	202.0	5.81	5.65	8.76	8.60	8.59	9.00	225.9
V7	203x102x23UB	101.5	102.0	202.0	202.5	5.55	5.73	9.02	8.74	8.89	8.79	226.8
V8	203x102x23UB	102.0	102.0	202.0	203.0	5.50	5.80	8.98	8.83	8.96	8.69	227.8
V9	203x102x23UB	102.0	102.5	202.0	202.0	5.65	5.75	9.05	8.65	8.67	8.60	225.8
V10	203x102x23UB	102.5	102.5	202.5	202.0	5.76	5.61	8.74	8.64	8.70	8.06	222.7
W1	152x152x30UC	152.0	152.0	157.0	156.5	5.98	5.63	8.93	9.33	9.05	8.91	234.5
W2	152x152x30UC	151.5	151.5	157.0	157.0	6.16	6.02	8.92	9.11	9.23	8.89	235.3
W3	152x152x30UC	151.5	151.0	157.5	157.0	6.09	6.01	9.01	9.17	9.17	8.96	236.1
W4	152x152x30UC	152.0	151.0	157.0	156.5	6.06	6.00	8.89	9.14	9.16	8.89	234.2
W5	152x152x30UC	151.0	151.0	157.0	157.0	5.98	6.07	8.96	9.30	9.06	8.88	234.6
W6	152x152x30UC	151.0	151.5	155.5	157.5	6.00	6.07	8.96	9.34	9.05	8.86	234.1
W7	152x152x30UC	151.5	151.5	157.5	157.0	6.00	6.10	8.95	9.38	9.09	8.95	236.7
W8	152x152x30UC	152.0	151.0	157.5	156.0	6.24	6.07	8.96	9.10	9.13	8.94	235.1
W9	152x152x30UC	151.0	152.0	157.5	156.0	6.00	6.06	8.98	9.29	9.07	8.87	234.9
W10	152x152x30UC	151.0	152.0	156.0	157.0	6.13	6.22	9.04	9.31	9.10	8.95	236.1

Table 8. 2: Geometric properties of test specimens

8.2.5 MATERIAL PROPERTIES

Mill tests were carried out to determine the material properties of the test specimens. It was considered important to obtain mill tests for reasons of compatibility, since the calibration of the partial safety factors is based on a measure of yield strength variability also determined from mill tests.

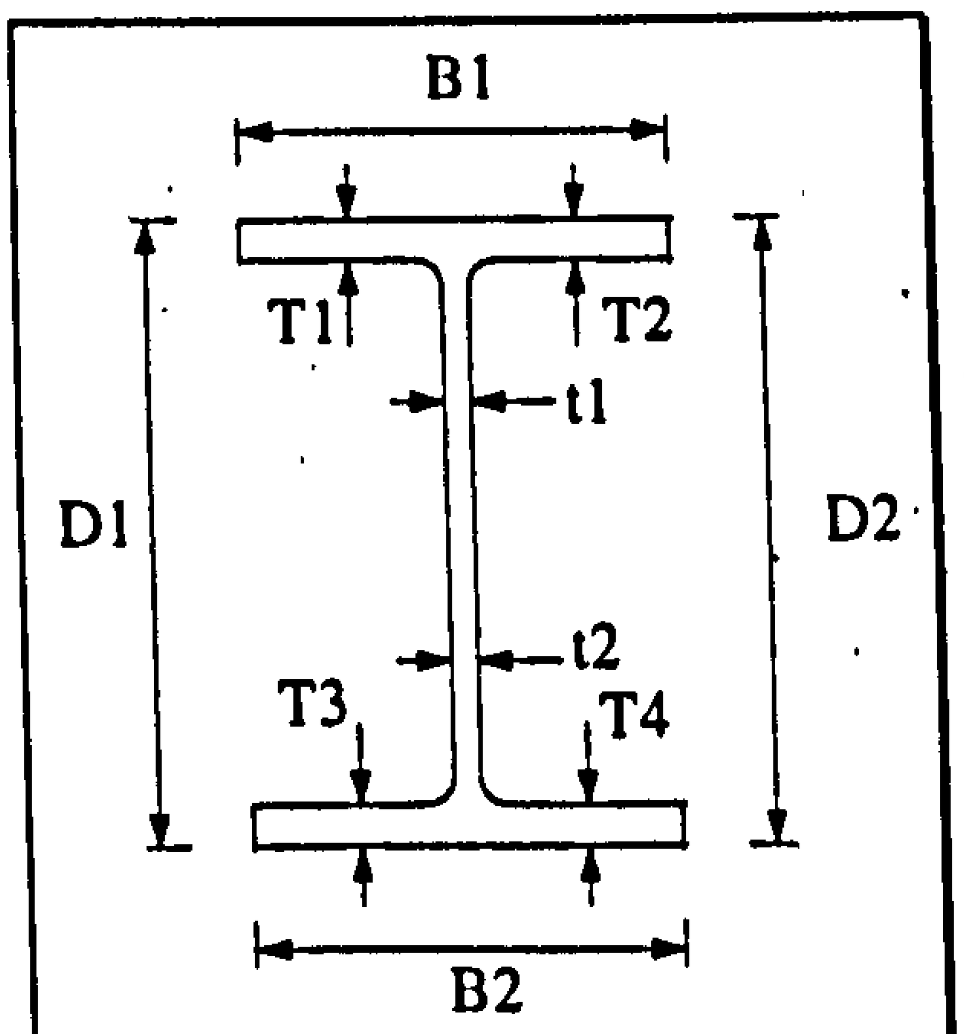


Fig. 8. 9: Notation used in Table 8. 2

The tests were carried out using digital logging equipment, and in accordance with BS EN10002-1:1990. The testing apparatus was calibrated at the beginning of the day and the samples were shot blasted prior to testing. A full list of the material properties is listed in Table 8. 3. The yield stress corresponds to 0.5% total strain and is not a proof stress.

Test No.	Grade	flange f_y N/mm^2	flange f_u N/mm^2	flange % elongation	web f_y N/mm^2	web f_u N/mm^2	web % elongation	flange E N/mm^2
V1	FE430A	310	480	24	333	492	26	201900
V2	FE430A	324	489	24	339	498	25	197030
V3	FE430A	323	489	23	335	504	25	206890
V4	FE430A	329	499	19	342	496	26	198940
V5	FE430A	315	471	22	344	484	26	205110
V6	FE430A	322	484	21	363	521	23	222490
V7	FE430A	315	476	21	334	479	26	233810
V8	FE430A	315	478	21	345	516	25	213750
V9	FE430A	317	472	23	344	483	25	210600
V10	FE430A	317	482	22	355	511	24	186180
W1	FE430A	286	484	32	334	503	30	200670
W2	FE430A	288	477	36	341	509	28	194780
W3	FE430A	289	476	38	379	548	24	not available
W4	FE430A	293	482	34	330	504	27	not available
W5	FE430A	290	477	35	360	539	25	not available
W6	FE430A	287	481	35	350	545	32	not available
W7	FE430A	294	482	33	344	513	27	not available
W8	FE430A	299	478	34	352	519	25	not available
W9	FE430A	291	479	35	357	515	27	not available
W10	FE430A	299	481	38	365	531	25	not available

Table 8. 3: Material properties of test specimens

8.2.6 EXPERIMENTAL LOAD CAPACITIES OF THE TEST SPECIMENS

Each of the specimens was tested to failure with the load, mid-span deflection and end rotation recorded digitally throughout the test. The moment vs. rotation curves of the specimens are shown in Fig. 8. 10 and Fig. 8. 11. The experimental moment capacities of the test specimens are listed in Table 8. 4 along with the moment capacities achieved at 2, 4, 6 and 8 degrees of end rotation.

All specimens demonstrated the ability to develop a controlled and stable failure mechanism. Prior to the attainment of the maximum load, the compression flanges were observed to buckle. Despite often considerable local distortions the sections remained able to withstand additional loading. Failure of the specimens was characterised by the following sequence of events:

- Prior to the formation of the plastic hinge the specimens remained largely unchanged.
- The development of the plastic hinge was characterised by rapid deformation.
- During the early stage of plastic deformation the compression and tension flanges remained largely parallel.

- With continued rotation the specimens tended to buckle between the points of lateral restraint. Whilst the compression flanges buckled considerably, the sections remained able to sustain additional loading.
- The tension flanges remained largely transversely horizontal throughout the tests.

Ref.	Section type	Date tested	L mm	L_e mm	$\bar{\lambda}_{LT}$	$M_{pl.Rd}$ kN.m	M_{max} kN.m	$M/M_{pl.Rd}$ @ 2° ϕ	$M/M_{pl.Rd}$ @ 4° ϕ	$M/M_{pl.Rd}$ @ 6° ϕ	$M/M_{pl.Rd}$ @ 8° ϕ
V1	203x102x23	29/9/95	2100	800	0.38	70.9	79.9	1.00	1.13	0.91	0.91
V2	203x102x23	10/2/95	2100	800	0.39	74.3	80.9	0.97	1.07	0.90	0.85
V3	203x102x23	10/3/95	2100	800	0.38	74.2	87.5	1.01	1.11	1.18	1.02
V4	203x102x23	10/3/95	2100	800	0.39	75.2	79.4	0.98	1.06	0.86	0.86
V5	203x102x23	3/10/95	2100	800	0.38	71.8	79.3	1.01	1.10	0.86	0.86
V6	203x102x23	3/10/95	2100	800	0.37	72.7	79.2	1.01	1.09	0.88	0.88
V7	203x102x23	4/10/95	2100	800	0.35	71.5	89.1	1.01	1.10	1.19	1.24
V8	203x102x23	4/10/95	2100	800	0.37	71.8	82.3	1.01	1.13	1.10	0.96
V9	203x102x23	5/10/95	2100	800	0.37	71.6	80.2	0.98	1.09	1.11	0.92
V10	203x102x23	5/10/95	2100	800	0.40	70.6	82.9	1.05	1.14	1.06	1.06
W1	152x152x30UC	31/10/95	2100	1100	0.31	67.1	78.0	1.00	1.09	1.16	1.14
W2	152x152x30UC	16/11/95	2100	1100	0.32	67.8	80.0	0.99	1.08	1.16	1.16
W3	152x152x30UC	17/11/95	2100	1100	0.31	68.2	81.9	0.98	1.06	1.16	1.19
W4	152x152x30UC	17/11/95	2100	1100	0.31	68.6	81.3	0.99	1.06	1.15	1.17
W5	152x152x30UC	20/11/95	2100	1100	0.31	68.0	81.5	0.99	1.08	1.16	1.19
W6	152x152x30UC	20/11/95	2100	1100	0.31	67.2	81.2	1.01	1.09	1.18	1.18
W7	152x152x30UC	21/11/95	2100	1100	0.31	69.6	81.2	0.96	1.05	1.13	1.17
W8	152x152x30UC	21/11/95	2100	1100	0.31	70.3	82.3	0.96	1.04	1.13	1.17
W9	152x152x30UC	21/11/95	2100	1100	0.31	68.4	80.7	0.98	1.08	1.14	1.18
W10	152x152x30UC	23/11/95	2100	1100	0.31	70.6	80.0	0.96	1.04	1.12	1.11

Table 8. 4: Load capacities of test specimens

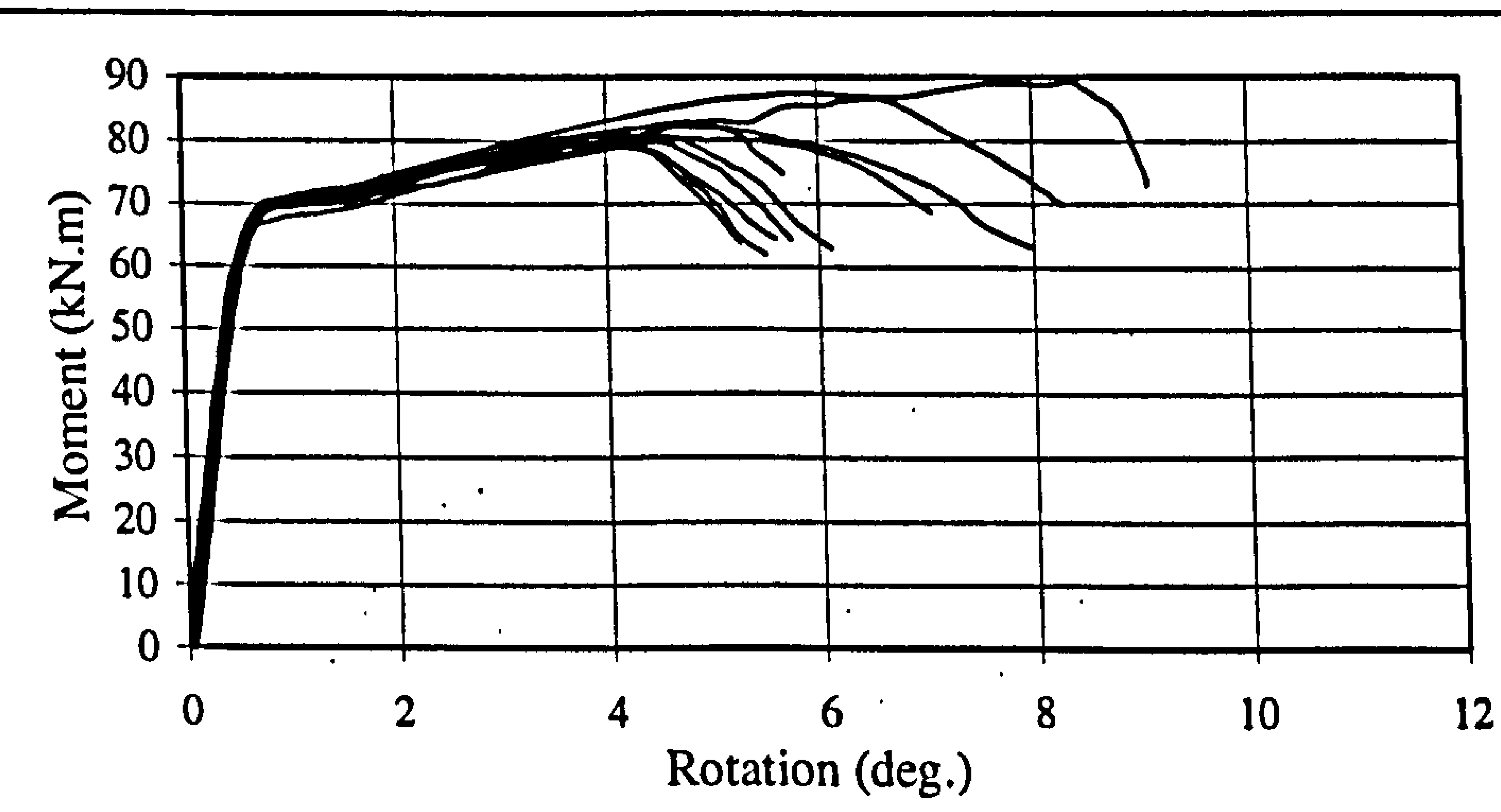


Fig. 8. 10: 203x102x23UB, test specimens (V1 to V10)

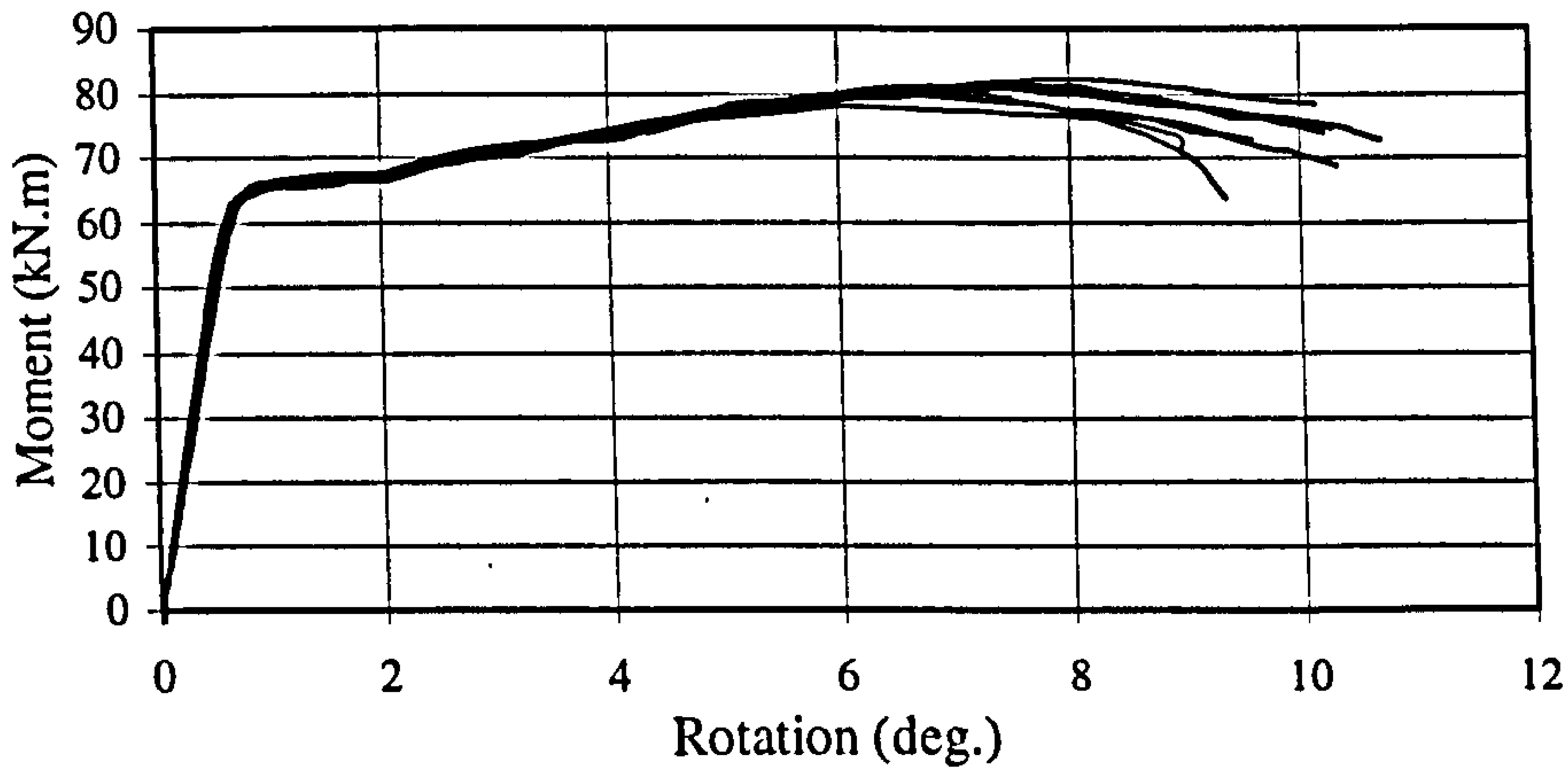


Fig. 8. 11: 152x152x30UC, test specimens (W1 to W10)

8.3 THE COMPARISON BETWEEN EXPERIMENTAL AND PREDICTED RESISTANCES

The theoretical moment capacity of each of the test specimens has been calculated using the mill tests and measured geometric properties. The resulting moment capacity ($M_{pl.Rd}$) has been compared with the experimental moment capacity (M), so that the accuracy of the resistance function can be determined.

Fig. 8. 12 shows the graph of the normalised experimental moment capacity vs. end rotation. This graph illustrates the amount of rotation necessary to achieve and exceed the theoretical moment capacity of the specimens. This is important, since a limit needs to be a set on the amount of end rotation necessary to achieve the moment capacity used during calibration of the resistance function. A failure mechanism that required an unreasonable degree of end rotation in order to achieve the desired resistance would be an impractical requirement during a limit state. Unfortunately, the Eurocodes provide no guidance on the amount of end rotation acceptable during a failure mechanism. In the absence of other guidance, a cut-off point of 6 degrees end rotation has been selected, above which the moment capacity achieved by test specimens cannot be used for calibration purposes. The figure of 6 degrees has been selected, because it represents what may be considered a reasonable degree of rotation for an ultimate limit state. However, this is an arbitrary figure based on the author's judgement. Further research is necessary in order to determine what constitutes an acceptable amount of end rotation during an ultimate limit state.

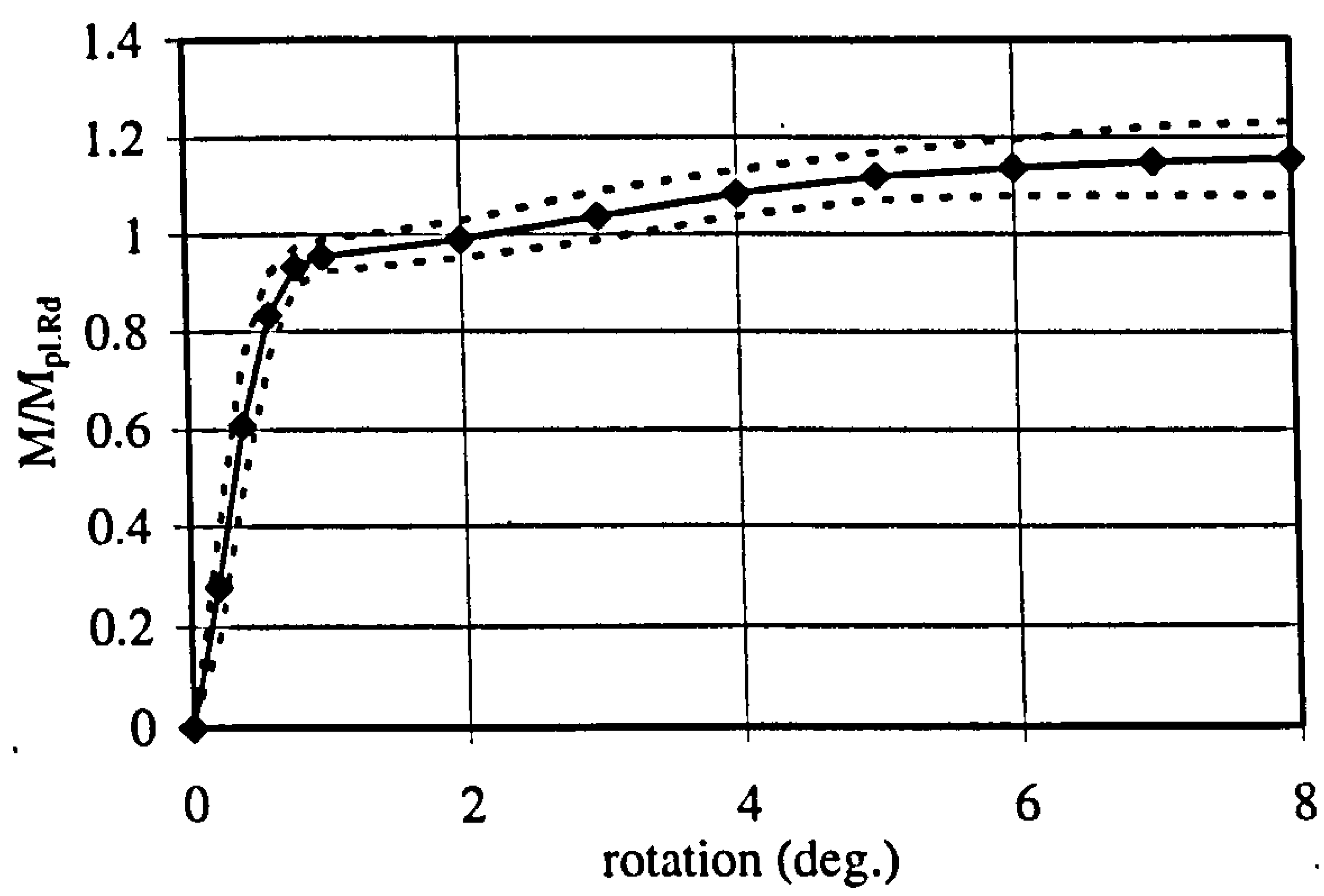


Fig. 8. 12: $M/M_{pl.Rd}$ vs. end rotation, showing the mean of the 20 tested sections, together with the upper and lower 95% confidence limits; M is the maximum experimental moment of resistance achieved up to the given rotation.

The statistical parameters summarising the comparison of predicted and experimental resistances are listed in Table 8. 5. The correction factor ($\bar{b}$) shows that the $M_{pl.Rd}$ resistance function underestimated the moment capacity of specimens by an average of 14%. The low degree of scatter between experimental and predicted resistances (σ_b) indicates that the objective of a low friction testing apparatus has been accomplished.

Test numbers	Section type	No. of tests	$\bar{b}$	σ_b
V1 to V10	203x102x23UB	10	1.127	0.045
W1 to W10	152x152x30UC	10	1.149	0.019
all		20	1.138	0.036

Table 8. 5: Statistical data obtained from the comparison of experimental with predicted resistances; a 6 degrees limit on end rotation has been imposed.

8.4 THE CALIBRATION OF γ_R^*

Listed below are the statistical parameters used to calculate γ_R^*. The measures of material variability are values determined from the analysis reported in Chapter 6. The factors $\bar{b}$ and σ_b are taken from Table 8. 5.

V_{fy} = 0.05

$V_{Wpl.y}$ = 0.02

$\bar{b}$ = 1.138

σ_b = 0.036

The calculations used to determine γ_R^* are listed below (for details on the statistical method used reference should be made to Chapter 5).

$$V_b = \frac{\sigma_b}{\bar{b}} = \frac{0.036}{1.138} = 0.032 \qquad (8.\ 1)$$

$$V_r = \sqrt{V_b^2 + V_{f_y}^2 + V_{w_{pl.y}}^2} \qquad (8.\ 2)$$

$$v_r = \sqrt{0.032^2 + 0.05^2 + 0.02^2} = 0.062 \qquad (8.\ 3)$$

$$\gamma_R^* = \frac{r_n \exp(0.5V_r^2 + \alpha_R.\beta.V_r)}{\bar{b}r_m} \qquad (8.\ 4)$$

$$\gamma_R^* = \frac{\exp(0.5x0.062^2 + 3.04x0.062)}{1.16x1.138} = 0.92 \qquad (8.\ 5)$$

This analysis shows that a γ_R^* factor of 0.92 is adequate to achieve the desired target reliability for the application of the $M_{pl.Rd}$ resistance function. This analysis was carried out without taking into account the effect of sample size. When the additional uncertainty of a limited sample size of 20 is considered, then the γ_R^* factor increases to 0.94.

The UK NAD sets γ_{M0} equal to 1.05, whilst the EC3 boxed value is 1.10. Although it is appreciated that numerical values of the γ_M-factors are chosen using a combination of calibration and judgement, it is of interest to determine the effect these seemingly high safety factors have on the probability of resistance falling below the design resistance. The target probability of this event occurring is 1 in 845. A probability set by $\alpha_R.\beta=3.04$, see equation (8. 4), where $\alpha_R.\beta$ represents the number of standard deviations between the mean resistance and design resistance.

As γ_{M0} is increased above the value necessary to achieve target reliability the design resistance is moved further away from the mean resistance and towards the extreme lower tail of the distribution of resistance. The corresponding effect on the probability of resistance falling below design resistance is dramatic. Table 8. 6 shows the effect that varying the γ_{M0} factor has on both the number of standard deviations between mean resistance and design resistance ($\alpha_R.\beta$), and on the pr(r<r_d).

Clearly, a γ_{M0}-factor of 1.0 substantially exceeds the target reliability. Increasing the value of γ_{M0} beyond 1.05 decreases the probability of resistance falling below design resistance to an amount so small that it is difficult to quantify. Instead it is represented more easily by the number of standard deviations between mean resistance and design resistance ($\alpha_R.\beta$), a factor that sets the location of design resistance on the lower tail of the distribution of resistance. Once again, when $\alpha_R.\beta=3.04$, the pr(r<r_d) is set to 1 in 845; when γ_{M0} is equal to 1.10, then $\alpha_R.\beta=5.99$, thus moving the design resistance almost twice as far from the mean resistance than is necessary. Since the design resistance is located on an extremely remote part of the lower tail of the distribution of resistance, the corresponding probability of resistance falling below design resistance becomes extremely low, as illustrated by Fig. 8. 13.

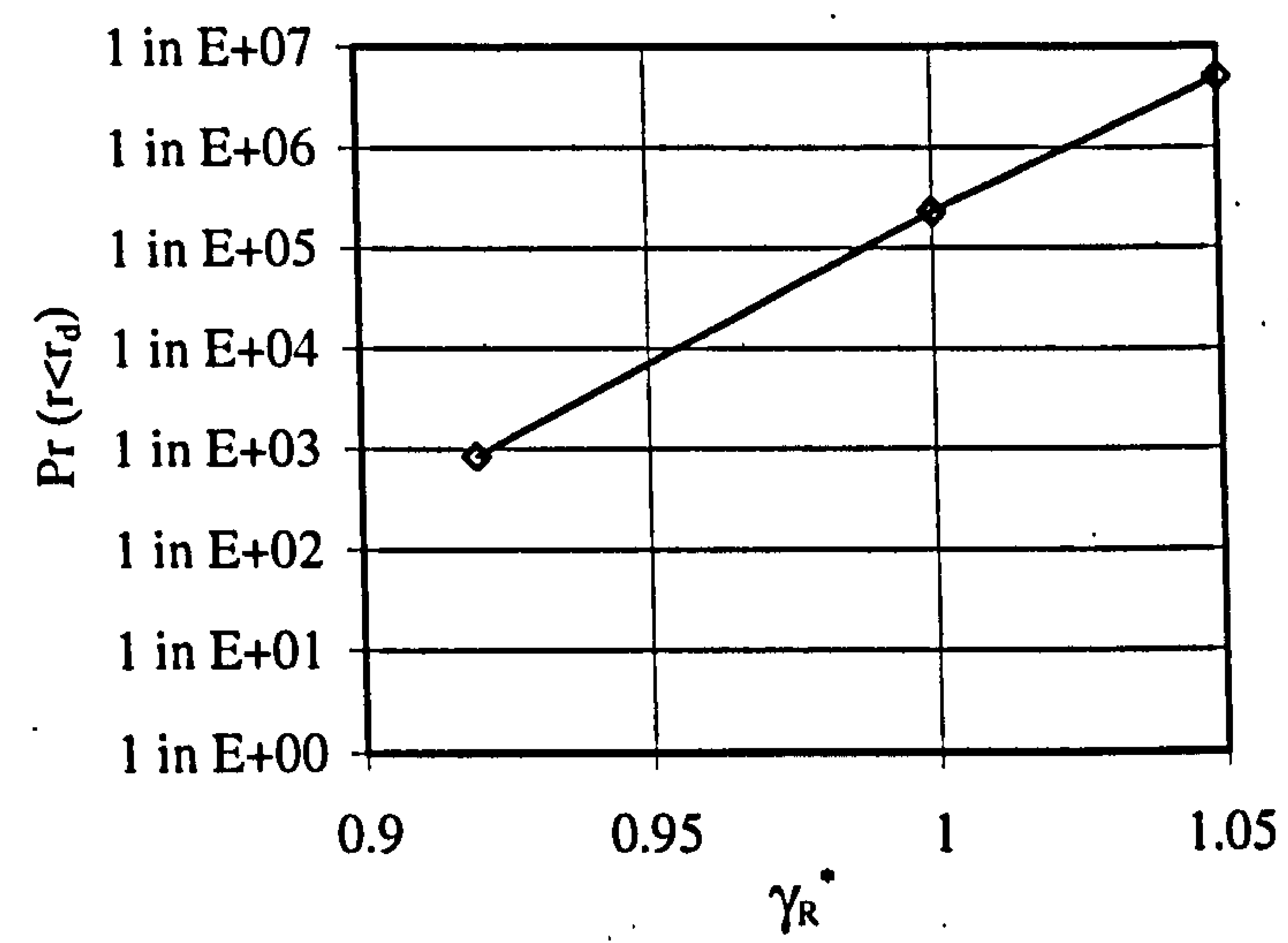


Fig. 8. 13

γ_{M0}	$\alpha_R.\beta$	pr(r<r_d)
0.92	3.04	1 in 845
1.00	4.45	1 in 230000
1.05	5.23	1 in 5000000
1.10	5.99	>1 in 5000000

Table 8. 6

8.5 CONCLUSIONS

These bending tests illustrate the controlled and ductile nature of the mechanism that characterises the failure of laterally restrained steel beams. The test specimens were able to accommodate additional loading after undergoing substantial buckling to the compression flanges.

The experimental load capacities of the specimens were observed to be consistently higher than the load capacity predicted from simple plastic design theory; (in which the resistance predictions were based on the actual material and geometric properties of the test specimens). On average the $M_{pl.Rd}$ resistance function ($W_{pl}.f_y$) underestimated the bending strength of class 1 sections by 14%. The standard deviation of experimental resistance over predicted resistance was 0.036. The bending tests were carried out with the non-dimensional slenderness set just below 0.4. Thus they correspond to the limit of the range of applicability for this resistance function.

Combining this measure of resistance function accuracy with measures of the variability of material and geometric properties (reported in Chapter 6), the value of γ_R^* necessary to achieve the target reliability is 0.9. Clearly it is impractical to utilise a partial safety factor of less than unity. This analysis does however present sufficient justification for the reduction of γ_{M0} applied to this resistance function from 1.05 to 1.0. The analysis clearly demonstrates that design using a γ_{M0}-factor of 1.05 substantially exceeds the desired target reliability. Thus, the full bending strength of restrained beams is not being exploited.

In Chapter 7 the reliability of plate girder design was evaluated. The analysis demonstrated that the value of γ_{M1} applied to plate girder design should be increased from its present value of 1.05 to 1.35 if the target reliability is to be achieved in a worst case scenario. Therefore, if the present values of the γ_M-factors are maintained, plate girder design should be considered substantially less reliable than restrained beam design.

The difference between the reliability of plate girders and restrained beams is due mainly to inconsistencies between the experimental and predicted resistances, due to the relatively high degree of instability characterising the failure of slender structural elements like girders. Thus, the proposal made in Chapter 7, that partial safety factors should be tailored to the requirements of individual resistance functions is confirmed.

Chapter Nine

UTILISING THE FULL BENDING STRENGTH OF RESTRAINED BEAMS

9.1 INTRODUCTION

The limited series of carefully conducted tests on rectangular hollow sections (Hasan and Hancock, 1988) has demonstrated the ease with which values of experimental moment capacity significantly greater than $M_{pl.Rd}$ can be achieved. The bending tests reported in Chapter 8 demonstrate that class 1 I-sections, like cold-formed hollow sections, are capable of failing in a controlled and ductile manner and attaining moments greater than $M_{pl.Rd}$. The experimental capacity of the test specimens was on average 16% higher than the design value, with little variation being observed about this mean value.

In a continuation of the work reported in Chapter 8, this chapter describes a series of bending tests carried out on fully restrained beams. These tests are used to establish whether the ultimate load capacity of fully restrained beams is substantially different to beams where the non-dimensional slenderness is just less than 0.4 (as was the case with the Chapter 8 bending tests). Further to this, theoretical predictions of the moment capacity of I-section beams have been made based on the use of the full material stress-strain curves together with moment-curvature techniques. Thus, an understanding of the mechanism by which the test specimens exceed the design load capacity is developed.

The bending test results reported in Chapter 8 demonstrate that class 1 I-section beams posses a considerably higher degree of strength than predicted by the plastic moment of resistance design method. The additional strength is derived from the strain hardening that characterises the deformation of typical structural steels. This consistent underestimation of resistance has lead to a partial safety factor considerably less than 1.0 in order to achieve the target reliability specified by CEN.

Clearly it would be inappropriate to recommend a partial safety factor less than unity. Therefore, the existing $M_{pl.Rd}$ design expression will be unable to fully utilise the ultimate moment capacity. In this chapter a modification is proposed to the $M_{pl.Rd}$ design expression that improves accuracy and allows the full moment capacity of restrained beams to be utilised. In addition, an alternative design method is proposed that utilises a non-linear model of stress distribution for moment calculation.

9.2 THE EXPERIMENTAL TESTING OF FULLY RESTRAINED BEAMS

Using the test apparatus described in Chapter 8, a series of 12 fully restrained beam tests have been carried out. The section designation and steel grade of the specimens are listed in Table 9. 1.

Test numbers	Sample size	Grade	Section designation
Y1 to Y6	6	FE430A	203x102x23UB
Z1 to Z6	6	FE430A	152x152x30UC

Table 9. 1

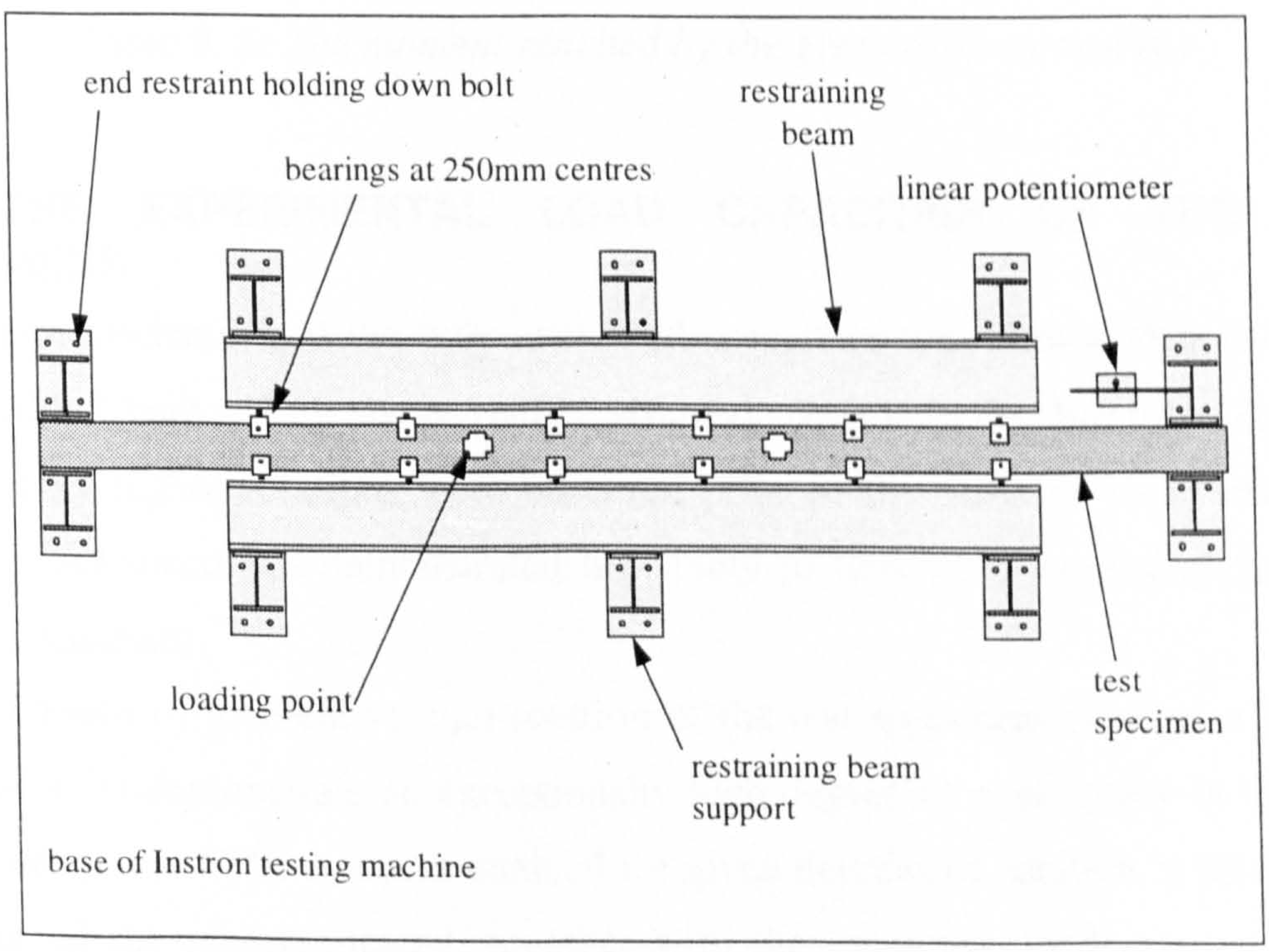


Fig. 9. 1: Plan of testing arrangement

During the testing of the partially restrained beams reported in Chapter 8, 4 roller bearings were used to provide lateral restraint to the compression flanges of the specimens. In order to model the full lateral restraint typically provided by the supported slab of restrained beams, 12 roller bearings were attached to the compression flanges of the test specimens reported in this chapter. Bearings were arranged at 250mm centres (illustrated in Fig. 9. 1). With the lateral restraints provided at such close centres, test specimens can be considered fully restrained against lateral movement.

Test No.	$\phi=1$ M kN.m	$\phi=2$ M kN.m	$\phi=3$ M kN.m	$\phi=4$ M kN.m	$\phi=5$ M kN.m	$\phi=6$ M kN.m	$\phi=7$ M kN.m	$\phi=8$ M kN.m	M_{max} kN.m
Y1	69	72	75	79	82	86	86	72	86.1
Y2	70	73	78	82	87	88	91	93	94.2
Y3	71	74	76	82	85	88	79	72	87.7
Y4	69	73	76	81	84	87	89	91	91.9
Y5	69	71	75	79	82	85	87	88	88.4
Y6	69	71	75	79	82	85	85	65	85.7
Z1	65	67	70	74	77	80	82	83	83.0
Z2	64	67	69	73	76	79	81	83	83.3
Z3	65	67	71	74	77	79	81	82	82.0
Z4	65	67	71	72	76	78	81	82	82.4
Z5	65	67	70	74	76	80	81	80	81.7
Z6	65	68	71	74	79	80	83	84	84.7

Table 9. 2: The moment reached by the given end rotation (ϕ)

9.2.2 THE EXPERIMENTAL LOAD CAPACITIES OF THE TEST SPECIMENS

The failure mechanism of the fully restrained specimens was almost identical to that described in Chapter 8 for the partially restrained specimens, the difference being that considerably higher rotations were observed prior to the onset of local and overall buckling. All specimens demonstrated the ability to develop a controlled and stable failure mechanism.

Graphs of moment vs. end rotation of the test specimens (shown in Fig. 9. 2 and Fig. 9. 3) demonstrate an exceptionally high degree of consistency in these two sets of beam tests. The moment attained for given degrees of rotation is listed in Fig. 9. 2 for all the sections tested, together with the maximum load resisted by each specimen.

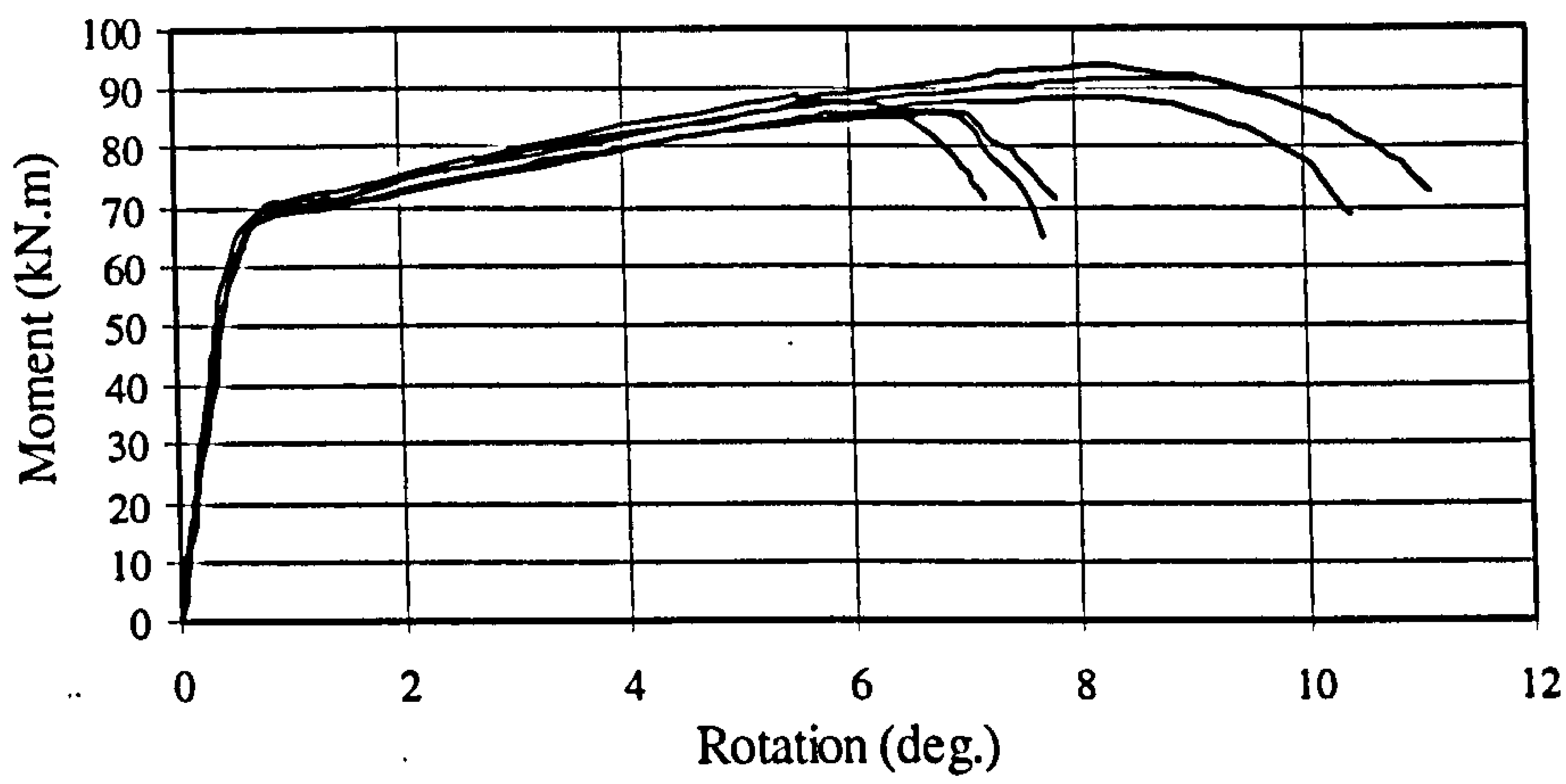


Fig. 9. 2: 203x102x23UB

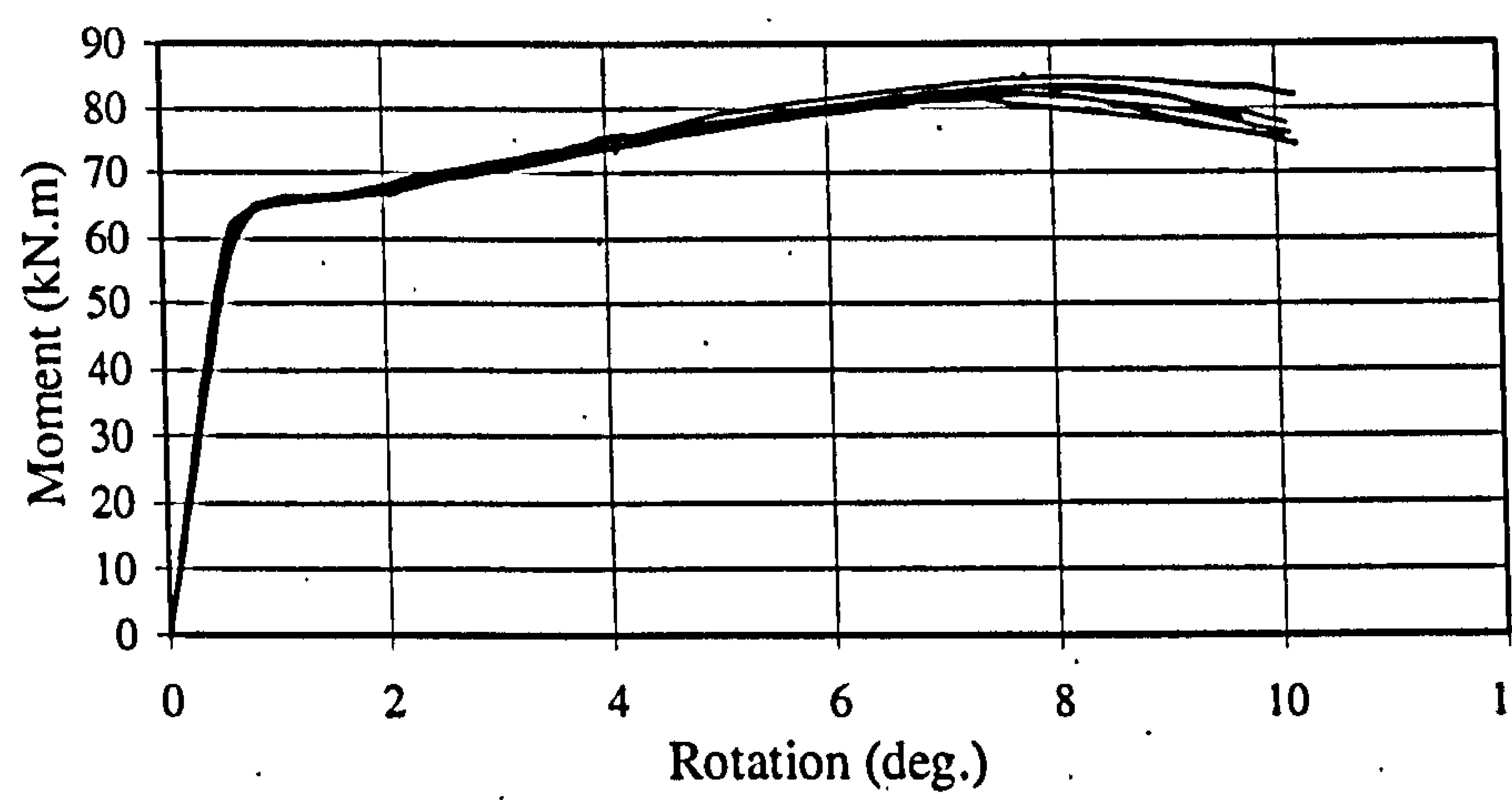


Fig. 9. 3: 152x152x30UC

Test No.	Section type	B1 mm	B2 mm	D1 mm	D2 mm	t1 mm	t2 mm	T1 mm	T2 mm	T3 mm	T4 mm	$W_{pl.y}$ cm^3
Y1	203x102x23	102.0	102.0	202.0	202.0	5.76	5.58	8.74	8.60	8.58	9.05	225.1
Y2	203x102x23	102.5	103.0	203.0	203.0	5.73	5.80	9.08	8.33	8.88	8.77	229.0
Y3	203x102x23	102.5	102.0	203.0	202.0	5.64	5.50	8.74	8.86	8.87	9.04	227.8
Y4	203x102x23	103.0	103.0	202.5	202.5	5.65	5.80	8.64	9.08	8.64	8.61	228.0
Y5	203x102x23	103.0	103.0	203.0	202.5	5.57	5.75	8.62	9.00	8.88	8.67	228.7
Y6	203x102x23	102.0	102.0	203.0	203.0	5.63	5.51	9.07	8.86	8.89	8.73	228.2
Z1	152x152x30UC	152.0	151.5	157.5	157.5	5.99	6.10	9.07	8.90	8.92	9.29	236.5
Z2	152x152x30UC	152.0	151.5	157.0	157.5	5.62	5.97	9.00	9.18	9.16	8.95	235.4
Z3	152x152x30	151.5	150.5	156.0	157.0	6.13	5.95	8.97	9.15	9.12	8.96	233.8
Z4	152x152x30UC	151.0	150.5	156.5	157.0	6.16	6.06	9.01	9.10	9.27	8.97	235.0
Z5	152x152x30UC	151.0	151.5	156.5	156.5	5.98	5.88	9.20	8.96	8.92	9.03	233.1
Z6	152x152x30UC	151.0	151.5	156.0	157.0	6.00	6.12	9.03	9.28	9.05	8.85	234.2

Table 9. 3: The geometric properties of the test specimens

9.2.3 GEOMETRIC PROPERTIES OF TEST SPECIMENS

The geometric properties of each test specimen were recorded in order to determine the theoretical moment capacity. These measurements are listed in Table 9. 3, along with the major axis plastic section modulus computed from these measurements. The notation used in Table 9. 3 is illustrated in Fig. 8. 9.

Test No.	Grade	flange f_y N/mm^2	flange f_u N/mm^2	flange % elongation	web f_y N/mm^2	web f_u N/mm^2	web % elongation	flange E N/mm^2
Y1	FE430A	303	470	24	308	462	24	194230
Y2	FE430A	330	500	17	336	509	23	213920
Y3	FE430A	330	480	25	354	516	24	181740
Y4	FE430A	317	484	24	345	511	23	211650
Y5	FE430A	314	486	25	346	483	24	200500
Y6	FE430A	316	476	24	325	475	27	201270
Z1	FE430A	284	482	36	403	565	25	208150
Z2	FE430A	290	479	36	353	528	29	188290
Z3	FE430A	284	476	36	354	509	30	210000
Z4	FE430A	285	476	36	353	543	30	210000
Z5	FE430A	292	476	38	358	531	27	210000
Z6	FE430A	283	482	38	366	555	27	210000

Table 9. 4: The material properties of the test specimens

9.2.4 MATERIAL PROPERTIES OF TEST SPECIMENS

Mill tests were carried out to determine the material properties of test specimens in accordance with BS EN10002-1:1990. Material properties are listed in Table 9. 4. The yield stress corresponds to 0.5% total strain and it is not a proof stress.

Fig. 9. 4 shows the mean values of stress vs. strain taken from the 12 mill tests. Also shown are the upper and lower 95% confidence limits. Of these only a minority ($^3/_{12}$) produced stress vs. strain profiles approaching the classical shape; upper and lower yield points followed by a brief period of yielding without increasing material strength. The majority ($^9/_{12}$) of test coupons began strain hardening immediately following yielding.

Fig. 9. 4 demonstrates the decreased variability of material strength with increasing strain. The yield point appears to be associated with a considerable degree of variation. In contrast, the material strength corresponding to 3% strain is associated with a reduced amount of variation (indicated by the upper and lower 95% confidence limits).

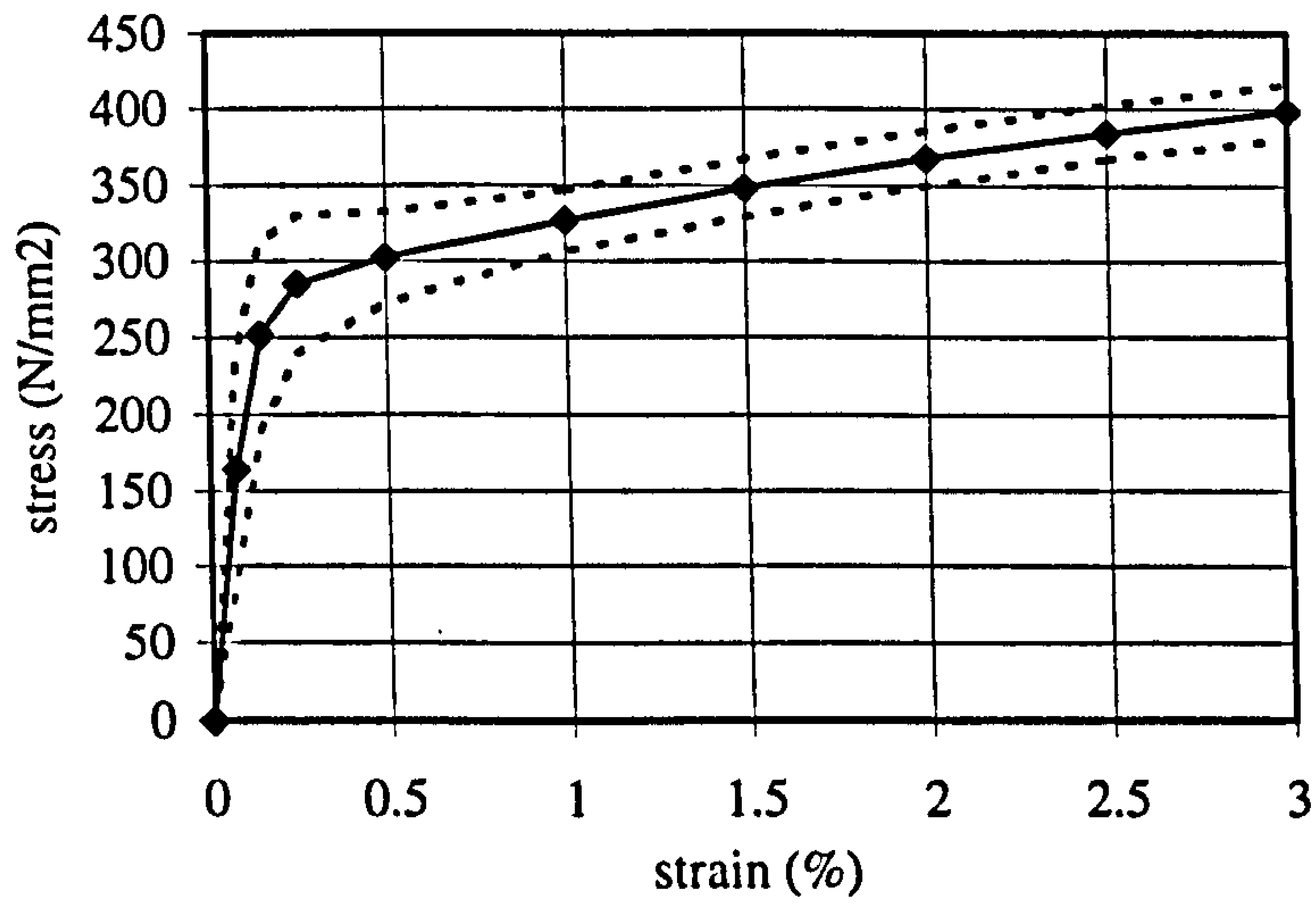


Fig. 9. 4: Stress vs. strain, showing the mean values from the 12 specimens together with the upper and lower 95% confidence limits. Data based on mill tests taken from the tension flanges.

9.2.5 CREEP EFFECTS

A test was carried out to briefly investigate creep effects during the formation of a failure mechanism. The test was carried out on beam Z6, the results of which are shown in Fig. 9. 5 and Fig. 9. 6. These show a characteristic response to static loading within the plastic region; initially the specimen deforms whilst loading remains constant. Over time the rate of deformation reduces until the specimen stabilises. Based on this test, the load capacity recorded during the plastic region represents a relatively sustainable resistance, given that the investigation concerns an ultimate limit state.

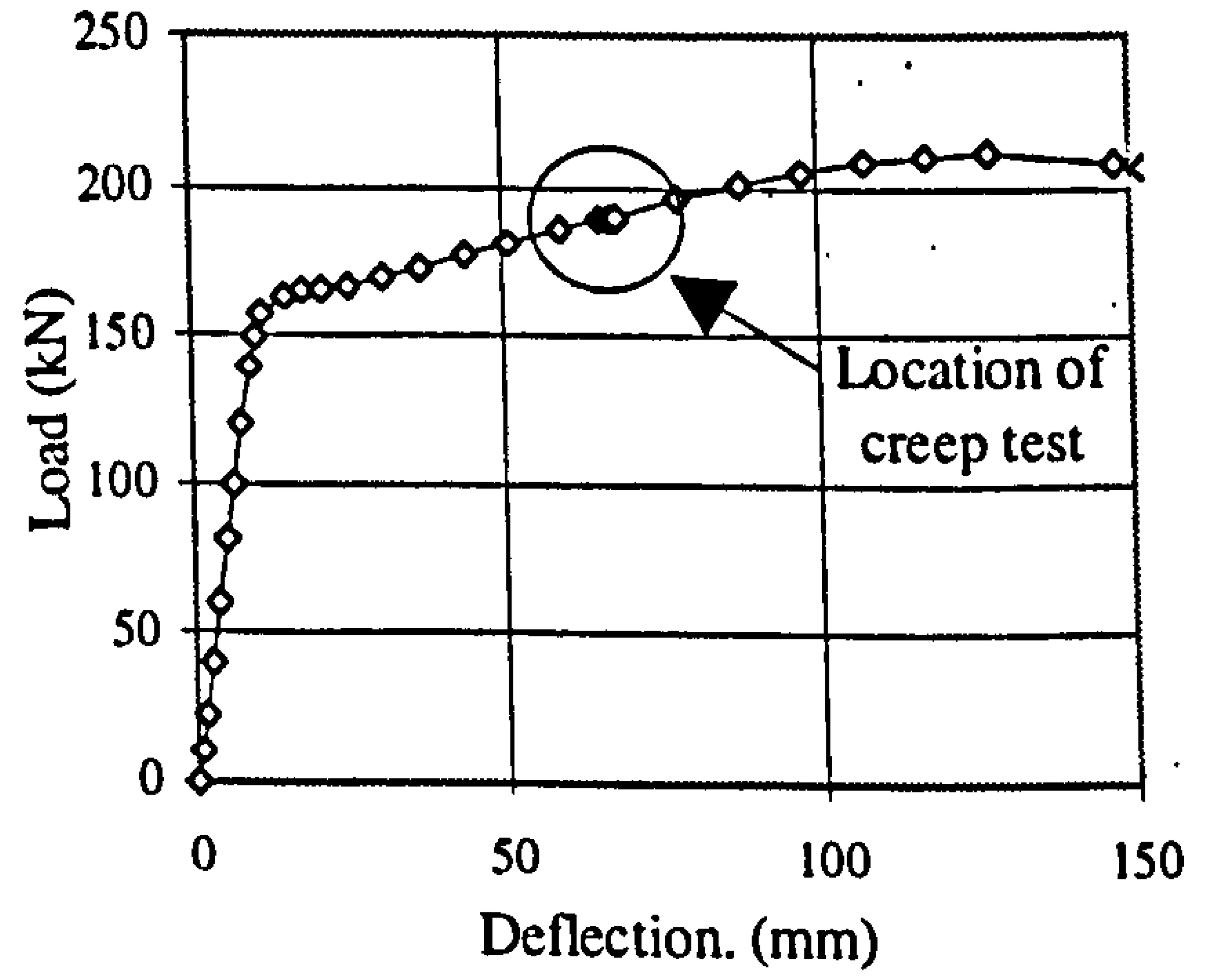


Fig. 9. 5: Z6 creep test

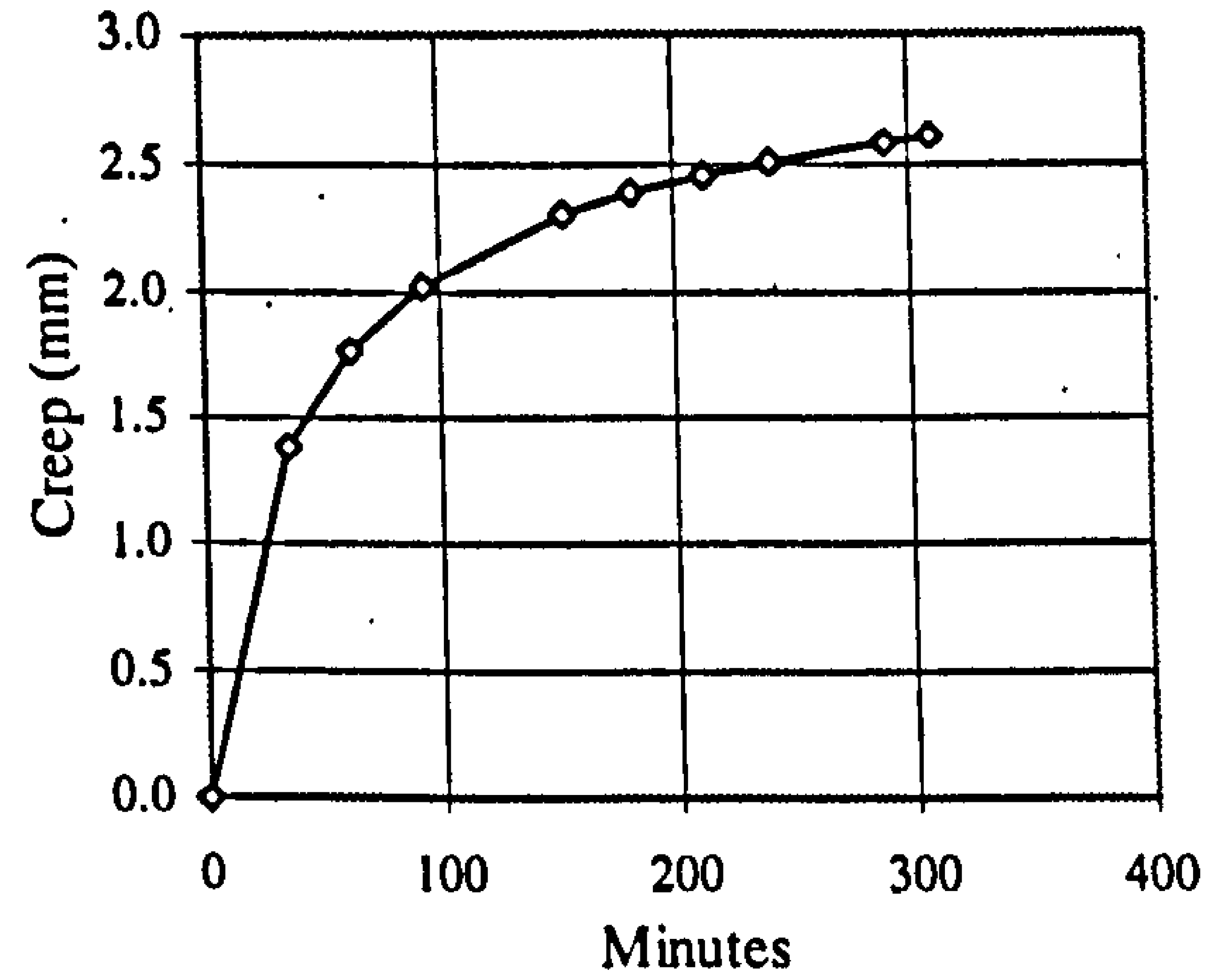


Fig. 9. 6: Z6 creep test

9.3.6 STRAIN DISTRIBUTION DURING PLASTIC DEFORMATION

In order to examine the distribution of strain throughout the rotation of a plastic hinge, strain gauges were distributed throughout the depth of the section of certain of the test specimens. Fig. 9. 7 shows the distributions of strain throughout the testing of specimen Z3. The strain vs. distance from the section centre-line is shown for various points during the test, up to and including the point of maximum load capacity. Fig. 9. 8 shows the relationship between the end rotation and the strain recorded at the outer-edges of the section.

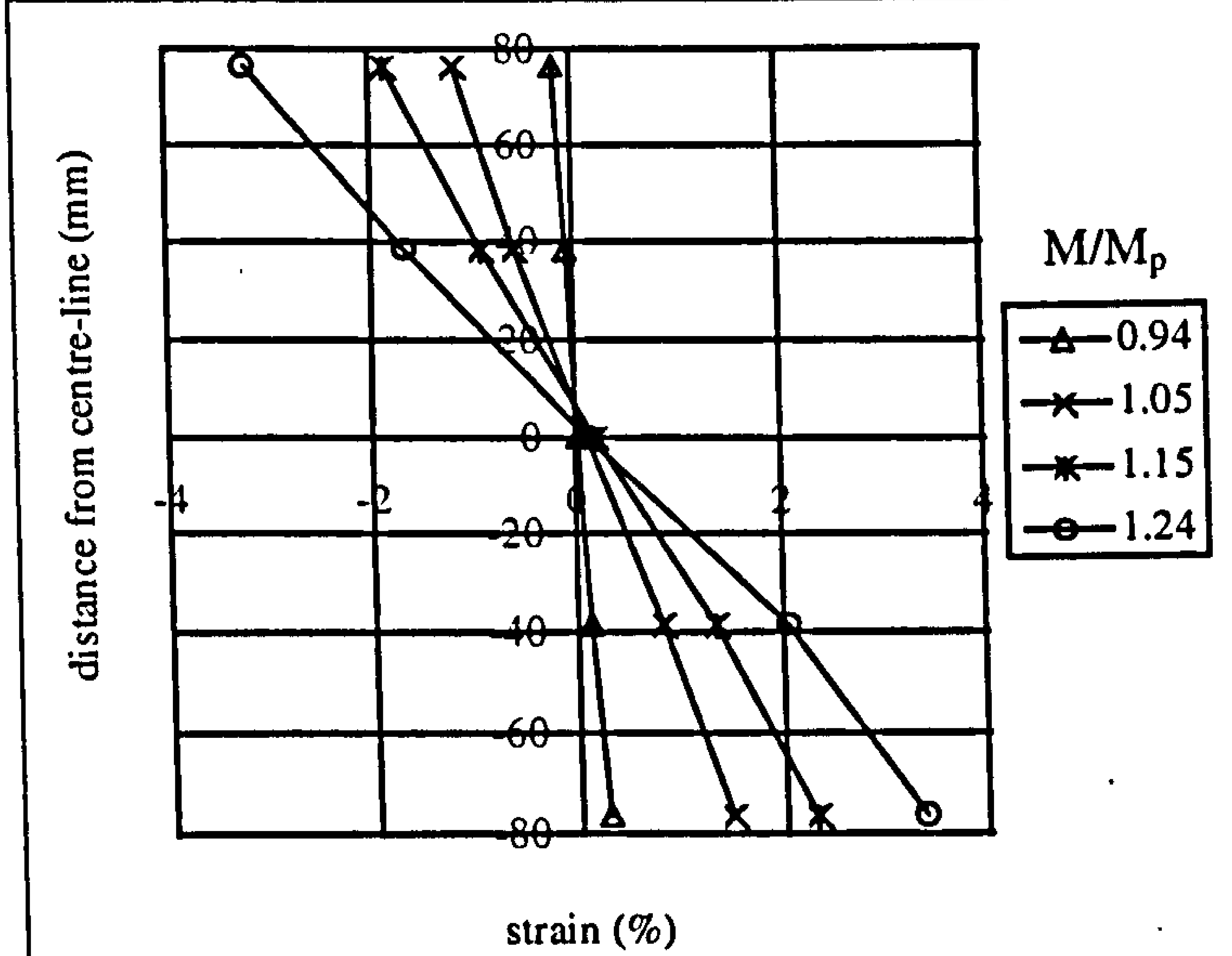


Fig. 9. 7: The distributions of strain (test specimen Z3)

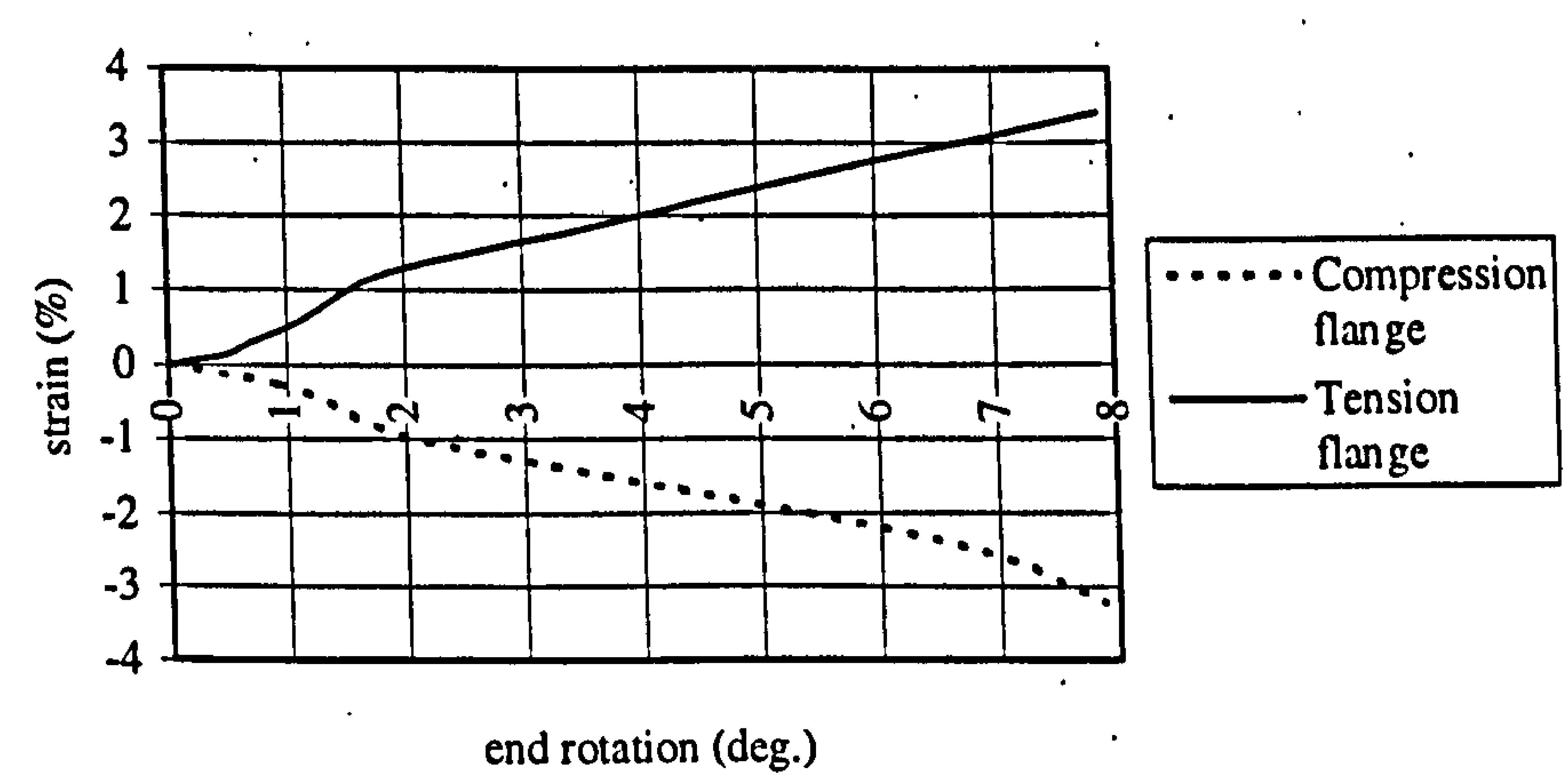


Fig. 9. 8: Outer-edge strain vs. end rotation up to and including the point of maximum load (test specimen Z3)

The maximum load capacity of the section corresponds to a maximum outer strain of approximately 3%. This figure is representative of strain gauge readings recorded for the other test specimens. Fig. 9. 7 demonstrates that the distribution of strain remains linear throughout the plastic deformation of the test specimen. An observation is that the neutral axis moves towards the compression flange. Since the movement is relatively small, it should not have significantly affected the resistance of the section. Thus, the basic assumptions made during the calculation of $M_{pl.Rd}$ - that the distribution of strain throughout the depth of the section is linear and that the compression and tension stress vs. strain relationships are equal and opposite are justified given the need for a workable numerical method.

9.3 THE THEORETICAL LOAD CAPACITY OF RESTRAINED BEAMS

During the formation of a plastic hinge, material located in the tension and compression flanges undergoes considerable straining. When the strain exceeds a certain point the material begins to strain harden, thus the material strength begins to exceed the stress assumed during plastic and elastic design. This increased material strength results in the increased load capacity of steel sections over that assumed from simple plastic design, providing the plastic hinge can accommodate sufficient rotation without failure via an alternative mechanism.

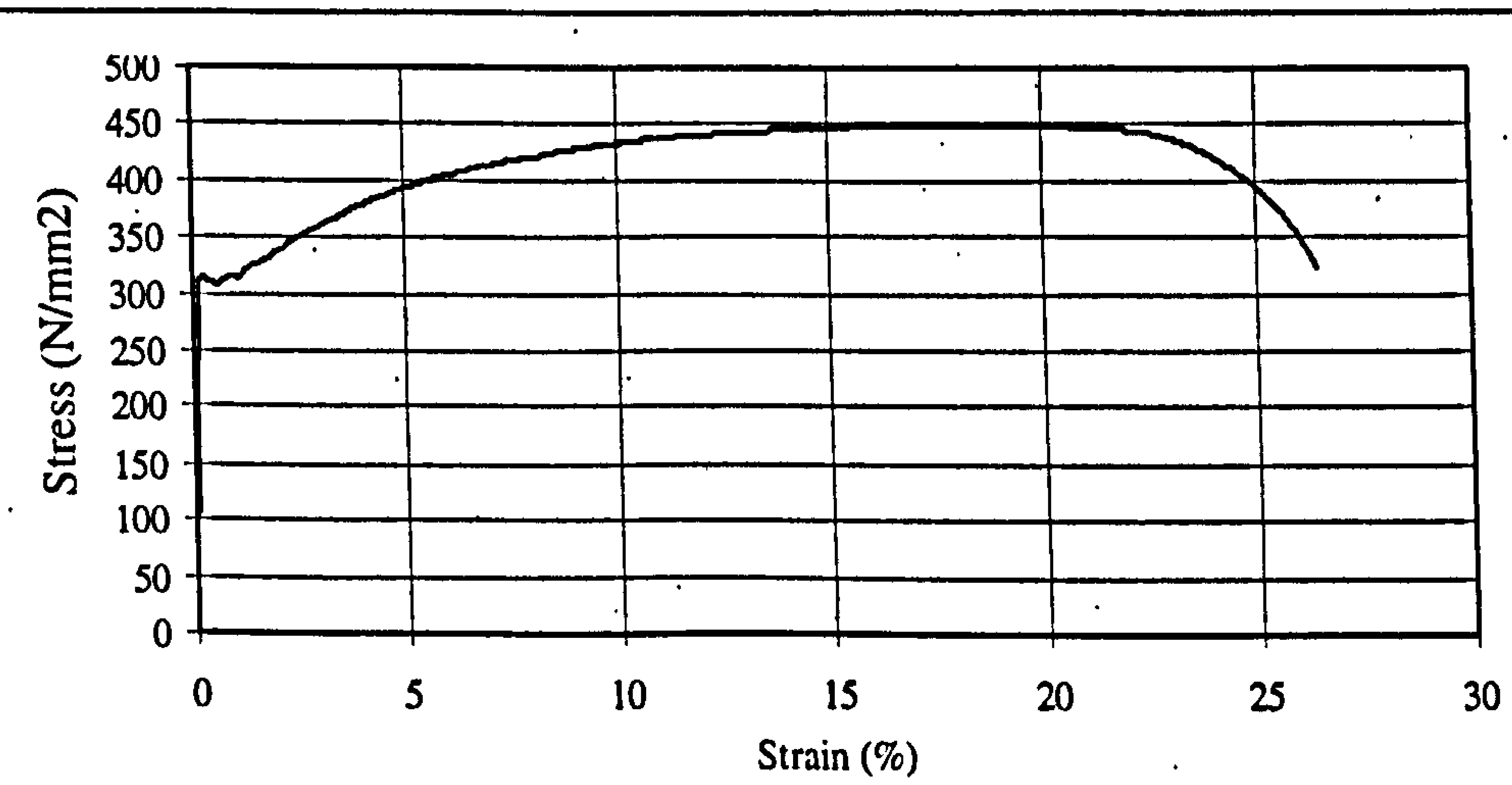


Fig. 9. 9: Stress vs. strain for a typical grade 43a hot rolled steel

The characteristic shape of the curve is a linear stress strain relationship prior to the onset of yielding, followed by a short plastic region in which additional strain is achieved without increasing stress. After this strain hardening effects become important. The material can accommodate considerably higher loading before reaching its ultimate tensile strength although additional loading will result in considerable permanent deformation. Typically, hot rolled steel can accommodate between 25 to 35% strain before fracture. The ultimate tensile stress will be some 50% higher than the yield stress, although this strength will not be reached until a strain of some 15 to 20% is reached. Clearly it is impractical for structural members to develop strains large enough to utilise the ultimate tensile strength of the material. However, during the development of a plastic hinge, restrained beams develop strains in the extreme fibres of the section large enough to partially incur stain hardening effects.

In theory, cold-formed steels should have a reduced capacity for strain hardening, since the margin between yield strength and ultimate tensile strength is reduced. This is because cold-formed steels are work hardened during manufacture, i.e, a proportion of the strain hardening capacity is utilised during the manufacturing process.

Bending tests reported in (Hasan and Hancock, 1988) on cold-formed rectangular hollow sections demonstrate that despite the mean ultimate tensile stress of the sections being only 1.17 times the yield stress, sections were capable of exceeding the predicted plastic moment capacity by an average of 23%. This is an interesting finding, since the maximum theoretical moment capacity of a section is slightly less than the ultimate tensile stress multiplied by the plastic section modulus. In this case such a calculation would produce an experimental load capacity nearly 17% greater than the plastic moment capacity, providing that the member was able to accommodate an impractical amount of rotation without failing via another mechanism first.

Hasan and Hancock's tests demonstrate the simplicity and limitations of the approach to calculating bending strength based on simple tensile coupon tests. During the rotation of a plastic hinge, it is likely that the true strength of material located in the plastic hinge will be different to the strength of coupons taken from the specimen and tested in pure tension, due to the complex nature of the 3-D stress system found in a plastic hinge.

It is still of interest to understand the theoretical strain hardened resistance of steel members, ignoring possible failures from local buckling or other failure mechanisms. The theoretical strain hardened resistance has been established by assuming a linear distribution of strain throughout the depth of the plastic hinge. The corresponding stress for each portion of a section is calculated from a graph of stress vs. strain taken from a tensile test, where the relationship between stress and strain is assumed to be the same in tension and compression. Both assumptions seem justified following the analysis of data obtained from strain gauges attached to test specimens, such as those already reported for specimen Z3.

Using this technique, the distribution of stress shown in Fig. 9. 10 has been calculated (where the maximum strain in the outer fibres of the section was taken as equal to 3%). The stress vs. strain graph sketched in Fig. 9. 9 was used for generating this profile.

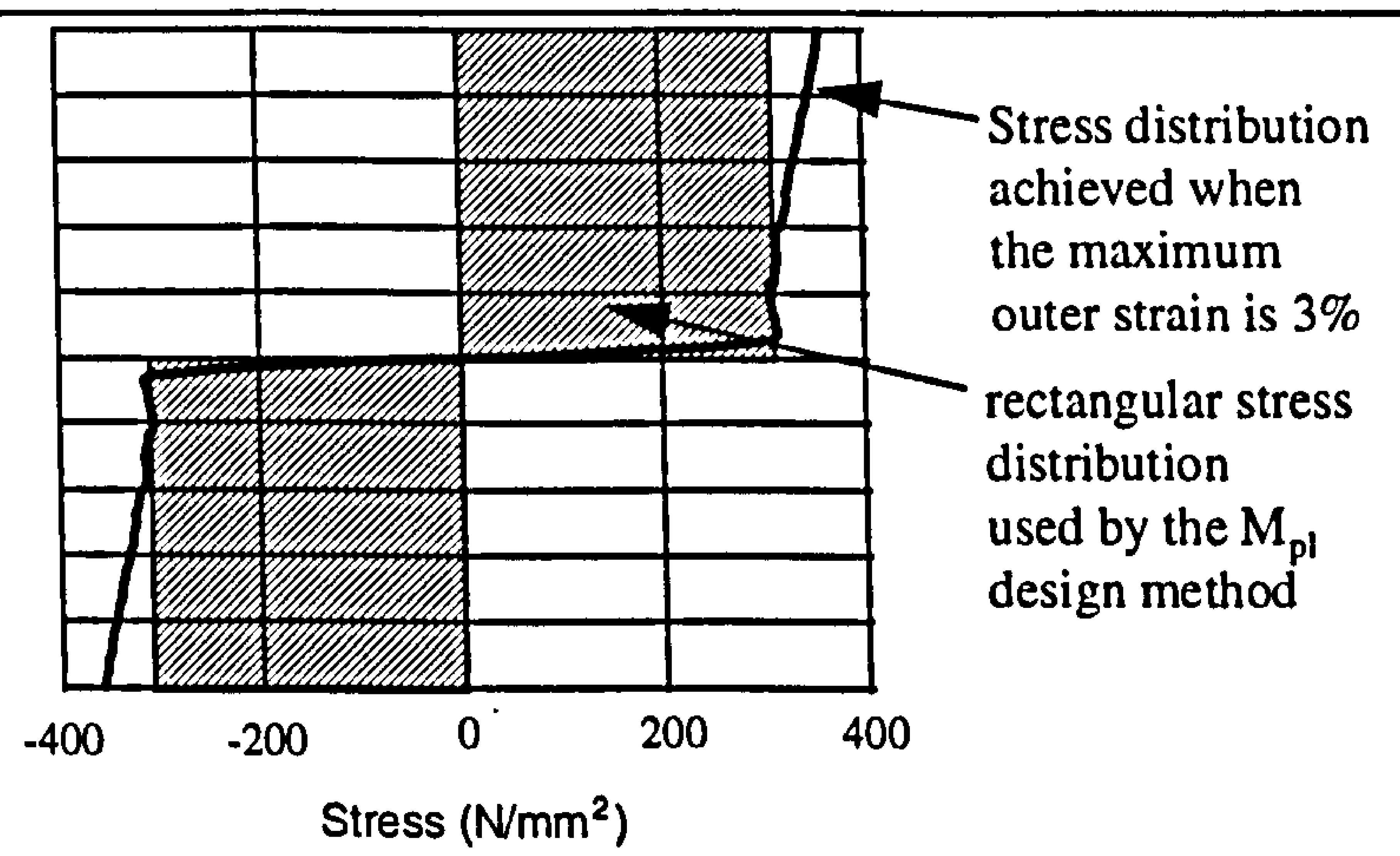


Fig. 9. 10

Fig. 9. 10 illustrates the contribution that various components of an I-section make to the moment capacity. This highlights the importance of strain hardening, since partial strain hardening is capable of increasing the material strength of the outermost fibres of the section (which contribute the majority of moment capacity to the member). The figure shows that the rectangular stress block assumption utilised by the $M_{pl.Rd}$ resistance function does provide a conservative reflection of the observed distribution of stress.

Fig. 9. 11 shows the various stress distributions achieved for differing amounts of outer-edge strain. Again the figure is generated using the stress strain graph sketched in Fig. 9. 9. At an outer strain of 0.16% the section has just reached the yield stress. The distribution of stress is therefore almost linear, in line with elastic theory. At a strain of 1% the distribution of stress is almost identical to the plastic design stress block assumption, as the material has strained without strain hardening. At an outer strain of 5%, a considerable degree of strain hardening has taken place, with the material located in the flanges exhibiting a stress approximately 20% greater than the

yield stress. The distribution of stress is also shown for outer strains of 10% and 20%, although strains of this magnitude are associated with an impractical amount of rotation. These stress distribution profiles are useful for illustrating the progressive movement of strain hardened material towards the centre of the section.

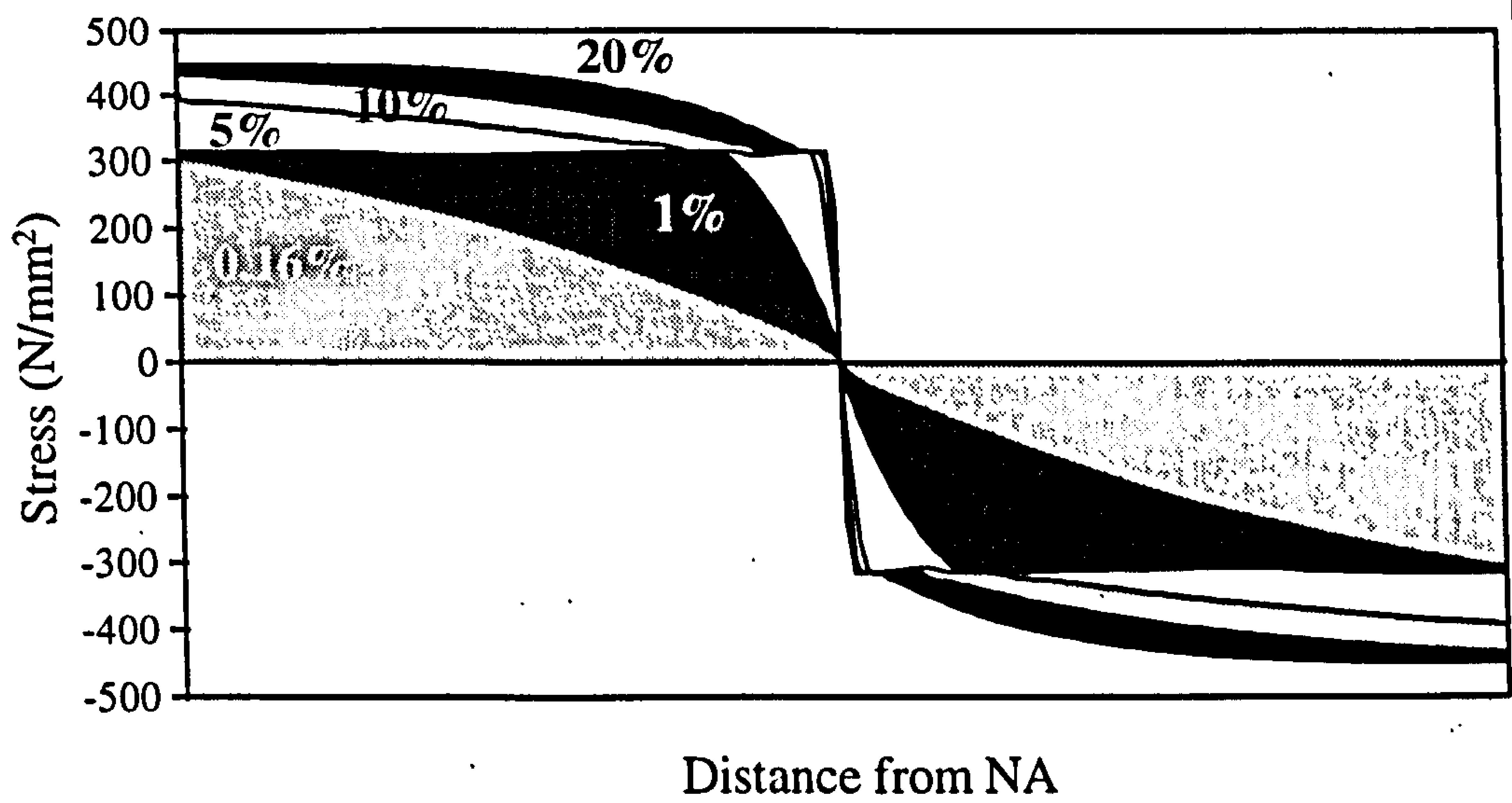


Fig. 9. 11: The stress distribution profiles for varying strain, the maximum outer strain is shown on each profile

Fig. 9. 12 shows the graph of stress vs. strain taken from the mill test of specimen Z3. The curve shows a typical shape exhibited by the batch of steel used in this testing program; a notable lack of an upper and lower yield point combined with the early onset of strain hardening. This shape is different to the classical profile shown in Fig. 9. 9. However, following discussions with British Steel this stress vs. strain relationship would appear to be representative of modern hot rolled steels.

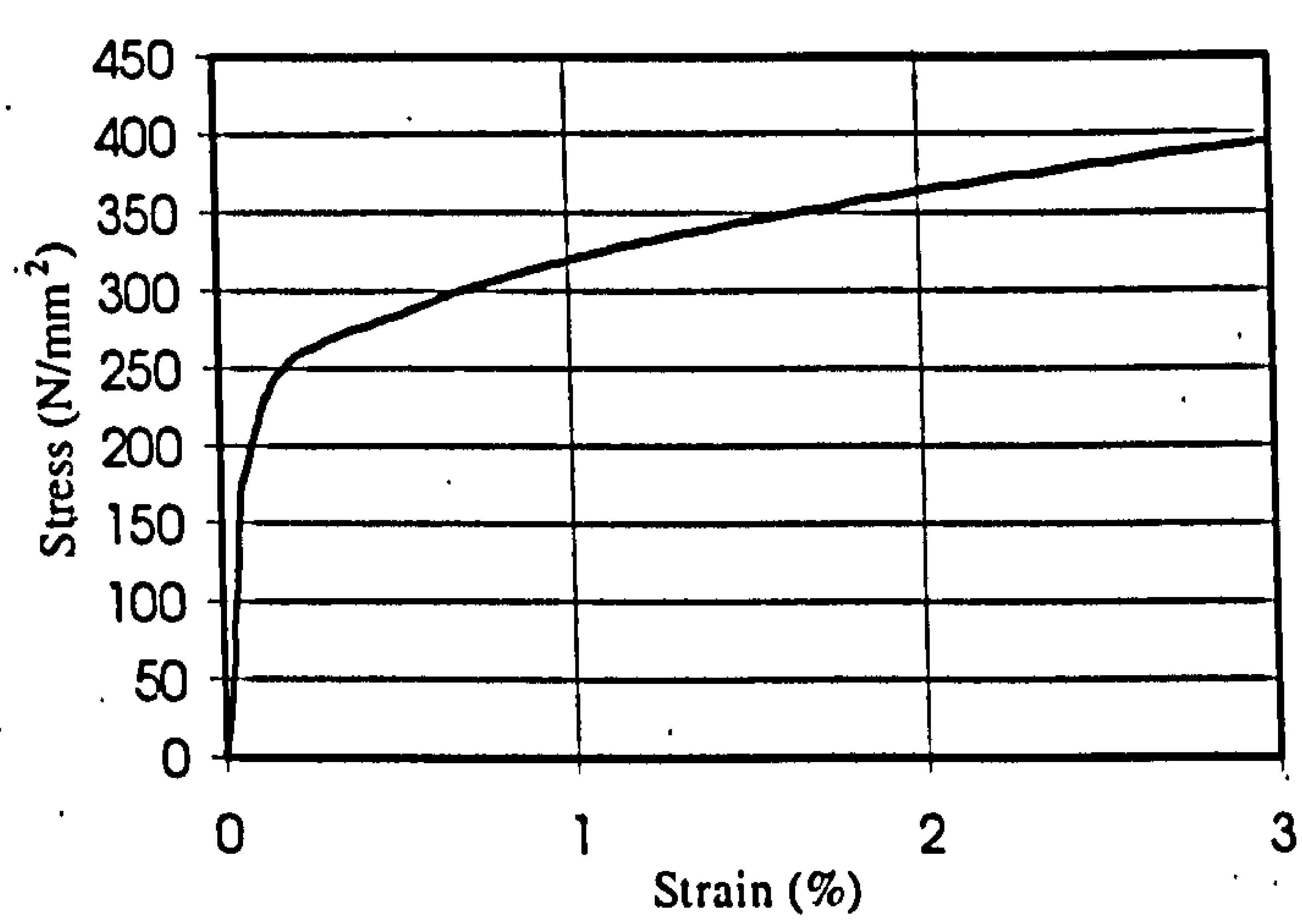


Fig. 9. 12: Stress vs. strain for test specimen Z3

Fig. 9. 13 shows the theoretical distribution of stress throughout specimen Z3. Stress was calculated from the stress vs. strain graph shown in Fig. 9. 12 using the assumption that the strain distribution is linear. Considerable strain hardening has

occurred after as little as 3% outer strain of the section. Fig. 9. 7 (showing the distribution of strain) throughout the depth of the section demonstrates that in practice the strain corresponding to the point of maximum moment resisted by specimen Z3 is approximately 3%. Therefore, these distributions of stress can be realistically achieved prior to failure via by a buckling type mechanism.

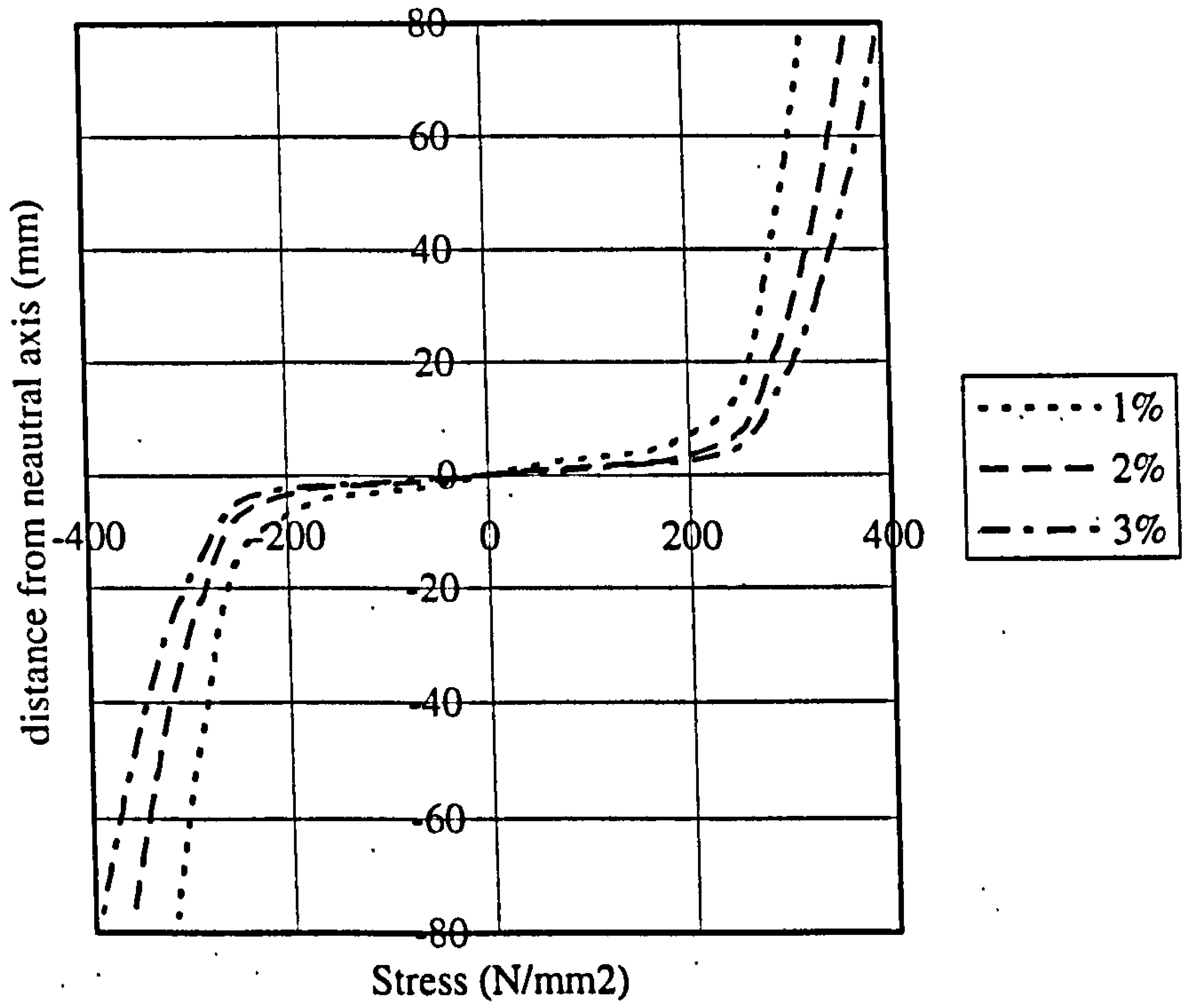


Fig. 9. 13: The theoretical distribution of stress in the plastic hinge of specimen Z3, the profiles correspond to 1, 2 and 3% of outer-edge strain.

Fig. 9. 14 illustrates the relationship between the maximum outer edge strain attained during the rotation of a plastic hinge with the normalised theoretical moment of resistance. Given enough rotation to produce a 3% outer-edge strain the theoretical moment capacity of a section is just under 1.4 times the plastic moment of resistance. In practice, Fig. 9. 7 demonstrates that specimen Z3 produced an experimental moment capacity of $1.24M_{pl.Rd}$ for a maximum outer edge strain of 3%. Thus, the theoretical moment capacities shown in Fig. 9. 14 are overestimates.

This difference may be partly due to the considerable buckling observed in the compression flange of the test specimens. Although sections remained able to withstand additional loading, this local buckling is certain to have reduced the moment capacity. Had the compression flange remained in plane throughout the test, the theoretical and experimental moment capacities may have been in closer agreement.

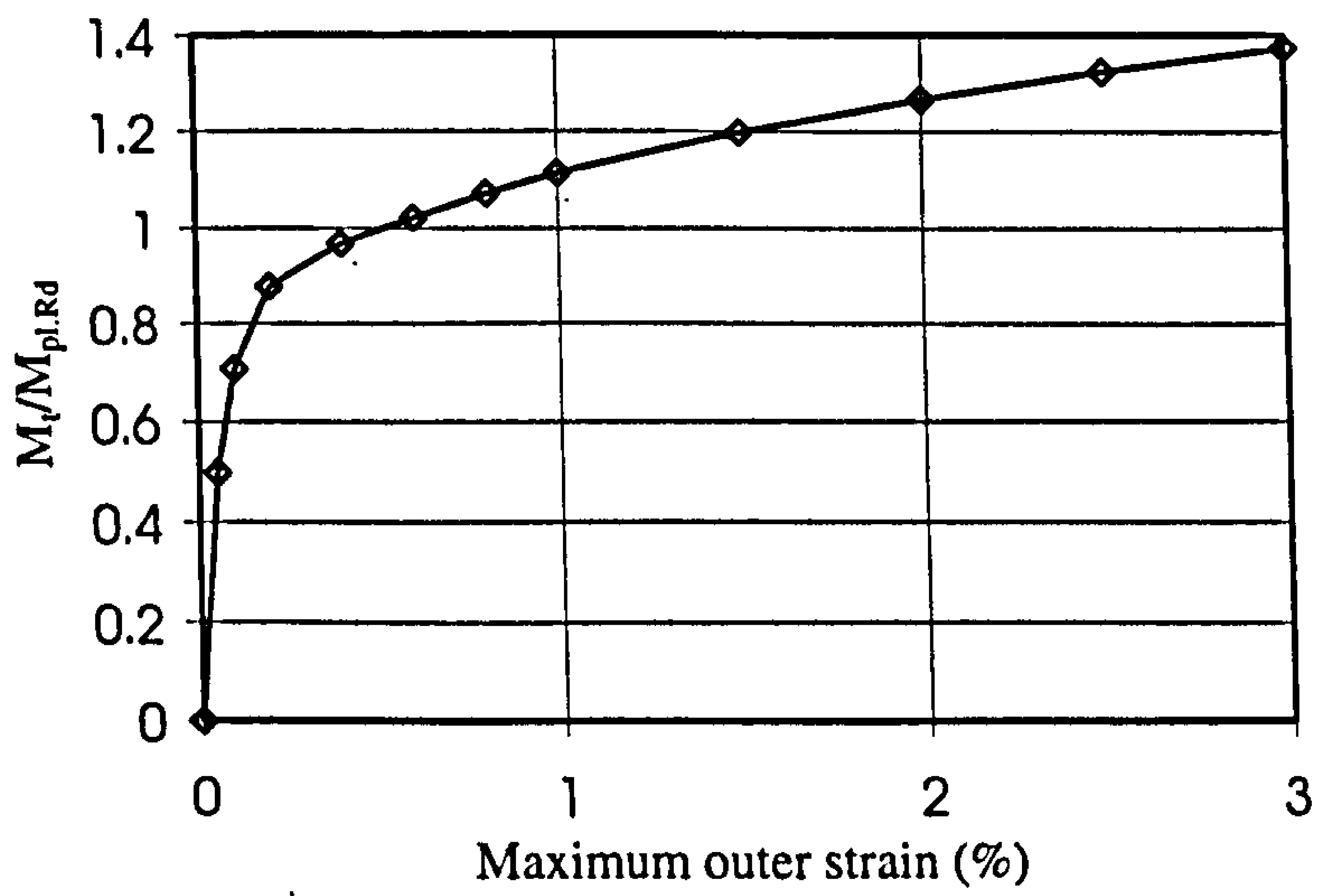


Fig. 9. 14: $M_t/M_{pl/Rd}$ vs. the maximum outer strain, where M_t is the theoretical moment capacity determined using the moment rotation technique

9.4 THE PLASTIC MOMENT OF RESISTANCE

In Chapter 8, the γ_R^*-factor applied to the $M_{pl.Rd}$ resistance function was calibrated using bending test results where the non-dimensional slenderness was set just below 0.4. The bending tests reported in this chapter will be utilised to once again determine γ_R^* for the $M_{pl.Rd}$ function, and in so doing, determine whether the reliability of this function is significantly different for fully restrained beams, as opposed to partially restrained beams. Since the majority of steel beams are of the fully restrained type, any additional reliability may be worth utilising.

9.4.1 THE METHOD

The application of plastic theory to the design of steel structures was pioneered by Baker with publication of the classic volume entitled "The steel skeleton" (J. F. Baker, 1956). In that work Baker credits (Ewing, 1899) for development of the familiar formula (9. 1) for calculating the plastic moment of resistance of a rectangular bar - now termed $M_{pl.Rd}$ in Eurocode 3.

$$M_{pl.Rd}=0.25\, f_y . b . h^2 \qquad (9.\ 1)$$

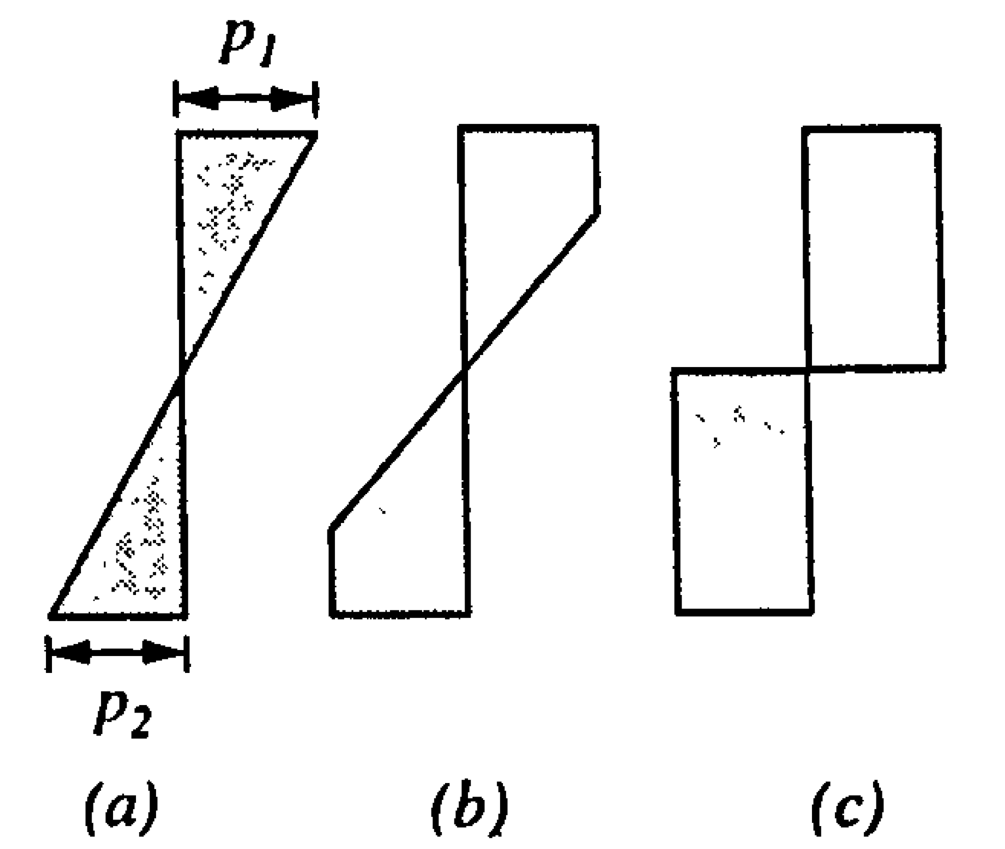


Fig. 9. 15

In (Ewing, 1899) a full account is given of the effect that bending sections past their elastic limit has on stress distribution. Ewing said:

The assumption...that a bending moment gives rise to a uniformly-varying distribution of stress applies only when the material is homogeneous and when the greatest intensity of stress falls below the elastic limit (Fig. 9. 15a)...

If however, the bending moment is increased, non-elastic strain will begin as soon as either p_1 or p_2 exceeds the corresponding limit of elasticity. The distribution of stress will then be modified. The outer layers of the beam are taking permanent set while the inner layers are still following Hooke's law. As a simple instance it will be sufficient to consider in a general way a material which is strictly elastic up to a certain limit of stress, and then so plastic that any small addition to the stress produces a relatively very large amount of strain - a case not far from being the case in good wrought or mild steel. When a beam of such material is overstrained the diagram exhibiting the distribution of stress will take a form generally resembling the sketched (Fig. 9. 15b).

...if the material tested is in the form of a rectangular bar...the distribution of stress may approach an ultimate condition in which the upper half of the section is in uniform tension f_t and the lower half is in uniform compression of the same intensity (Fig. 9. 15c). The moment of the stress is then equal to $1/4 f_t\, bh^2$...

Baker provided a commentary to explain why the pioneering work of scientists like Ewing took years to be applied. He said:

The fate of this early work lends point to the assertion...that the applied scientist's task is particularly onerous. It is not enough for him to throw out an idea, however novel or attractive, and leave it at that. Nor is it particularly helpful if in his research he leaps forward, avoiding some nasty jagged rocks. Someone must deal with those difficult places before the designer can follow the new road. The applied scientist's aim, in fact, should be to push forward the boundary of knowledge in an unbroken line, surveying all the country as he goes, even though his observations and deductions cannot be, at the first attempt, as precise as those made by the pure scientist in dealing with more limited objectives.

Clearly, Baker's work including "The steel skeleton" overcame the jagged rocks, and it is for this contribution that history tends to credit Baker, rather than Ewing, for the theory of plastic design.

9.4.2 COMPARISON BETWEEN EXPERIMENTAL AND PREDICTED RESISTANCE'S

The theoretical moment capacity of each of the test specimens has been calculated using their material and geometric properties (listed in Table 9. 3 and Table 9. 4). The resulting moment capacity ($M_{pl.Rd}$) has been compared with the experimental moment capacity (M), so that the accuracy of the resistance function can be determined.

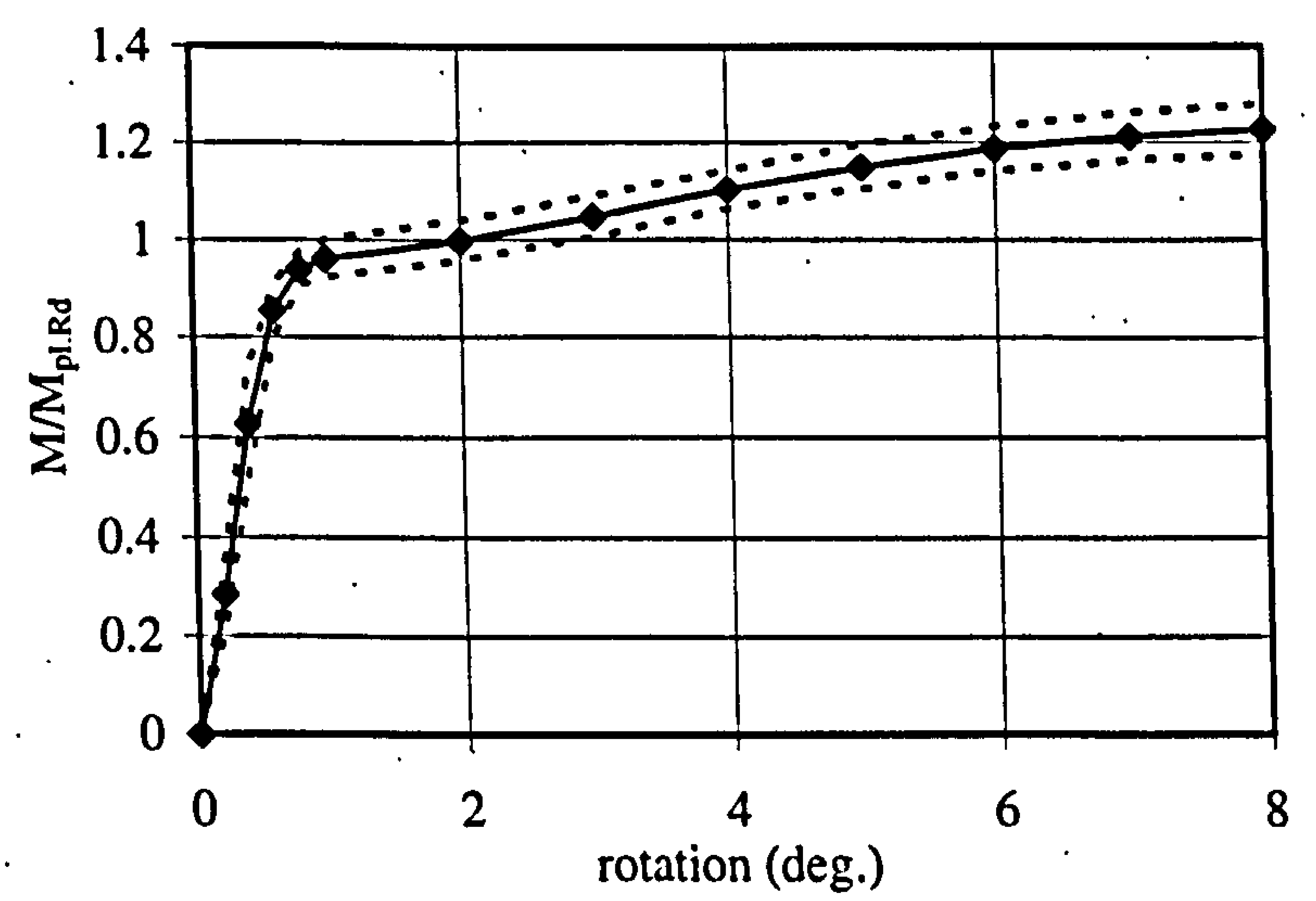


Fig. 9. 16: $M/M_{pl.Rd}$ vs. end rotation, showing the mean of the 12 tested sections, together with the upper and lower 95% confidence limits, where M is the maximum moment achieved by the given rotation

Fig. 9. 16 shows the graph of normalised experimental moment capacity vs. end rotation. This graph illustrates the amount of rotation necessary to achieve and exceed the theoretical moment capacity of the specimens. In comparison with the partially restrained beam tests (Chapter 7), the maximum load capacity was not reached until an end rotation of at least 8 degrees.

In accordance with the method used in Chapter 7, a data cut-off point of 6 degrees end rotation has been applied to the calibration of γ_R^*. Moments achieved at a rotation above 6 degrees will not be included in the calibration exercise. This is a conservative limit on the amount of end rotation acceptable. A less cautious approach will yield lower values for γ_R^*.

Test numbers	Section type	No. of tests	$\bar{b}$	σ_b
Y1 to Y6	203x102x23UB	6	1.193	0.034
Z1 to Z6	152x152x30UC	6	1.183	0.021
all		12	1.188	0.027

Table 9. 5: Statistical data obtained from the bending tests based on the maximum load achieved by test specimens up to 6 degrees of end rotation.

9.4.3 THE CALIBRATION OF γ_R^*

Listed below are the statistical parameters used to calculate γ_R^*. These measures of material variability are determined from the analysis reported in Chapter 6. The factors $\bar{b}$ and σ_b are taken from a comparison between $M_{pl.Rd}$ and the maximum experimental moment of resistance achieved by test specimens with up to 6 degrees of end rotation (see Table 9. 5).

V_{fy} = 0.05

$V_{Wpl.y}$ = 0.02

$\bar{b}$ = 1.188

σ_b = 0.027

Since this work is of a purely investigative nature, the effect of sample size has been omitted from this analysis. A calibration exercise should take account of uncertainty due to small sample size before a value for γ_R^* can be recommended for Eurocode 3. Unfortunately, with a sample size as low as 12, γ_R^* will be more influenced by the sample size than by any other factor. This factor has therefore been omitted. The calculations used to determine γ_R^* are listed as follows, the calibration method is identical to that used in Chapter 7.

$$V_b = \frac{\sigma_b}{\bar{b}} = \frac{0.027}{1.188} = 0.023 \qquad (9.2)$$

$$v_R = \sqrt{0.023^2 + 0.05^2 + 0.02^2} = 0.058 \qquad (9.3)$$

$$\gamma_R^* = \frac{\exp(0.5x0.058^2 + 3.04x0.058)}{1.16x1.188} = 0.87 \qquad (9.4)$$

According to this analysis γ_R^* can be reduced to 0.87, whilst still achieving the target reliability specified by CEN. Fig. 9. 17 shows the relationship between the maximum allowable end rotation and the corresponding value for γ_R^*. If the maximum end rotation is extended to 8 degrees, then γ_R^* can be reduced to 0.84.

The UK NAD sets γ_{m0}=1.05 and the EC3 boxed value is γ_{m0}=1.10. Whilst it is unlikely that γ_R^* will ever be reduced below 1.0, this analysis does illustrate the extreme conservatism of the current γ_{m0}-factors applied to this resistance function. In comparison with the partially restrained beam tests reported in chapter 7 there is an increased degree of reliability, as would be expected, but the differences in γ_R^* are only significant if the maximum allowable end rotation is extended to 8 degrees.

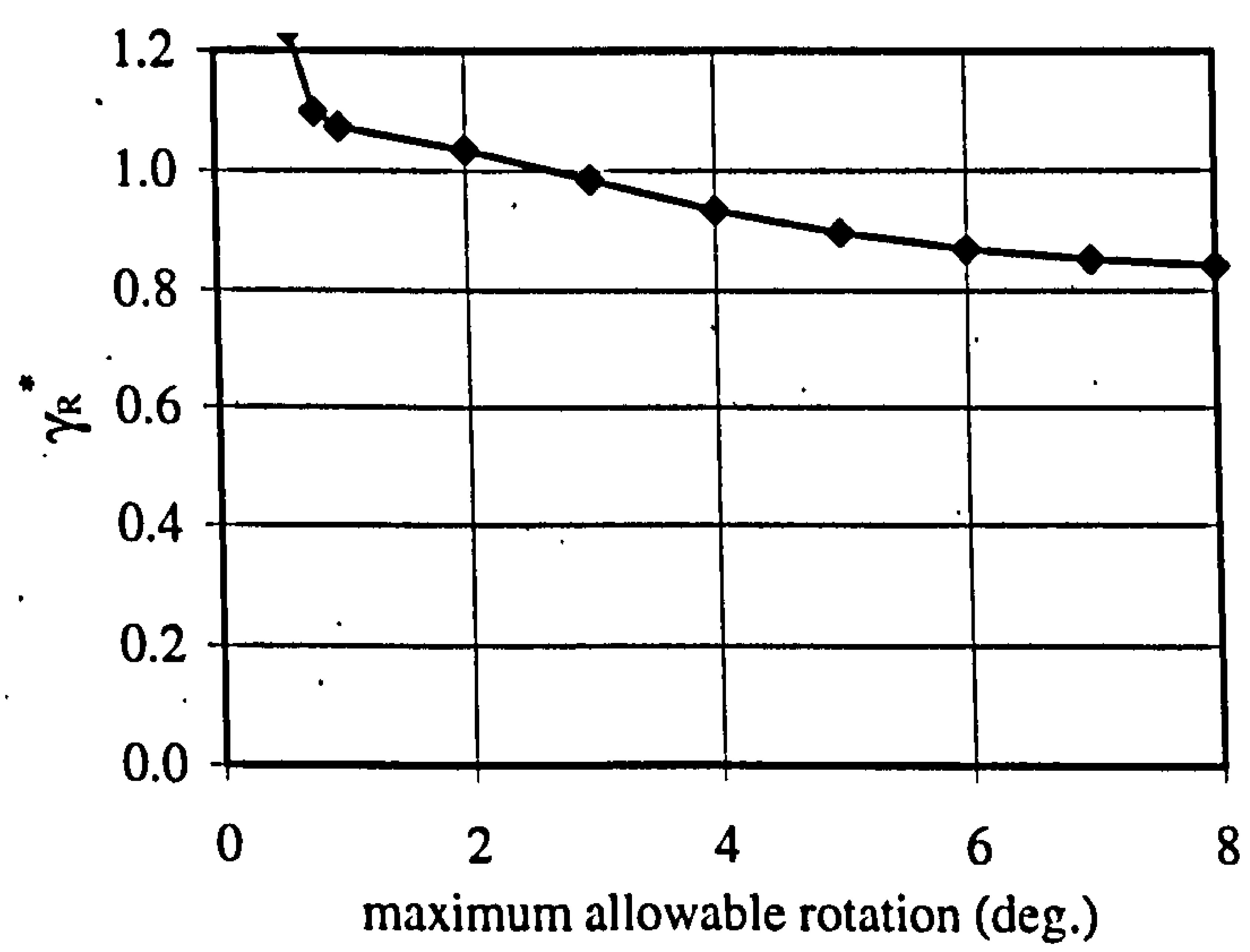


Fig. 9. 17: The relationship between the maximum allowable end rotation and the corresponding value of γ_R^*

9.5 AN ALTERNATIVE RESISTANCE FUNCTION

The preceding analysis has shown that the $M_{pl.Rd}$ resistance function is unable to fully utilise the plastic moment capacity of restrained beams. This leads to the possibility of introducing a new design technique, that fully utilises the bending strength of restrained beams. An alternative technique may prove attractive to the steel construction industry since the plastic moment of resistance is the critical design expression for a large proportion of steel used in construction. This study proposes a possible alternative, with the resulting moment capacity termed $M_{pl.u}$.

9.5.1 THE $M_{pl.u}$ DESIGN EXPRESSION

Work reported in the earlier part of the chapter used graphs of stress vs. strain taken from tensile tests to determine the distribution of stress in sections where the limit of elasticity has been exceeded. Fig. 9. 18 shows such a theoretical stress distribution, where the outer edge strain has reached 1.5%.

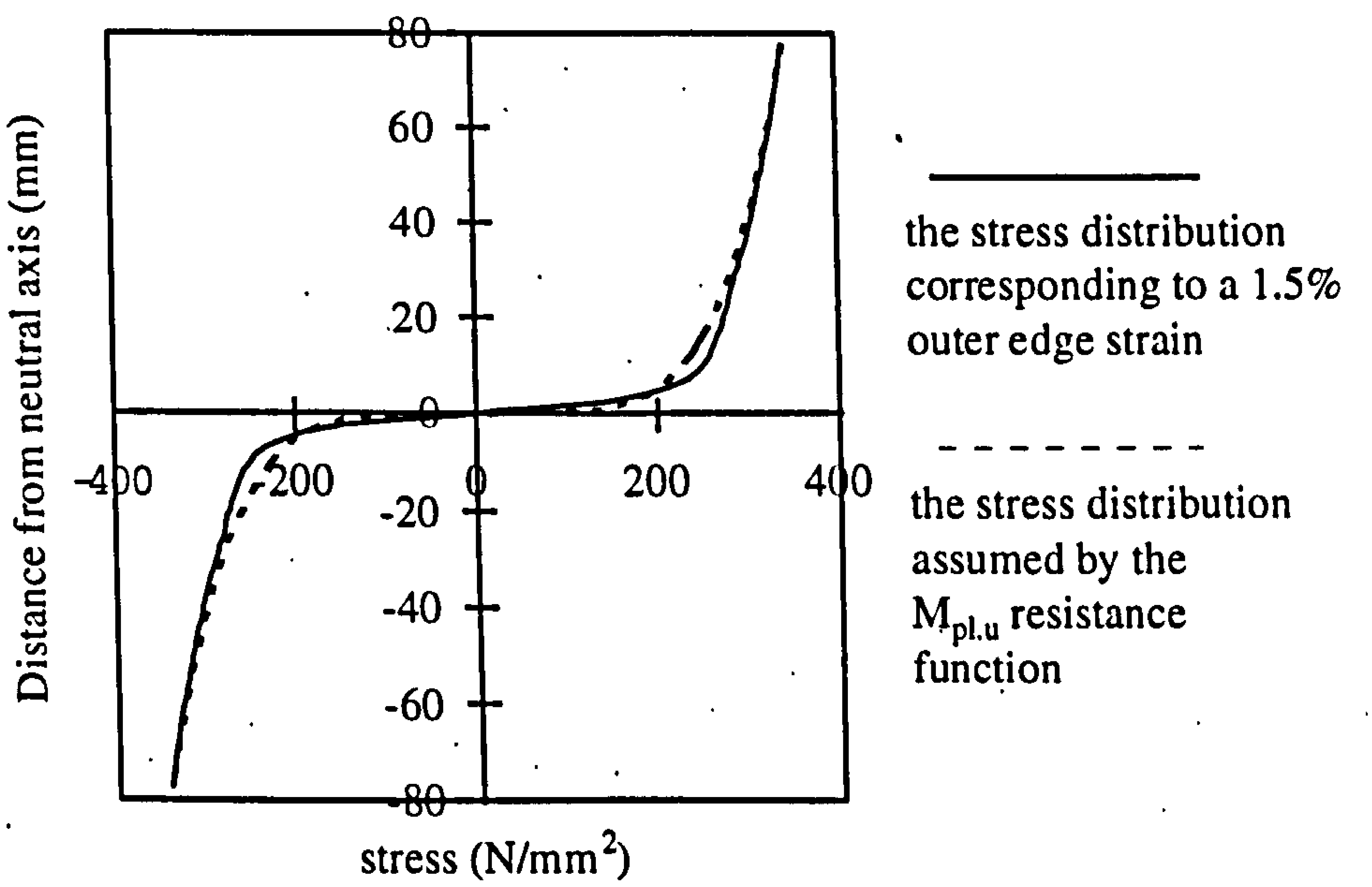


Fig. 9. 18: Comparison between the theoretical stress distribution and the distribution used in the $M_{pl.u}$ resistance function

As discussed earlier, the $M_{pl.Rd}$ formulae uses a rectangular stress block model of the stress distribution, where the yield stress is distributed uniformly throughout the depth of the plastic hinge. By comparison, the proposed technique assumes that the stress reached at the outer-edge of the section is equal to the stress corresponding to a 1.5% strain (hereafter termed $f_{1.5}$). Clearly, the optimum amount of strain to which the material strength is set may not be 1.5% but it is convenient for the purpose of illustrating the method. Below the outer-edge of the section the stress distribution used in this model is non-linear and set by equation (9. 5).

$$\sigma = x^{1/5} f_{1.5} \left(\frac{h}{2} \right)^{-1/5} \qquad (9.5)$$

where σ is the stress

and x is the distance from the neutral axis

The resulting stress distribution is compared with the theoretical stress distribution in Fig. 9. 18. Alternative versions of equation (9. 5) are possible, such as using $x^{1/3}$. This version does however provide a particularly accurate model of the distribution of stress close to the neutral axis.

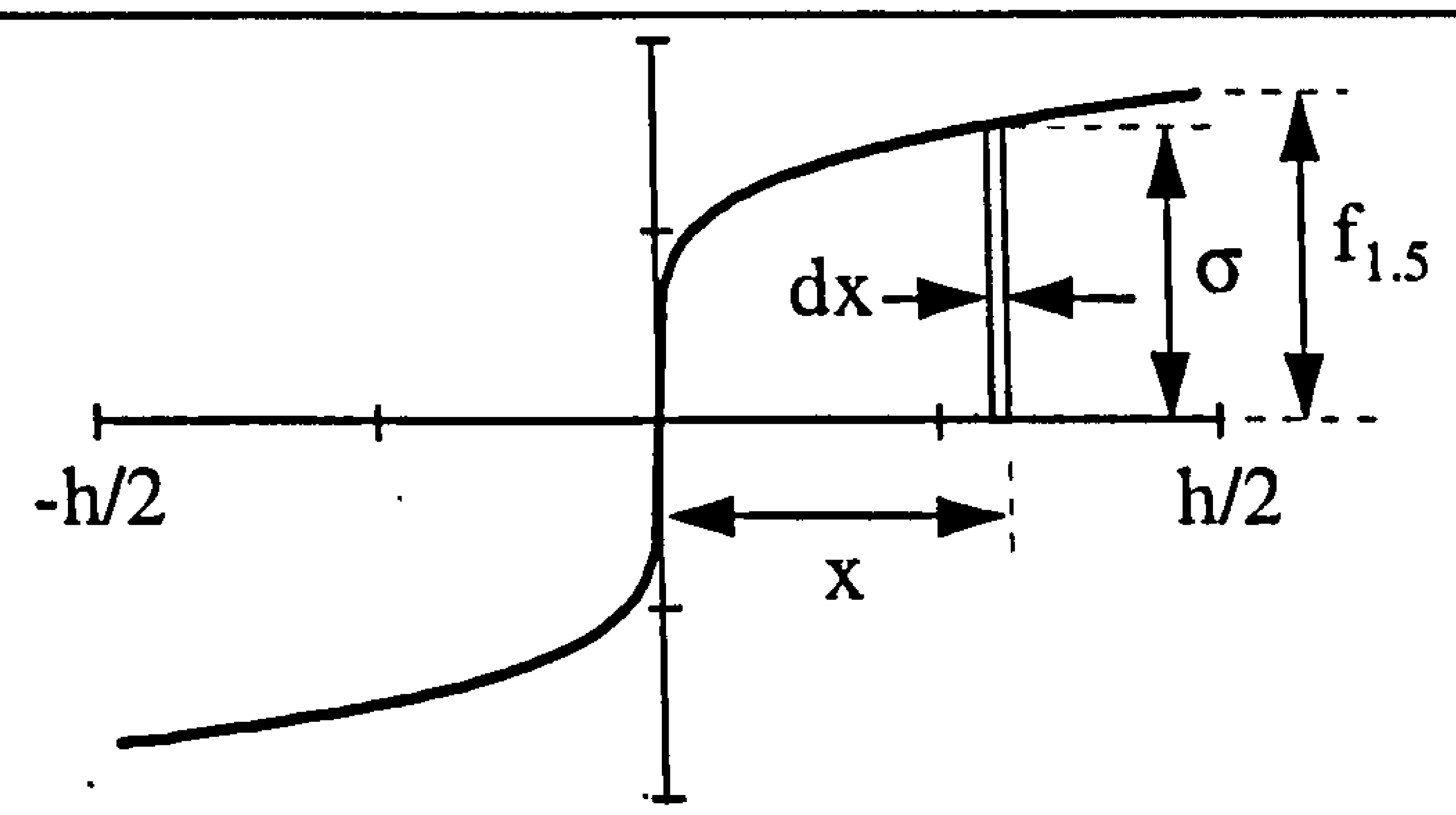


Fig. 9. 19: The distribution of stress utilised by the proposed design method

The method used to translate equation (9. 5) into a useful design expression is sketched in Fig. 9. 19. The moment of resistance $M_{pl.u}$ for a rectangular bar is given by:

$$M_{pl.u} = 2b\int_0^{h/2} x.\sigma.dx \tag{9. 6}$$

Combining (9. 5) with (9. 6) gives:

$$M_{pl.u} = 2b\int_0^{h/2} x.x^{1/5} f_{1.5}\left(\frac{h}{2}\right)^{-1/5} dx \tag{9. 7}$$

$$M_{pl.u} = 2b.\left[\frac{5}{11}x^{11/5} f_{1.5}\left(\frac{h}{2}\right)^{-1/5} + c\right]_0^{h/2} \tag{9. 8}$$

$$M_{pl.u} = \frac{5}{22} f_{1.5} bh^2 \tag{9. 9}$$

Finally, equation (9. 9) gives an expression defining $M_{pl.u}$ for a rectangular bar. For an I-section, the expression for $M_{pl.u}$ is as follows:

$$M_{pl.u} = \frac{5}{22} f_{1.5} bh^2 - 2(b - t_w) \int_0^{h/2-tf} x.x^{1/5} f_{1.5} \left(\frac{h}{2}\right)^{-1/5} dx \qquad (9.\ 10)$$

$$M_{pl.u} = \frac{5}{22} f_{1.5} bh^2 - \frac{10}{11} f_{1.5} (b - t_w) \left(\frac{h}{2} - t_f\right)^{11/5} \left(\frac{h}{2}\right)^{-1/5} \qquad (9.\ 11)$$

9.5.3 COMPARISON BETWEEN EXPERIMENTAL AND PREDICTED RESISTANCES

The normalised graph of the experimental vs. predicted resistance is sketched in Fig. 9. 20. In comparison with the analysis carried out on the $M_{pl.Rd}$ resistance function, the degree of underestimation of resistance is far less. Indeed, at the point of maximum moment of resistance, the resistance function underestimated resistance by 10%.

A statistical summary of the comparison between the predicted and experimental resistances is given in Table 9. 6. As previously, moments of resistance achieved for an end rotation greater than 6 degrees have been omitted from the comparison, since they are associated with a high degree of mid-span deflection.

Test numbers	Section type	No. of tests	$\bar{b}$	σ_b
Y1 to Y6	203x102x23UB	6	1.104	0.036
Z1 to Z6	152x152x30UC	6	1.018	0.026
all		12	1.061	0.054

Table 9. 6: Statistical data obtained from the comparison between the predicted resistance $M_{pl.u}$ with the maximum load resisted by the test specimens up to a 6 degree end rotation.

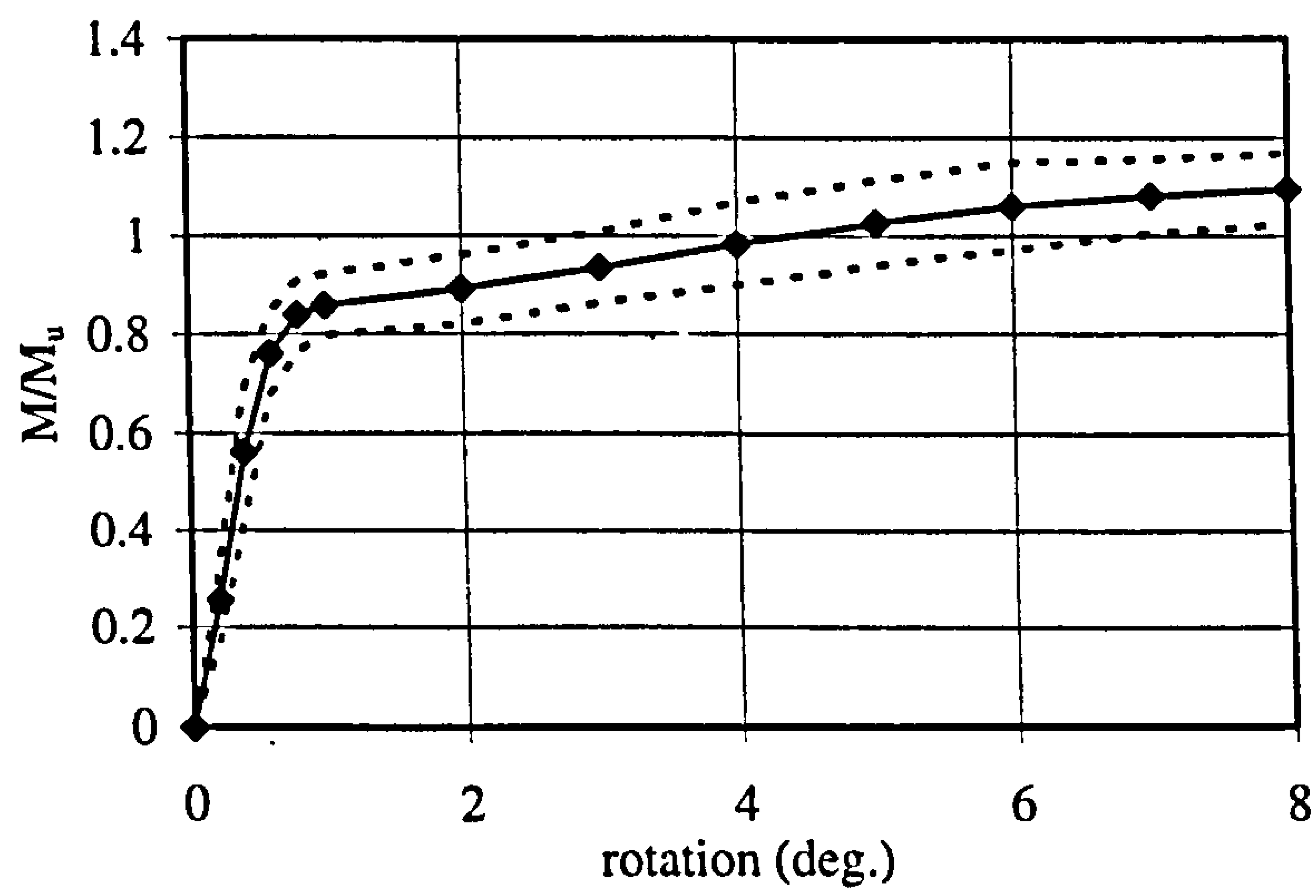


Fig. 9. 20: $M/M_{pl.u}$ vs. end rotation, showing the mean of the 12 tested sections, together with the upper and lower 95% confidence limits, where M is the maximum moment achieved by the given rotation

9.5.4 THE CALIBRATION OF γ_R^*

Listed below are the statistical parameters used to calculate γ_R^*. Values of material variability were determined from the analysis reported in Chapter 6. The factors $\bar{b}$ and σ_b are taken from Table 9. 6.

V_{fy} = 0.05

$V_{Wpl.y}$ = 0.02

$\bar{b}$ = 1.061

σ_b = 0.054

The calculations used to determine γ_R^* are listed as follows:

$$V_b = \frac{\sigma_b}{\bar{b}} = \frac{0.054}{1.061} = 0.051 \tag{9. 12}$$

$$v_r = \sqrt{0.051^2 + 0.05^2 + 0.02^2} = 0.074 \tag{9. 13}$$

$$\gamma_R^* = \frac{\exp(0.5x0.074^2 + 3.04x0.074)}{1.16x1.061} = 1.02 \tag{9. 14}$$

Therefore, using the proposed design method the γ_R^*-factor equals 1.02. Assuming that the γ_R^*-factor applied to the $M_{pl.Rd}$ resistance function is reduced to 1.0, then $(M_{pl.u}/1.02)/(M_{pl.Rd}/1.0) = 1.10$. In other words the alternative design expression offers a 10% increase to the design moment of resistance in comparison with the conventional expression.

If the maximum allowable end rotation is increased to 8 degrees, then γ_R^* applied to the $M_{pl.u}$ resistance function can be reduced to 0.97. Thus, the corresponding net increase in design moment would be 12% - a figure that is roughly equal to the efficiency gain offered by the $M_{pl.Rd}$ resistance function over the $M_{el.Rd}$ function.

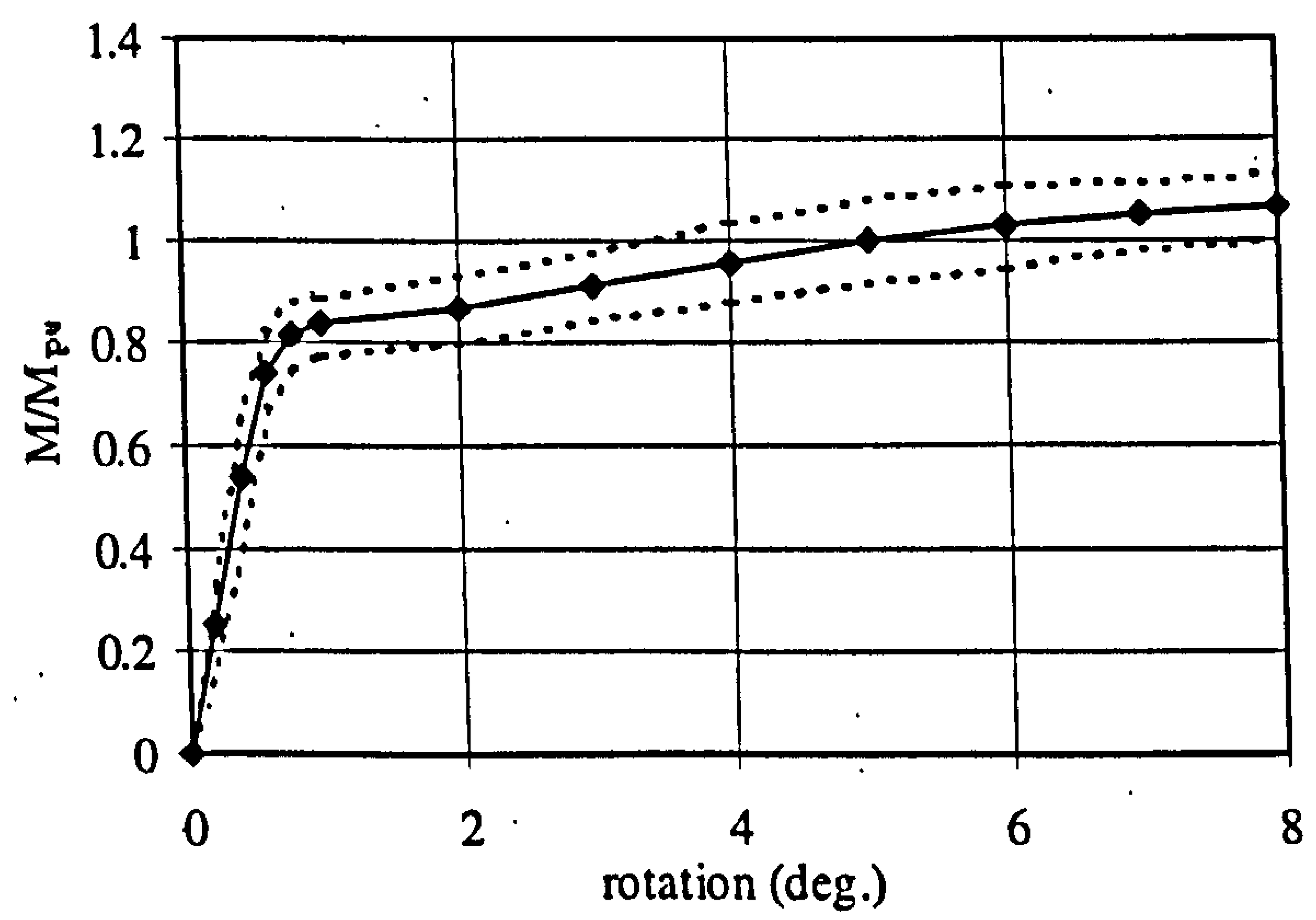


Fig. 9. 21: $M/M_{pu\,@\,1.5}$ vs. end rotation, showing the mean of the 12 tested sections, together with the upper and lower 95% confidence limits, where M is the maximum moment achieved by the given rotation

9.6 A MODIFICATION TO THE $M_{pl.Rd}$ RESISTANCE FUNCTION

A modification to the $M_{pl.Rd}$ resistance function is proposed that makes use of the classic rectangular distribution of stress approach to predicting resistance. The modified resistance function, termed M_{pu}, is identical to equation (9.1) with the exception that f_y is replaced with $f_{1.5}$. The resulting design expression (for a rectangular bar) is given as follows:

$$M_{pu}=0.25\ f_{1.5}\ .b.\ h^2 \qquad (9.\ 15)$$

9.6.1 COMPARISON BETWEEN EXPERIMENTAL AND PREDICTED RESISTANCE'S

Using the same method applied to the $M_{pl.Rd}$ and $M_{pl.u}$ resistance functions, the M_{pu} expression (for an I-section) has been compared with the experimental moment capacities of the test specimens. The results of the analysis are shown in Fig. 9. 21 and listed in Fig. 9. 7.

Test numbers	Section type	No. of tests	$\bar{b}$	σ_b
Y1 to Y6	203x102x23UB	6	1.070	0.035
Z1 to Z6	152x152x30UC	6	0.993	0.025
all		12	1.031	0.050

Table 9. 7: Statistical data obtained from a comparison between the ultimate resistance (calculated using a material strength corresponding to the 1.5% strain) with the maximum load resisted by test specimens up to a 6 degree end rotation.

9.6.2 THE CALIBRATION OF γ_R^*

Listed below are the statistical parameters used to calculate γ_R^*. The factors $\bar{b}$ and σ_b are taken from Table 9. 7.

V_{fy} = 0.05

$V_{Wpl.y}$ = 0.02

$\bar{b}$ = 1.031

σ_b = 0.050

The calculations used to determine γ_R^* are as follows:

$$V_b = \frac{\sigma_b}{\bar{b}} = \frac{0.050}{1.031} = 0.048 \qquad (9.\ 16)$$

$$v_r = \sqrt{0.048^2 + 0.05^2 + 0.02^2} = 0.072 \qquad (9.\ 17)$$

$$\gamma_R^* = \frac{\exp(0.5x0.072^2 + 3.04x0.072)}{1.16x1.031} = 1.04 \qquad (9.\ 18)$$

It follows that a γ_R^*-factor of 1.04 is sufficient to achieve the desired target reliability. The corresponding increase in design efficiency when $M_{pu}/1.04$ is compared to $M_{pl.Rd}$ /1.0 is 10%. This improved efficiency can be increased marginally by raising the maximum allowable end rotation from 6 degrees to 8 degrees.

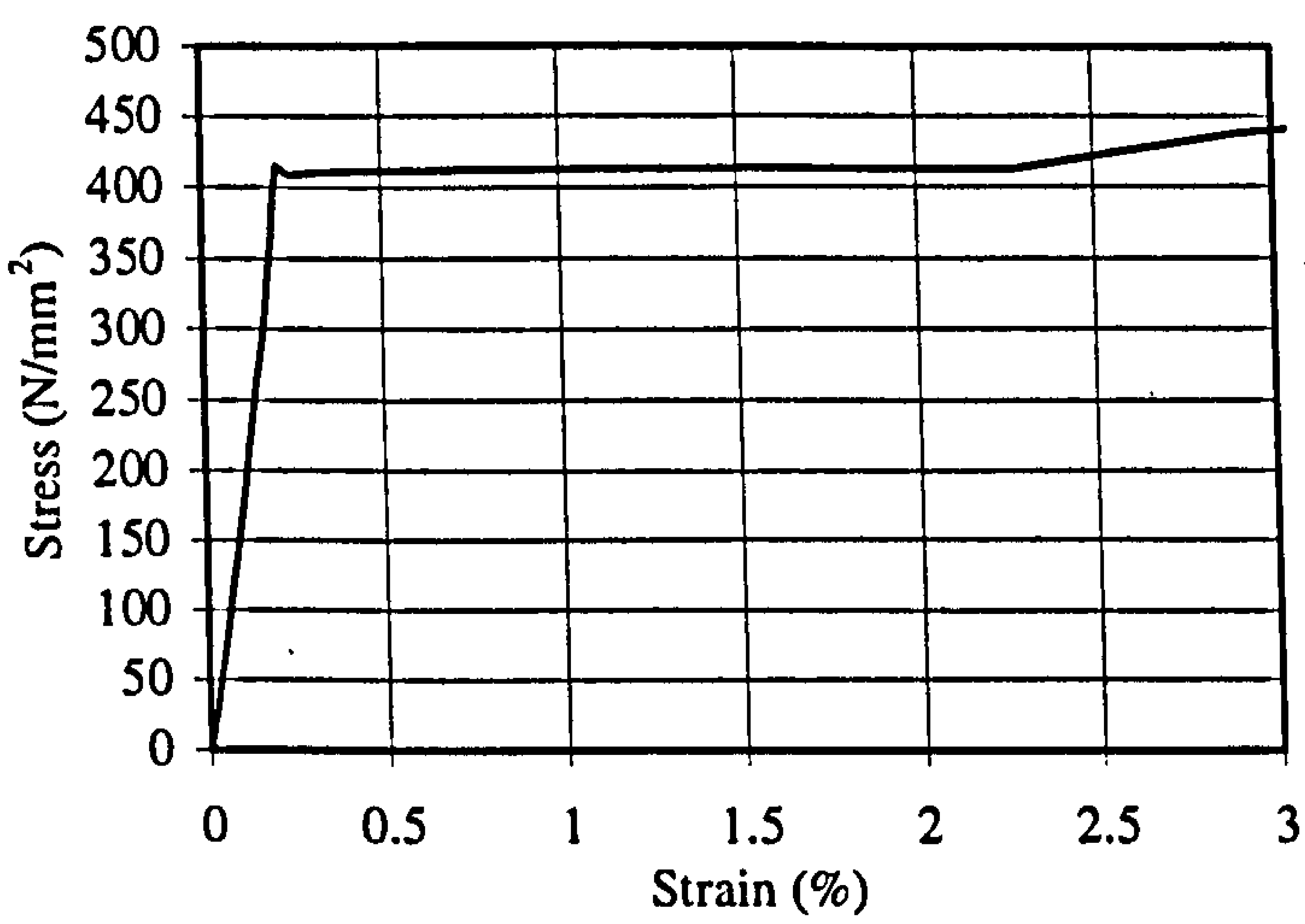


Fig. 9. 22: The classical mild steel stress vs. strain graph

9.7 CONCLUSIONS

Fig. 9. 22 shows the classic shape of the stress vs. strain graph described by (Ewing, 1899) as:

> *"...a material which is strictly elastic up to a certain limit of stress, and then so plastic that any small addition to the stress produces a relatively very large amount of strain..."*

Given these material properties, the $M_{pl.Rd}$ resistance function will provide an ideal resistance model. Unfortunately, the material properties of the test specimens from this study do not fit the classical shape for a mild steel. Instead, the material properties exhibit a shape characteristic of mild steel manufactured using the latest rolling technology. That is, immediate strain hardening after the elastic region, with no upper or lower yield points and no region *"so plastic that any small addition to the stress produces a relatively very large amount of strain..."* (see Fig. 9. 4). These material properties have adversely affected the ability of the $M_{pl.Rd}$ resistance function to model resistance, resulting in a considerable underestimation of the observed resistance.

The comparison between experimental and predicted resistances has demonstrated that an improved prediction of bending strength can be achieved by using a material strength corresponding to a 1.5% strain ($f_{1.5}$) instead of the yield stress. Specifying $f_{1.5}$ as the material strength offers an increased design resistance of approximately 10% for steels that demonstrate an early onset of strain hardening, such as those steels examined in this study. In addition this modified design approach will result in a reduction in the coefficient of variation of material strength, since the variability of f_y is greater than $f_{1.5}$ (see Fig. 9. 4). This reduced variability will translate into a slightly lower γ_R^*-factor.

Chapter Ten

CONCLUSIONS

This thesis has been concerned both with the methods used to present code information and with design reliability (according to the recently introduced Eurocode 3 (EC3): Design of steel structures - Part 1.1: General rules and rules for buildings). Within these two broad subject areas several different aspects of EC3 have been investigated. These include:

- the development of an improved format for EC3;
- the application of hypertext to codes;
- the method used for calculating partial safety factors;
- the variability of the basic material and geometric properties;
- the reliability of plate girder design;
- the reliability of restrained beam design.

The complex array of design clauses contained within EC3 are arranged on the basis of the structural phenomena to which they relate. Thus, a design engineer transferring from an existing national standard to EC3 must consider a wide range of possible failure modes in order to identify all of the clauses relevant to a particular design. An alternative structuring system has been proposed, whereby design clauses are arranged on the basis of design tasks - substantially improving the ease of use. A restructured version of EC3 (known as F-EC3) has been developed using this system; this allows the codes to become more comprehensive and user-friendly. Relevant design information is presented in a logical sequence similar to that followed during design. Although previous attempts have been made to improve the style of codes, this system is unusual in requiring no reduction of technical content. Hypertext versions of both EC3 and F-EC3 have been created on PC-based, Microsoft Windows compatible software. These contain sophisticated search and reference facilities in addition to permitting the user to transfer text to other software applications for amendment or direct use. Hypertext codes offer substantial benefits to the design engineer, both by reducing the time taken to locate suitable design clauses and during their subsequent application.

The method used for calibrating the resistance partial safety factors (γ_R-factors) contained within EC3 is known as the Annex Z method. In it the characteristic resistance is assumed to correspond to a 95% confidence limit. Since the equations defining both the design and characteristic resistances are similar, and γ_R is equal to the characteristic resistance divided by the design resistance, most of the terms cancel. This leaves a restricted expression for γ_R that is insensitive to certain key effects. Under normal conditions of use, the method will produce high values for γ_R - a result directly attributable to the assumption concerning characteristic resistance. This deficiency has been overcome by replacing γ_R with a term known as γ_R* (where γ_R* is equal to γ_R multiplied by a modification factor that reintroduces the cancelled terms). Whilst the Annex Z method finally arrives at the correct formulae for defining γ_R, it involves unnecessary assumption and is complex and difficult to apply. An alternative technique for calibrating γ_R has therefore been proposed, where design resistance is defined directly as equal to the nominal resistance divided by γ_R. Nominal resistance is not a characteristic value. Rather, it is the resistance determined using values of basic variables specified by manufacturers for design purposes. This alternative technique involves less assumption and simplifies a seemingly complex procedure.

A comprehensive set of measurements recording the material strength and geometric properties of steel were obtained and collated. This large data set (over 7000 tests) was sufficient to evaluate the type of probability distribution characterising the variability of the basic material and geometric properties of structural steel. Based on this examination, the following conclusions were drawn about the variability of the basic properties relating to steel design:

- material and geometric properties exhibit the log-normal type probability distribution;
- the nominal value of geometric properties corresponds to the manufactured mean;
- the mean value of yield stress exceeds the nominal value specified for design by an average of 16%;
- the coefficient of variation of geometric properties approximates to 0.03;
- the coefficient of variation of yield stress approximates to 0.05.

This review of the variability of steel properties provides unique information essential for an accurate quantification of steel reliability. During calibration of the γ_R-factors contained within EC3, assumptions were made concerning the variability of the basic material and geometric properties based on work originally undertaken more than 20 years ago. This review of more current data is in close agreement with these assumptions, although some of these assumptions are conservative and would benefit from modification. This study found that the nominal yield stress levels (specified in product standards referenced in the UK NAD to EC3) negate the effect of the correlation between material strength and thickness. By contrast, the nominal yield stress levels specified in the main text of EC3 are insufficiently accurate to negate the correlation, with the result that reliability is adversely affected. In the light of these findings Table 3.1 of EC3 should be amended.

Using these up-to-date measures of the variability of material and geometric properties, the theoretical shear buckling resistance of plate girders (predicted by the simple post-critical and tension field methods) was compared with experimental test results to determine reliability. Experimental resistances were not found to conform closely to their predicted resistances. The reliability of plate girder design falls well short of the target reliability and an adjustment is required to the design methods in order to ensure safe design. Plate girder reliability is dependent on web slenderness ($\bar{\lambda}_w$), and the worst case scenario occurs when $\bar{\lambda}_w > 1.2$ - at which point the probability of resistance falling below the design resistance is approximately 1 in 50 (the target probability being 1 in 845). If the reliability of plate girder design is to be improved to the level specified by CEN, then the γ_R-factor applied to girder design should be increased from 1.05 (as specified in the UK NAD to EC3) to 1.35.

A series of 4-point bending tests on partially restrained beams was carried out in order to establish the accuracy of the $M_{pl.Rd}$ resistance function. Lateral restraints were appropriately spaced to set the non-dimensional slenderness just less than 0.4 - a value that represents the worst case scenario. A comparison between the predicted and experimental resistances showed that the function underestimates the bending strength of class 1 I-section beams by an average of 16%. In addition, the experimental resistances of the test specimens proved to be highly repeatable. Calibration of γ_R using this measure of resistance function accuracy in addition to the measures of the variability of material and geometric properties discussed earlier revealed that a γ_R-factor of 0.92 is sufficient to achieve the target level of reliability set by CEN. The existing value of 1.05 produces reliability levels far in excess of those required for safe design. Unlike many other resistance functions, little test data has previously been available to compare against the $M_{pl.Rd}$ function. This study has quantified the degree of the conservatism inherent in the resistance function and provides convincing evidence of the need to reduce the γ_R-factor applied to this widely used resistance function.

An additional series of bending tests was undertaken with sufficient lateral restraints to regard the test specimens as fully restrained against lateral movement. A comparison between the experimental and predicted resistances revealed that the $M_{pl.Rd}$ function underestimated the resistance of fully restrained beams by an average of 22%. Thus, fully restrained beams are capable of resisting marginally higher loads than partially restrained beams (where the non-dimensional slenderness is < 0.4). Calibration of the $M_{pl.Rd}$ function using these results produced a γ_R-factor of 0.84. If a 6 degree limit is imposed on the maximum allowable end rotation then a γ_R-factor of 0.87 is necessary to achieve the target reliability.

These test results have led to an improved design method. The accuracy of the $M_{pl.Rd}$ resistance function can be enhanced by using a material strength that corresponds to a 1.5% strain ($f_{1.5}$). Strength predictions based on the use of the yield stress produce underestimates of resistance if the material exhibits the early onset of strain hardening. The use of $f_{1.5}$ in strength predictions will typically increase the design resistance by 10% over that calculated using the yield stress. Thus, the resulting design economies are large enough to justify a modification to the existing design procedure. Alternatively, a new design method is proposed that assumes a non-linear model of stress distribution for moment calculation.

Further work is required before this additional strength can be utilised. A suitable value for the maximum allowable rotation required to develop the full moment of resistance needs to be determined. In addition, the implications on serviceability aspects of beam design and the scope of any modified design technique need to be studied. This survey of two different resistance functions has demonstrated a considerable degree of variation in reliability levels. Further work to examine the reliability of a wider range of resistance functions is necessary in order to understand whether this variation in reliability levels is typical and if so to develop a system whereby design reliability can become independent of the design task considered.

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Appendix 1

F-EC3: A FUTURE STRUCTURE FOR EUROCODE 3

CONTENTS

1 Introduction

1.1 Scope

1.1.1 Scope of Eurocode 3

(1) Eurocode 3 applies to the design of buildings and civil engineering works in steel. It is subdivided into various separate parts, see 1.1.2 and 1.1.3.

(2) This Eurocode is only concerned with the requirements for resistance, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation are not considered.

(3) Execution[1)] is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Generally, the rules related to execution and workmanship are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works[1)] and methods of construction[1)].

(4) Eurocode 3 does not cover the special requirements of seismic design. Rules related to such requirements are provided in ENV 1998 Eurocode 8 "Design of structures for earthquake resistance"[2)] which complements or adapts the rules of Eurocode 3 specifically for this purpose.

(5) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in Eurocode 3. They are provided in ENV 1991 Eurocode 1 "Basis of design and actions on structures"[2)] which is applicable to all types of construction[1)].

1.1.2 Scope of Part 1.1 of Eurocode 3

(1) Part 1.1 of Eurocode 3 gives a general basis for the design of buildings and civil engineering works in steel.

(2) In addition, Part 1.1 gives detailed rules which are mainly applicable to ordinary buildings. The applicability of these rules may be limited, for practical reasons or due to simplifications; their use and any limits of applicability are explained in the text where necessary.

(3) The following subjects are dealt with in this initial version of Eurocode 3: Part 1.1:

- Chapter 1 : Introduction
- Chapter 2 : Performance requirements
- Chapter 3 : Materials
- Chapter 4 : Analysis of structures
- Chapter 5 : Member design
- Chapter 6 : Connection design
- Chapter 7 : Design of welds and fasteners
- Chapter 8 : Fabrication and erection
- Chapter 9 : Design assisted by testing

(4) This Part 1.1 does not cover:

- resistance to fire

- particular aspects of special types of buildings
- particular aspects of special types of civil engineering works (such as bridges, masts and towers or offshore platforms)
- cases where special measures may be necessary to limit the consequences of accidents.

1.1.3 Further Parts of Eurocode 3

(1) This Part 1.1 of Eurocode 3 will be supplemented by further Parts 2, 3 etc. which will complement or adapt it for particular aspects of special types of buildings and civil engineering works, special methods of construction and certain other aspects of design which are of general practical importance.

(2) Further Parts of Eurocode 3 which, at present, are being prepared or are planned include the following:

- Part 1.2 Fire resistance
- Part 1.3 Cold formed thin gauge members and sheeting
- Part 2 Bridges and plated structures
- Part 3 Towers, masts and chimneys
- Part 4 Tanks, silos and pipelines
- Part 5 Piling
- Part 6 Crane structures
- Part 7 Marine and maritime structures
- Part 8 Agricultural structures

1.2 Assumptions

(1) The following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

(2) The design procedures are valid only when the requirements for execution and workmanship given in Chapter 8 are also complied with.

(3) Numerical values identified by $\boxed{}$ are given as indications. Other values may be specified by Member States.

1.3 Definitions and classifications

1.3.1 Principle Rules and Application Rules

(1) Depending on the character of the individual clauses, distinction is made in this Eurocode between Principles and Application Rules.

(2) The **Principles** comprise:

- general statements and definitions for which there is no alternative, as well as
- requirements and analytical models for which no alternative is permitted unless specifically stated.

(3) The Principles are printed in roman type.

(4) The **Application Rules** are generally recognised rules which follow the Principles and satisfy their requirements.

(5) It is permissible to use alternative design rules different from the Application Rules given in the Eurocode, provided that it is shown that the alternative rule accords with the relevant Principles and is at least equivalent with regard to the resistance, serviceability and durability achieved by the structure.

(6) The Application Rules are printed in italics. This is an Application Rule.

1.3.2 Servicability limit states and ultimate limit states

(1) Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

- ultimate limit states
- serviceability limit states.

(2) Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

(3) States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states.

(4) Ultimate limit states which may require consideration include:

- *loss of equilibrium of the structure or any part of it, considered as a rigid body,*
- *failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.*

(5) Serviceability limit states correspond to states beyond which specified service criteria are no longer met.

(6) Serviceability limit states which may require consideration include:

- *deformations or deflections which adversely affect the appearance or effective use of the structure (including the proper functioning of machines or services) or cause* *damage to finishes or non-structural elements*

- *vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.*

1.3.3 Design situations

(1) Design situations are classified as:

- persistent situations corresponding to normal conditions of use of the structure
- transient situations, for example during construction or repair
- accidental situations.

1.3.4 Actions

(1) An action (F) is:

- a force (load) applied to the structure (direct action), or
- an imposed deformation (indirect action); for example, temperature effects or settlement.

(2) Actions are classified:

(i) by their variation in time:

- permanent actions (G), e.g. self-weight of structures, fittings, ancillaries and fixed equipment
- variable actions (Q), e.g. imposed loads, wind loads or snow loads
- accidental actions (A), e.g. explosions or impact from vehicles

(ii) by their spatial variation:

- fixed actions, e.g. self-weight (but see 4.1.5.3 (2) for structures very sensitive to variations in self-weight)
- free actions, which result in different arrangements of actions, e.g. movable imposed loads, wind loads, snow loads.

(3) Supplementary classifications relating to the response of the structure are given in the relevant clauses.

1.3.4.1 Characteristic values of actions

(1) Characteristic values F_k are specified:

- in ENV 1991 Eurocode 1 or other relevant loading codes, or
- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in the relevant loading standards or by the competent authority are observed.

(2) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two

characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(3) *The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.*

(4) For variable actions the characteristic value (Q_k) corresponds to either:

- the upper value with an intended probability of not being exceeded, or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
- the specified value.

(5) For accidental actions the characteristic value A_k (when relevant) generally corresponds to a specified value.

1.3.4.2 Representative values of variable actions[1]

(1) The main representative value is the characteristic value Q_k.

(2) Other representative values are related to the characteristic value Q_k by means of a factor ψ_i.

These values are defined as:

- combination value: $\psi_0 Q_k$ (see 4.1.5.2)
- frequent value: $\psi_1 Q_k$ (see 4.1.6)
- quasi-permanent value: $\psi_2 Q_k$ (see 4.1.6)

(3) Supplementary representative values are used for fatigue verification and dynamic analysis.

(4) The factors ψ_0, ψ_1 and ψ_2 are specified:

- in ENV 1991 Eurocode 1 or other relevant loading standards, or
- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in the relevant loading standards or by the competent authority are observed.

1.3.4.3 Design values of actions

(1) The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F F_k \tag{1.0}$$

where γ_F is the partial safety factor for the action considered - taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions and uncertainties in the assessment of the limit state considered.

(2) Specific examples of the use of γ_F are:

$$G_d = \gamma_G G_k$$

$$Q_d = \gamma_Q Q_k \quad \text{or} \quad \gamma_Q \psi_i Q_k$$

$$A_d = \gamma_A A_k \quad \text{(if } A_d \text{ is not directly specified)}$$

(3) The upper and lower design values of permanent actions are expressed as follows:

- where only a single characteristic value G_k is used (see 1.3.4.1(2)) then:

$$G_{d,sup} = \gamma_{G,sup} G_k$$
$$G_{d,inf} = \gamma_{G,inf} G_k$$

- where upper and lower characteristic values of permanent actions are used (see 1.3.4.1(2)) then:

$$G_{d,sup} = \gamma_{G,sup} G_{k,sup}$$
$$G_{d,inf} = \gamma_{G,inf} G_{k,inf}$$

where $G_{k,inf}$ is the lower characteristic value of the permanent action

$G_{k,sup}$ is the upper characteristic value of the permanent action

$\gamma_{G,inf}$ is the lower value of the partial safety factor for the permanent action

$\gamma_{G,sup}$ is the upper value of the partial safety factor for the permanent action

1.3.4.4 Design values of the effects of actions

(1) The effects of actions (E) are responses (for example, internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions (E_d) are determined from the design values of the actions, geometrical data and material properties when relevant:

$$E_d = E(F_d\ ,\ a_d\ ,\ ....) \qquad (1.1)$$

where a_d is defined in 1.3.5.3.

1.3.5 Material properties

1.3.5.1 Characteristic values

(1) A material property is represented by a characteristic value X_k which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

(2) In certain cases a nominal value is used as the characteristic value.

(3) Material properties for steel structures are generally represented by nominal values used as characteristic values.

(4) A material property may have two characteristic values, the upper value and the lower value. In most cases only the lower value need be considered. However, higher values of the yield strength, for example, should be considered in special cases where overstrength effects may produce a reduction in safety.

1.3.5.2 Design values

(1) The design value X_d of a material property is generally defined as:

$$X_d = X_k/\gamma_M$$

where γ_M is the partial safety factor for the material property.

(2) For steel structures, the design resistance R_d is generally determined directly from the characteristic values of the material properties and geometrical data:

$$R_d = R(X_k, a_k, ...)/\gamma_M \quad (1.2)$$

where γ_M is the partial safety factor for the resistance.

(3) The design value R_d may be determined from tests. Guidance is given in Chapter 8.

1.3.5.3 Geometrical data

(1) Geometrical data are generally represented by their nominal values:

$$a_d = a_{nom} \quad (1.3)$$

(2) In some cases the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta a \quad (1.4)$$

The values of Δa are given in the appropriate clauses.

(3) For imperfections to be adopted in the global analysis of the structure, see 4.1.4.

1.3.5.3 Load arrangements and load cases[2]

(1) A load arrangement identifies the position, magnitude and direction of a free action.

(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

1.3.6 Terms common to all Structural Eurocodes

(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

(2) The following terms are used in common for all Structural Eurocodes with the following meanings:

- **Construction works:** Everything that is constructed or results from construction operations. This term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.

- **Execution:** The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

  Note: *In English "construction" may be used instead of "execution" in certain combinations of words where there is no ambiguity (e.g. "during construction").*

- **Structure:** Organized combination of connected parts designed to provide some measure of rigidity.[4)] This term refers to load carrying parts.

- **Type of building or civil engineering works:** Type of "construction works" designating its intended purpose, e.g. dwelling house, industrial building, road bridge.

  Note: *"Type of construction works" is not used in English.*

- **Form of structure:** Structural type designating the arrangement of structural elements, e.g. beam, triangulated structure, arch, suspension bridge.

- **Construction material:** A material used in construction work, e.g. concrete, steel, timber, masonry.

- **Type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction.

- **Method of construction:** Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.

- **Structural system:** The load bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

(3) The equivalent terms in various languages are given in [EC3: table 1.1].

1.3.7 Special terms used in this Part 1.1 of Eurocode 3

(1) The following terms are used in Part 1.1 of Eurocode 3 with the following meanings:

- **Frame:** Portion of a structure, comprising an assembly of directly connected structural elements, designed to act together to resist load. This term refers to both rigid-jointed frames and triangulated frames. It covers both plane frames and three-dimensional frames.

- **Sub-frame:** A frame which forms part of a larger frame, but is treated as an isolated frame in a structural analysis.

- **Type of framing:** Terms used to distinguish between frames which are either:

  - **Semi-continuous**, in which the structural properties of the connections need explicit consideration in the global analysis.

  - **Continuous**, in which only the structural properties of the members need be considered in the global analysis.

  - **Simple**, in which the joints are not required to resist moments.

- **Global analysis:** The determination of a consistent set of internal forces and moments in a structure, which are in equilibrium with a particular set of actions on the structure.

- **System length:** Distance between two adjacent points at which a member is braced against lateral displacement in a given plane, or between one such point and the end of the member.

- **Buckling length:** System length of an otherwise similar member with pinned ends, which has the same buckling resistance as a given member.

- **Designer:** Appropriately qualified and experienced person responsible for the structural design.

1.4 S.I. units

(1) S.I. units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

- forces and loads : kN, kN/m, kN/m^2
- unit mass : kg/m^3
- unit weight : kN/m^3
- stresses and strengths : N/mm^2 (= MN/m^2 or MPa)
- moments (bending ....) : kNm.

1.5 Symbols used in Part 1.1 of Eurocode 3

1.5.1 Latin upper case letters

A	Accidental action
A	Area
B	Bolt force
C	Capacity; Fixed value; Factor
D	Damage (fatigue assessment)
E	Modulus of elasticity
E	Effect of actions
F	Action
F	Force
G	Permanent action
G	Shear modulus
H	Total horizontal load or reaction
I	Second moment of area
K	Stiffness factor (I/L)
L	Length; Span; System length
M	Moment in general
M	Bending moment
N	Axial force
Q	Variable action
R	Resistance; Reaction
S	Internal forces and moments (with subscripts d or k)
S	Stiffness (shear, rotational ... stiffness with subscripts v, j ...)
T	Torsional moment; Temperature
V	Shear force; Total vertical load or reaction
W	Section modulus
X	Value of a property of a material

1.5.2 Greek upper case letters

Δ	Difference in ... (precedes main symbol)

1.5.3 Latin lower case letters

a	Distance; Geometrical data
a	Throat thickness of a weld
a	Area ratio
b	Width; Breadth
c	Distance; Outstand
d	Diameter; Depth; Length of diagonal
e	Eccentricity; Shift of centroidal axis
e	Edge distance; End distance

f	Strength (of a material)
g	Gap; Width of a tension field
h	Height
i	Radius of gyration; Integer
k	Coefficient; Factor
l (or ℓ or L)	Length; Span; Buckling length *)
n	Ratio of normal forces or normal stresses
n	Number of ...
p	Pitch; Spacing
q	Uniformly distributed force
r	Radius; Root radius
s	Staggered pitch; Distance
t	Thickness
uu	Major axis
vv	Minor axis
xx, yy, zz	Rectangular axes

1.5.4 Greek lower case letters

α	(alpha)	Angle; Ratio; Factor
α		Coefficient of linear thermal expansion
β	(beta)	Angle; Ratio; Factor
γ	(gamma)	Partial safety factor; Ratio
δ	(delta)	Deflection; Deformation
ε	(epsilon)	Strain; Coefficient = $[235/f_y]^{0.5}$ (f_y in N/mm^2)
η	(eta)	Coefficient (in Annex E)
θ	(theta)	Angle; Slope
λ	(lambda)	Slenderness ratio; Ratio
μ	(mu)	Slip factor; Factor
ν	(nu)	Poisson's ratio
ρ	(rho)	Reduction factor; Unit mass
σ	(sigma)	Normal stress
τ	(tau)	Shear stress
ϕ	(phi)	Rotation; Slope; Ratio
χ	(chi)	Reduction factor (for buckling)
ψ	(psi)	Stress ratio; Reduction factor
ψ		Factors defining representative values of variable actions.

1.5.5 Subscripts

A	Accidental; Area
a	Average (yield strength)
a,b....	First, second .... alternative
b	Basic (yield strength)
b	Bearing; Buckling
b	Bolt; Beam; Batten
C	Capacity; Consequences
c	Cross section
c	Concrete; Column
com	Compression
cr	Critical
d	Design; Diagonal
dst	Destabilizing

E	Effect of actions (with d or k)
E	Euler
eff	Effective
e	Effective (with further subscript)
eℓ	Elastic
ext	External
f	Flange; Fastener
g	Gross
G	Permanent action
h	Height; Higher
h	Horizontal
i	Inner
inf	Inferior; Lower
i, j, k	Indices (replace by numeral)
j	Joint
k	Characteristic
ℓ	Lower
L	Long
LT	Lateral-torsional
M	Material
M	(Allowing for) bending moment
m	Bending; Mean
max	Maximum
min	Minimum
N	(Allowing for) axial force
n	Normal
net	Net
nom	Nominal
o	Hole; Initial; Outer
o	Local buckling
o	Point of zero moment
ov	Overlap
p	Plate; Pin; Packing
p	Preloading (force)
p	Partial; Punching shear
pℓ	Plastic
Q	Variable action
R	Resistance
r	Rivet; Restraint
rep	Representative
S	Internal force; Internal moment
s	Tensile stress (area)
s	Slip; Storey
s	Stiff; Stiffener
ser	Serviceability
stb	Stabilizing
sup	Superior; Upper
t (or ten*))	Tension; Tensile
t (or tor*))	Torsion
u	Major axis of cross-section
u	Ultimate (tensile strength)
ult	Ultimate (limit state)
V	(Allowing for) shear force
v	Shear; Vertical

v	Minor axis of cross-section
vec	Vectorial effects
w	Web; Weld; Warping
x	Axis along member; Extension
y	Yield
y	Axis of cross-section
z	Axis of cross-section
σ	Normal stress
τ	Shear stress
$\perp$	Perpendicular
$/\!/$	Parallel

1.5.6 Use of subscripts in Part 1.1 of Eurocode 3

(1) Strengths and properties of steel materials are nominal values, treated as characteristic values but written as below:

f_y	yield strength	[rather than f_{yk}]
f_u	ultimate strength	[rather than f_{uk}]
E	modulus of elasticity	[rather than E_k]

(2) To avoid ambiguity, subscripts are given in full in this Eurocode, but some may be omitted in practice where ambiguity is not caused by their omission.

(3) Where symbols with multiple subscripts are needed, they have been assembled in the following sequence:

- main parameter: *eg. M, N, β*
- variant type: *eg. pℓ, eff, b, c*
- sense: *eg. t, v*
- axis: *eg. y, z*
- location: *eg. 1, 2, 3*
- nature: *eg. R, S*
- level: *eg. d, k*
- index: *eg. 1, 2, 3*

(4) Dots are used to separate subscripts into pairs of characters, except as follows:

- Subscripts with more than one character are not sub-divided.
- Combinations Rd, Sd etc. are not sub-divided.

(5) Where two variant type subscripts are needed to describe a parameter, they may be separated by a comma:

eg. M, ψ

1.5.7 Conventions for member axes

(1) In general the convention for member axes is:

x-x - along the member

y-y - axis of the cross-section

z-z - axis of the cross-section

(2) For steel members, the conventions used for cross-section axes are:

- generally:

  y-y - cross-section axis parallel to the flanges

  z-z - cross-section axis perpendicular to the flanges

- for angle sections:

  y-y - axis parallel to the smaller leg

  z-z - axis perpendicular to the smaller leg

- where necessary:

  u-u - major axis (where this does not coincide with the yy axis)

  v-v - minor axis (where this does not coincide with the zz axis)

(3) The symbols used for dimensions and axes of rolled steel sections are indicated in figure 1.1.

(4) For rolled steel sections, section properties were formerly tabulated in Reference Standards with the following convention for cross-section axes:

x - cross-section axis parallel to the flanges or the smaller leg.

y - cross-section axis perpendicular to the flanges or the smaller leg.

(5) The convention used for subscripts which indicate axes for moments is:

"Use the axis about which the moment acts."

(6) For example, for an I-section a moment acting in the plane of the web is denoted M_y because it acts about the cross-section axis parallel to the flanges.

1.6 Reference standards

1.6.1 Scope

(1) This Part 1.1 of Eurocode 3 mentions 10 Reference Standards. They define the product standards and execution standards which apply to steel structures designed in accordance with Eurocode 3: Part 1.1.

1.6.2 Definitions

1.6.2.1 Reference Standard 1: "Weldable structural steel"

(1) European Standard EN 10025 'Hot rolled products of non-alloy structural steels - Technical delivery conditions' grades Fe 360, Fe 430 and Fe 510 only.

(2) European Standard prEN 10113 `Hot rolled products in weldable fine grain structural steels' grades Fe E 275 and Fe E 355 only.

(3) For prEN 10113 grades Fe E 420 and Fe E 460 refer to Annex D***).

(4) European Standard prEN 10210-1 `Hot finished steel hollow sections: Part 1 Technical delivery requirements'*).

(5) European Standard prEN 10219-1 `Cold formed structural steel hollow sections: Part 1 Non-alloy and fine grain steels'*).

(6) It shall be ensured that the weldability of the material is sufficient for the purpose for which it is required.

(7) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3: Part 1.3*).

1.6.2.2 Reference Standard 2 : "Dimensions of sections and plates"

1.6.2.2.1 Hot rolled sections, other than structural hollow sections

(1) The Euronorms for sections listed in European Standard EN 10025 modified as follows:

- excluding tolerances
- including the `corresponding national standards' for hot rolled sections listed in Annex B of EN 10025 (but excluding tolerances).

(2) European Standard EN...... `Hot rolled tapered flange and parallel flange channels - dimensions and tolerances' (when available).

(3) European Standard EN...... `Hot rolled tees - Dimensions and tolerances' (when available)

(4) European Standard EN...... `Hot rolled bulb flats - Dimensions and tolerances' (when available).

(5) European Standard EN...... 'Hot rolled I and H sections - Dimensions' (when available).

(6) European Standard EN...... 'Hot rolled split tees - Dimensions and tolerances' (when available).

(7) European Standard EN...... 'Hot rolled equal leg and unequal leg angles - Dimensions' (when available).

(8) International Standard ISO 657 'Hot rolled steel sections': Part 1 'Equal leg angles' and Part 2 'Unequal leg angles'.

(9) European Standard EN...... 'Hot rolled flat, square and round steel bars - Dimensions' (when available).

(10) European Standard EN...... 'Hot rolled square steel bars - Dimensions' (when available).

(11) European Standard EN...... 'Hot rolled round steel bars - Dimensions' (when available).

1.6.2.2.2 Hot rolled structural hollow sections

(1) European Standard prEN 10210-2 'Hot finished steel hollow sections: Part 2 Dimensions and tolerances'*).

(2) International Standard ISO 657 'Hot rolled steel sections': Part 14 'Hot finished structural hollow sections, dimensional and sectional properties', as follows:

- except that steel is to be to EN 10025

1.6.2.2.3 Cold finished structural hollow sections

(1) European Standard prEN 10219-2 'Cold formed structural steel hollow sections: Part 2 Dimensions and tolerances'*).

(2) International Standard ISO 4019 'Cold finished steel structural hollow sections - Dimensions and sectional properties'.

1.6.2.2.4 Cold formed sections, other than structural hollow sections

(1) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3: Part 1.3.

1.6.2.3 Tolerances

1.6.2.3.1 Hot rolled sections, other than structural hollow sections

(1) European Standard prEN 10034 'Structural steel I and H sections - Tolerances on shape and dimensions'*).

(2) European Standard prEN 10056 'Structural steel equal leg and unequal leg angles - Tolerances on shape and dimensions'*).

(3) European Standard EN ...... 'Hot rolled tapered flange and parallel flange channels - Dimensions and tolerances' (when available).

(4) European Standard EN ...... 'Hot rolled tees - Dimensions and tolerances' (when available).

(5) European Standard EN ...... 'Hot rolled bulb flats - Dimensions and tolerances' (when available).

(6) European Standard EN ...... 'Hot rolled split tees - Dimensions and tolerances' (when available).

(7) European Standard EN ...... 'Hot rolled square steel bars - Tolerances' (when available).

(8) European Standard EN ...... `Hot rolled round steel bars - Tolerances' (when available).

1.6.2.3.2 Structural hollow sections

(1) European Standard prEN 10210-2 `Hot finished steel hollow sections Part 2 Dimensions and tolerances'*).

(2) European Standard prEN 10219-2 `Cold formed structural steel hollow sections Part 2 Dimensions and tolerances'*).

1.6.2.3.3 Cold formed sections, other than structural hollow sections

(1) For cold formed thin gauge members and sheeting refer to prENV 1993-1-3 Eurocode 3: Part 1.3*).

1.6.2.3.4 Plates and flats

(1) European Standard EN 10029 `Tolerances on dimensions, shape and mass for hot rolled steel plates 3mm thick or above' as follows:

- Class A tolerances

(2) European Standard EN..... `Tolerance requirements for wide flats' (when available).

(3) European Standard EN..... `Tolerance requirements for flat bars' (when available).

1.6.2.4 Reference Standard 3 : "Bolts, nuts and washers"

1.6.2.4.1 Non-preloaded bolts

(1) Bolts to European Standards EN24014, EN24016, EN24017 or EN24018, nuts to EN24032, EN24034 or ISO 7413, washers to ISO 7089, ISO 7090 or ISO 7091.

(2) Bolts to International Standard ISO 7411, nuts to ISO 4775, washers to ISO 7415 or ISO 7416.

(3) Bolts to International Standard ISO 7412, nuts to ISO 7414, washers to ISO 7415 or ISO 7416.

1.6.2.4.2 Preloaded bolts

(1) Bolts to International Standard ISO 7411, nuts to ISO 4775, washers to ISO 7415 or ISO 7416.

1.6.2.5 Reference Standard 4 : "Welding Consumables"

(1) European Standard EN ..... `Welding consumables' (when available).

1.6.2.6 Reference Standard 5 : "Rivets"

(1) European Standard EN ..... `Structural steel rivets' (when available).

1.6.2.7 Reference Standards 6 to 9 : "Execution standards"

(1) European Standard EN ..... `Execution of steel structures' Part 1 `General rules and rules for buildings'.

1.6.2.8 Reference Standard 10 : "Corrosion protection"

(1) European Standard EN ..... `Corrosion protection' (when available).

2 Performance requirements

2.1 Fundamental requirements

(1) A structure shall be designed and constructed in such a way that:

- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost, and

- with appropriate degrees of reliability, it will sustain all actions and other influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

(2) It shall be verified that no relevant limit state is exceeded.

(3) All relevant design situations and load cases shall be considered.

(4) Possible deviations from the assumed directions or positions of actions shall be considered.

(5) Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

(6) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project.

2.2 Deflections

2.2.1 Requirements

(1) Steel structures and components shall be so proportioned that deflections are within the limits agreed between the client, the designer and the competent authority as being appropriate to the intended use and occupancy of the building and the nature of the materials to be supported.

(2) Recommended limits for deflections are given in 2.2.2. In some cases more stringent limits (or exceptionally, less stringent limits) will be appropriate to suit the use of the building or the characteristics of the cladding materials or to ensure the proper operation of lifts etc.

(3) The values given in 2.2.2 are empirical values. They are intended for comparison with the results of calculations and should not be interpreted as performance criteria.

(4) The design values given in 4.1.6 for the rare combination should be used in connection with all limiting values given in section 2.2.

(5) The deflections should be calculated making due allowance for any second-order effects, the rotational stiffness of any semi-rigid joints and the possible occurrence of any plastic deformations at the serviceability limit state.

2.2.2 Limiting values

(1) *The limiting values for vertical deflections given below are illustrated by reference to the simply supported beam shown in [EC3:figure 4.1], in which:*

$$\delta_{max} = \delta_1 + \delta_2 - \delta_0 \qquad (2.1)$$

where δ_{max} *is the sagging in the final state relative to the straight line joining the supports.*

δ_0 *is the pre-camber (hogging) of the beam in the unloaded state, (state 0).*

δ_1 *is the variation of the deflection of the beam due to the permanent loads immediately after loading, (state 1).*

and δ_2 *is the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load, (state 2).*

(2) *For buildings, the recommended limits for vertical deflections are given in table 2.1, in which L is the span of the beam. For cantilever beams, the length L to be considered is twice the projecting length of the cantilever.*

(3) *For crane gantry girders and runway beams, the horizontal and vertical deflections should be limited according to the use and class of the equipment.*

(4) *For buildings the recommended limits for horizontal deflections at the tops of the columns are:*

- *Portal frames without gantry cranes:* $h/150$
- *Other single storey buildings:* $h/300$
- *In a multistorey building:*
  - *In each storey* $h/300$
  - *On the structure as a whole* $h_o/500$

where h *is the height of the column or of the storey*

and h_o *is the overall height of the structure.*

2.2.3 Ponding

(1) *To ensure the correct discharge of rainwater from a flat or nearly flat roof, the design of all roofs with a slope of less than 5% should be checked to ensure that rainwater cannot collect in pools. In this check, due allowance should be made for possible construction inaccuracies and settlements of foundations, deflections of roofing materials, deflections of structural members and the effects of precamber. This also applies to floors of car parks and other open sided structures.*

(2) *Precambering of beams may reduce the likelihood of rainwater collecting in pools, provided that rainwater outlets are appropriately located.*

(3) *Where the roof slope is less than 3% additional calculations should be made to check that collapse cannot occur due to the weight of water:*

- *either collected in pools which may be formed due to the deflection of structural members or roofing material*
- *or retained by snow.*

Table 2.1 Recommended limiting values for vertical deflections

Conditions	Limits (see figure 4.1)	
	δ_{max}	δ_2
Roofs generally	L/200	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors and roofs supporting plaster or other brittle finish or non-flexible partitions	L/250	L/350
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	L/400	L/500
Where δ_{max} can impair the appearance of the building	L/250	-

2.3 Dynamic effects

2.3.1 Requirements

(1) Suitable provisions shall be made in the design for the effects of imposed loads which can induce impact, vibration, etc.

(2) *The dynamic effects to be considered at the serviceability limit state are vibration caused by machines and oscillation caused by harmonic resonance.*

(3) *The natural frequencies of structures or parts of structures should be sufficiently different from those of the excitation source to avoid resonance.*

(4) *The design values given in 4.1.6 for the frequent combination should be used in connection with all limiting values given in section 2.3.*

2.3.2 Structures open to the public

(1) The oscillation and vibration of structures on which the public can walk shall be limited to avoid significant discomfort to users.

(2) *In the case of floors over which people walk regularly, such as the floors of dwellings, offices and the like, the lowest natural frequency of the floor construction should not be lower than 3 cycles/second. This condition will be satisfied if the instantaneous total deflection $\delta_1 + \delta_2$ (as defined in 2.2.2 but calculated using the frequent combination) is less than 28mm. These limits may be relaxed where justified by high damping values.*

(3) *In the case of a floor which is jumped or danced on in a rhythmical manner, such as the floor of a gymnasium or dance hall, the lowest natural frequency of that floor should not be less than 5 cycles/second. This condition will be satisfied if the deflection calculated as above is not greater than 10mm.*

(4) *If necessary, a dynamic analysis may be carried out to show that the accelerations and frequencies which would be produced would not be such as to cause significant discomfort to users or damage to equipment.*

2.3.3 Wind - excited oscillations

(1) Unusually flexible structures, such as very slender tall buildings or very large roofs, and unusually flexible elements, such as light tie rods, shall be investigated under dynamic wind loads both for vibrations in plane and also for vibrations normal to the wind direction.

(2) *Such structures should be examined for:*

- *gust induced vibrations*
- *vortex induced vibrations*

(3) *See also ENV 1991 Eurocode 1**).

2.4 Durability

(1) To ensure an adequately durable structure, the following inter-related factors shall be considered:

- the use of the structure
- the required performance criteria
- the expected environmental conditions
- the composition, properties and performance of the materials
- the shape of members and the structural detailing
- the quality of workmanship and level of control
- the particular protective measures
- the likely maintenance during the intended life.

(2) The internal and external environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

2.5 Fire resistance

(1) For fire resistance, refer to ENV 1993-1-2 Eurocode 3: Part 1.2.

2.6 Disproportionate collapse

(1) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(2) *The potential damage should be limited or avoided by appropriate choice of one or more of the following:*

- *avoiding, eliminating or reducing the hazards which the structure is to sustain*
- *selecting a structural form which has low sensitivity to the hazards considered*
- *selecting a structural form and design that can survive adequately the accidental removal of an individual element*
- *tying the structure together*

2.7 Fatigue

2.7.1 General

2.7.1.1 Basis

(1) The aim of designing a structure against the limit state of fatigue is to ensure, with an acceptable level of probability, that its performance is satisfactory during its entire design life, such that the structure is unlikely to fail by fatigue or to require repair of damage caused by fatigue.

(2) The required safety level shall be obtained by applying the appropriate partial safety factors (see 2.7.3).

2.7.1.2 Scope

(1) Where repeated fluctuating loads are applied to a structure, its resistance to fatigue shall be checked. This Chapter presents a general method for the fatigue assessment of structures and structural elements which are subjected to repeated fluctuations of stresses

(2) For hot-rolled steelwork and for hot finished and cold-finished structural hollow sections, the requirements given in this chapter shall be satisfied.

(3) For cold-formed steelwork, the design rules given in ENV 1993-1-3 Eurocode 3: Part 1.3 cover only structures which are predominantly statically loaded. Cold-formed steelwork should not be used for structures in which fatigue predominates, unless adequate data for the fatigue assessment are available which demonstrate that the fatigue resistance is sufficient.

(4) The fatigue assessment procedures assume that the structure also conforms with the other limit state requirements of this Eurocode.

(5) The fatigue assessment procedures given in this Chapter are applicable when all structural steel materials, fasteners and welding consumables conform with the requirements specified in Chapter 3.

2.7.1.3 Necessity for fatigue assessment

(1) No fatigue assessment is normally required for building structures except as follows:

(a) Members supporting lifting appliances or rolling loads.

(b) Members subject to repeated stress cycles from vibrating machinery.

(c) Members subject to wind-induced oscillations.

(d) Members subject to crowd-induced oscillations.

(2) No fatigue assessment is required when any of the following conditions is satisfied:

(a) The largest nominal stress range $\Delta\sigma$ satisfies:

$$\gamma_{Ff}\,\Delta\sigma \leq 26/\gamma_{Mf}\ \text{N/mm}^2. \qquad (2.1)$$

(b) The total number of stress cycles N satisfies:

$$N \leq 2 \times 10^6 \left[\frac{36/\gamma_{Mf}}{\gamma_{Ff}\ \Delta\sigma_{E.2}}\right]^3 \qquad (2.2)$$

where $\Delta\sigma_{E.2}$ is the equivalent constant amplitude stress range in N/mm^2.

(c) For a detail for which a constant amplitude fatigue limit $\Delta\sigma_D$ is specified, the largest stress range (nominal or geometric as appropriate) $\Delta\sigma$ satisfies:

$$\gamma_{Ff}\,\Delta\sigma \leq \Delta\sigma_D/\gamma_{Mf} \qquad (2.3)$$

2.7.1.4 Limitations

(1) For fatigue assessment, all nominal stresses (see 2.7.1.5(7)) shall be within the elastic limits of the material. The range of the design values of such stresses shall not exceed 1,5 f_y for normal stresses or 1,5 $f_y/\sqrt{3}$ for shear stresses.

(2) The fatigue strengths specified in this Chapter are applicable to structures with suitable corrosion protection, subjected only to mildly corrosive environments, such as normal atmospheric conditions (pit depth ≤ 1 mm).

(3) The fatigue assessment procedures given in this Chapter are applicable only to structures subjected to temperatures not exceeding 150°C.

2.7.1.5 Definitions

(1) **Fatigue:** Damage in a structural part, through gradual crack propagation caused by repeated stress fluctuations.

(2) **Fatigue loading:** A set of typical load events described by the positions of loads, their intensities and their relative frequencies of occurrence.

(3) **Loading event:** A defined loading sequence applied to the structure and giving rise to a stress history.

(4) **Equivalent constant amplitude fatigue loading:** Simplified constant amplitude loading representing the fatigue effects of actual variable amplitude loading events.

(5) **Stress history:** A record, or a calculation, of the stress variation at a particular point in a structure during a load event.

(6) **Stress range:** The algebraic difference between the two extremes of a particular stress cycle forming part of a stress history. ($\Delta\sigma = \sigma_{max} - \sigma_{min}$ or $\Delta\tau = \tau_{max} - \tau_{min}$).

(7) **Nominal stress:** A stress in the parent material adjacent to a potential crack location, calculated in accordance with simple elastic strength of materials theory, excluding all stress concentration effects.

(8) **Modified nominal stress:** A nominal stress increased by an appropriate stress concentration factor, to allow for a geometric discontinuity which has not been taken into account in the classification of a particular constructional detail.

(9) **Geometric stress:** The maximum principal stress in the parent material, adjacent to the weld toe, taking into account stress concentration effects due to the overall geometry of a particular

constructional detail, but excluding local stress concentration effects due to weld geometry and discontinuities in the weld and the adjacent parent metal.

Note: The geometric stress is also known as the "hot spot stress".

(10) **'Rainflow" method and "reservoir" method:** Particular methods of producing a stress-range spectrum from a given stress history.

Note: They are two versions of the same basic method.

(11) **Stress-range spectrum:** Histogram of the frequency of occurrence for all stress ranges of different magnitudes recorded or calculated for a particular loading event.

(12) **Design spectrum:** The total of all stress-range spectra relevant to the fatigue assessment, see [EC3: figure 9.1.1].

(13) **Equivalent constant amplitude stress range:** The constant-amplitude stress range that would result in the same fatigue life as for the spectrum of variable-amplitude stress ranges, when the comparison is based on a Miner's summation.

(14) For convenience, the equivalent constant amplitude stress range may be related to a total number of 2 million variable amplitude stress range cycles.

(15) **Fatigue life:** The total number of cycles of stress variation predicted to cause fatigue failure.

(16) **Miner's summation:** A linear cumulative damage calculation based on the Palmgren-Miner rule.

(17) **Constant amplitude fatigue limit:** The limiting stress range value above which a fatigue assessment is necessary.

(18) **Detail category:** The designation given to a particular welded or bolted detail, in order to indicate which fatigue strength curve is applicable for the fatigue assessment.

(19) **Fatigue strength curve:** The quantitative relationship relating fatigue failure to stress range and number of stress cycles, used for the fatigue assessment of a category of constructional detail, see [EC3: figure 9.1.2].

(20) **Design life:** The reference period of time for which a structure is required to perform safely with an acceptable probability that failure by fatigue cracking will not occur.

(21) **Cut-off limit:** Limit below which stress ranges of the design spectrum do not contribute to the calculated cumulative damage.

2.7.1.6 Symbols

γ_{Ff}	Partial safety factor for fatigue loads.
γ_{Mf}	Partial safety factor for fatigue strength.
α_{max} , α_{min}	Maximum and minimum values of the fluctuating stresses in a stress cycle.
$\Delta\sigma$	Nominal stress range (normal stress).
$\Delta\sigma_D$	Constant amplitude fatigue limit.
$\Delta\sigma_R$	Fatigue strength (normal stress).
$\Delta\sigma_C$	Reference value of the fatigue strength at 2 million cycles (normal stress).
$\Delta\sigma_E$	Equivalent constant amplitude stress range (normal stress).
$\Delta\sigma_{E.2}$	Equivalent constant amplitude stress range (normal stress) for 2 million cycles.
$\Delta\sigma_L$	Cut-off limit.

$\Delta\tau$	Nominal stress range (shear stress).
$\Delta\tau_R$	Fatigue strength (shear stress).
$\Delta\tau_E$	Equivalent constant amplitude stress range (shear stress).
$\Delta\tau_{E.2}$	Equivalent constant amplitude stress range (shear stress) for 2 million cycles.
$\Delta\tau_C$	Reference value of the fatigue strength at 2 million cycles (shear stress).
m	Slope constant of a fatigue strength curve, with values of 3 and/or 5.
n_i	Number of cycles of stress range $\Delta\sigma_i$.
N	Number (or total number) of stress range cycles.
N_i	Number of cycles of stress range $\gamma_{Ff}\gamma_{Mf}\Delta\sigma_i$ to cause failure.
N_C	Number of cycles (2 million) at which the reference value of the fatigue strength is defined.
N_D	Number of cycles (5 million) at which the constant amplitude fatigue limit is defined.
N_L	Number of cycles (100 million) at which the cut-off limit is defined.
log	Logarithm to base 10.

2.7.2 Fatigue loading

(1) The fatigue loading shall be obtained from ENV 1991 Eurocode 1[**)] or other relevant loading standard.

(2) The loading used for the fatigue assessment shall be a characteristic value which represents the anticipated service loading throughout the required design life of the structure with a sufficient, defined, reliability.

(3) The fatigue loading may comprise different loading events which are defined by complete loading sequences of the structure, each characterised by their relative frequency of occurrence as well as their magnitude and geometrical position.

(4) Dynamic effects shall be considered when the response of the structure contributes to the modification of the design spectrum.

(5) In the absence of more accurate information, the dynamic amplification factors used for the static limit state may be employed.

(6) The effect of a loading event shall be represented by its stress history, see 2.7.1.5(5).

(7) The load models used for fatigue assessment of such structures as bridges and cranes should take into account the possible changes in use, such as growth of traffic or changes in the loading rate.

(8) Allowance should also be made for such future changes where it is necessary to base a fatigue assessment on a measured stress history.

(9) Simplified design calculations may be based on an equivalent fatigue loading, representing the fatigue effects of the full spectrum of loading events.

(10) The equivalent fatigue loading may vary with the dimensions and location of the structural element.

2.7.3 Partial safety factors

2.7.3.1 General

(1) The values of the partial safety factors to be used shall be agreed between the client, the designer and competent public authority as being appropriate, considering:

- the ease of access for inspection or repair and likely frequency of inspection and maintenance,
- the consequences of failure.

(2) Inspection may detect fatigue cracks before subsequent damage is caused. Such inspection is visual unless specified otherwise in the Project Specification.

Note: *In-service inspection is not a requirement of Eurocode 3: Part 1.1 and, if it is required, it should be subject to agreement.*

(3) In any circumstances, the possibility of general failure without any pre-warning conditions is not tolerable.

(4) Difficulties of access for inspection or repair may be such as to make the detection or the repair of cracks impractical. The client should be made aware of this so that measures to perform inspection may be taken.

2.7.3.2 Partial safety factors for fatigue loading

(1) To take account of uncertainties in the fatigue response analysis, the design stress ranges for the fatigue assessment procedure shall incorporate a partial safety factor γ_{Ff}.

(2) The partial safety factor γ_{Ff} covers the uncertainties in estimating:

- the applied load levels,
- the conversion of these loads into stresses and stress ranges,
- the equivalent constant amplitude stress range from the design stress range spectrum,
- the design life of the structure, and the evolution of the fatigue loading within the required design life of the structure.

(3) The fatigue loading given in ENV 1991 Eurocode 1) already incorporates an appropriate value of the partial safety factor γ_{Ff}.*

(4) Unless otherwise stated in subsequent Parts of Eurocode 3, or in the relevant loading standard, a value of $\gamma_{Ff} = \boxed{1,0}$ may be applied to the fatigue loading.

2.7.3.3 Partial safety factors for fatigue strength

(1) In the fatigue assessment procedure, in order to take account of uncertainties in the fatigue resistance, the design value of the fatigue strength shall be obtained by dividing by a partial safety factor γ_{Mf}.

(2) The factor γ_{Mf} covers the uncertainties of the effects of:

- the size of the detail,

- the dimensions, shape and proximity of the discontinuities,
- local stress concentrations due to welding uncertainties.
- variable welding processes and metallurgical effects.

2.7.3.4 Recommended values of γ_{Mf}

(1) The recommended values given in this clause assume that Quality Assurance procedures are applied to ensure that the fabricated constructional details comply with the relevant quality requirements for structures subjected to fatigue as defined in Reference Standard 9, see section 1.6.

(2) Concerning the consequences of failure, two possible situations may arise as follows:

- "fail-safe" structural components with reduced consequences of failure, such that the local failure of one component does not result in failure of the structure.

- non "fail-safe" structural components where local failure of one component leads rapidly to failure of the structure.

(3) Recommended values of the partial safety factor γ_{Mf} are given in table 2.7.3.1. These values should be applied to the fatigue strength.

(4) Where values of γ_{Ff} other than $\boxed{1{,}0}$ are applied to the fatigue loading, the γ_{Mf} values may need corresponding adjustment.

2.7.4 Fatigue stress spectra

2.7.4.1 Calculation of stresses

(1) Stresses shall be determined by an elastic analysis of the structure under fatigue loading. Dynamic response of the structure or impact effect shall be considered when appropriate.

2.7.4.2 Stress range in parent material

(1) Depending upon the fatigue assessment carried out, either nominal stress ranges or geometric stress ranges shall be evaluated.

(2) When determining the stress at a detail, stresses arising from joint eccentricity and imposed deformations, secondary stresses due to joint stiffness, stress redistribution due to buckling and shear lag, and the effects of prying (see Chapter 6) shall be taken into account.

2.7.4.3 Stress range for welds

(1) In load-carrying partial penetration or fillet welded joints, the forces transmitted by a unit length of weld shall be resolved into components transverse and parallel to the longitudinal axis of the weld.

(2) The fatigue stresses in the weld shall be taken as:

- a normal stress σ_w transverse to the axis of the weld
- a shear stress τ_w longitudinal to the axis of the weld.

(3) The stresses σ_w and τ_w may be obtained by dividing the relevant component of the force transmitted per unit length of weld, by the throat size a.

(4) Alternatively σ_w and τ_w may be obtained by using the method given in section 6 and taking:

$$\sigma_w = [\sigma_\perp^2 + \tau_\perp^2]^{0,5} \quad \text{and} \quad \tau_w = \tau_{/\!/} \tag{2.4}$$

Table 2.7.3.1: Partial safety factor for fatigue strength γ_{Mf}

Inspection and access	"Fail-safe" components	Non "fail-safe" components
Periodic inspection*) and maintenance. Accessible joint detail.	*1,00*	*1,25*
Periodic inspection*) and maintenance. Poor accessibility.	*1,15*	*1,35*

*) See 2.7.3.1 (2) concerning inspection.

2.7.4.4 Design stress range spectrum

(1) The stress history due to a loading event shall be reduced to a stress range spectrum by employing a soundly based method of cycle counting.

(2) For a particular detail, the total of all stress range spectra, caused by all loading events, shall be compiled to produce the design stress range spectrum to be used for the fatigue assessment.

(3) The design stress range spectrum for a typical detail or structural element may be derived from the stress history obtained by appropriate tests or by numerical evaluations based on the theory of elasticity.

(4) For many applications the "rainflow" or "reservoir" stress cycle counting methods are appropriate for use in conjunction with the Palmgren-Miner summation.

(5) Different components of a structure may have different stress range spectra.

2.7.5 Fatigue assessment procedures

2.7.5.1 General

(1) The safety verification shall be carried out either:

- in terms of cumulative damage by comparing the applied damage to the limiting damage, or
- in terms of the equivalent stress range by comparing it with the fatigue strength for a given number of stress cycles.

(2) For a particular class of constructional detail, the stresses to be considered may be normal stresses or shear stresses or both.

(3) When a constructional detail is defined in the detail classification tables [EC3: tables 9.8.1 to 9.8.7) the nominal stress range shall be used, see 2.7.5.2.

(4) The effects of geometric discontinuities which are not part of the constructional detail itself,

such as holes, cut-outs or re-entrant corners shall be taken into account separately, either by a special analysis or by the use of appropriate stress concentration factors, to determine the modified nominal stress range.

(5) When a constructional detail differs from a detail defined in the detail classification tables by the presence of a geometric discontinuity in the detail itself, the geometric stress range shall be used, see 2.7.5.3.

(6) For constructional details not included in the detail classification tables, the geometric stress range shall be used, see 2.7.5.3.

2.7.5.2 Fatigue assessment based on nominal stress ranges

2.7.5.2.1 Constant amplitude loading.

(1) For constant amplitude loading the fatigue assessment criterion is:

$$\gamma_{Ff}\, \Delta\sigma \leq \Delta\sigma_R/\gamma_{Mf} \qquad (2.5)$$

where $\Delta\sigma$ is the nominal stress range

and $\Delta\sigma_R$ is the fatigue strength for the relevant detail category (see 2.7.8) for the total number of stress cycles N during the required design life.

2.7.5.2.2 Variable amplitude loading

(1) For variable amplitude loading defined by a design spectrum, the fatigue assessment shall be based on Palmgren-Miner rule of cumulative damage.

(2) If the maximum stress range due to the variable amplitude loading is higher than the constant amplitude fatigue limit then one of the following types of fatigue assessment shall be made:

a) Cumulative damage, see (3).

b) Equivalent constant amplitude, see (7).

(3) A cumulative damage assessment may be made using:

$$D_d \leq 1 \quad \text{where} \quad D_d = \Sigma \frac{n_i}{N_i} \qquad (2.6)$$

in which n_i is the number of cycles of stress range $\Delta\sigma_i$ during the required design life

N_i is the number of cycles of stress range $\gamma_{Ff}\gamma_{Mf}\Delta\sigma_i$ to cause failure, for the relevant detail category, see 2.7.8.

(4) Cumulative damage calculations shall be based on one of the following:

a) a fatigue strength curve with a single slope constant m = 3,

b) a fatigue strength curve with double slope constants (m = 3 and m = 5), changing at the constant amplitude fatigue limit.

c) a fatigue strength curve with double slope constants (m = 3 and m = 5), and a cut-off limit at N = 100 million cycles,

d) in the case described in 2.7.6.2.2.(2), a fatigue strength curve with a single slope constant m = 5 and a cut-off limit at N = 100 million cycles.

(5) Case (c) is the most general. Stress ranges below the cut-off limit may be neglected.

(6) When using case (c) with a constant amplitude fatigue limit $\Delta\sigma_D$ at 5 million cycles, N_i may be

calculated as follows:

- *if* $\gamma_{Ff}\,\Delta\sigma_i \geq \Delta\sigma_D\,/\,\gamma_{Mf}$:

$$N_i \;=\; 5 \times 10^6 \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff}\Delta\sigma_i}\right]^3 \tag{2.7}$$

- *if* $\Delta\sigma_D\,/\,\gamma_{Mf} > \gamma_{Ff}\,\Delta\sigma_i \geq \Delta\sigma_L/\gamma_{Mf}$:

$$N_i \;=\; 5 \times 10^6 \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff}\Delta\sigma_i}\right]^5 \tag{2.8}$$

- *if* $\gamma_{Ff}\,\Delta\sigma_i < \Delta\sigma_L\,/\,\gamma_{Mf}$:

$$N_i \;=\; \infty \tag{2.9}$$

(7) An equivalent constant amplitude fatigue assessment may be made by checking the criterion:

$$\gamma_{Ff}\,\Delta\sigma_E \leq \Delta\sigma_R/\gamma_{Mf} \tag{2.10}$$

where $\Delta\sigma_E$ is the equivalent constant amplitude stress range which, for the given number of cycles, leads to the same cumulative damage as the design spectrum.

and $\Delta\sigma_R$ is the fatigue strength for the relevant detail category (see 2.7.8), for the same number of cycles as used to determine $\Delta\sigma_E$.

(8) A conservative assumption may be adopted in evaluating $\Delta\sigma_E$ and $\Delta\sigma_R$ by using a fatigue strength curve of unique slope constant m = 3.

(9) More generally, $\Delta\sigma_E$ may be calculated taking into account the double slope fatigue strength curve and the cut-off limit, as defined in [EC3: figure 9.1.2].

(10) Alternatively, an equivalent constant amplitude fatigue assessment may be made by checking the specific criterion:

$$\gamma_{Ff}\,\Delta\sigma_{E.2} \leq \Delta\sigma_C\,/\,\gamma_{Mf} \tag{2.11}$$

where $\Delta\sigma_{E.2}$ is the equivalent constant amplitude stress range for 2 million cycles, and

$\Delta\sigma_C$ is the reference value of the fatigue strength at 2 million cycles for the relevant detail category, see 2.7.8.

2.7.5.2.3 Shear stress ranges

(1) Nominal shear stress ranges, $\Delta\tau$, shall be treated similarly to nominal normal stress ranges, but using a single slope constant m = 5.

(2) For shear stresses, N_i may be calculated as follows:

- *if* $\gamma_{Ff}\,\Delta\tau_i \geq \Delta\tau_L\,/\gamma_{Mf}$:

$$N_i = 2 \times 10^6 \left[\frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff}\,\Delta\tau_i}\right]^5 \tag{2.12}$$

- *if* $\gamma_{Ff}\,\Delta\tau_i < \Delta\tau_L\,/\gamma_{Mf}$:

$$N_i = \infty \tag{2.13}$$

2.7.5.2.4 Combination of normal and shear stress ranges.

(1) In the case of a combination of normal and shear stresses the fatigue assessment shall consider their combined effects.

(2) If the equivalent nominal shear stress range is less than 15% of the equivalent nominal normal stress range, the effects of the shear stress range may be neglected.

(3) At locations other than weld throats, if the normal and shear stresses induced by the same loading event vary simultaneously, or if the plane of the maximum principal stress does not change significantly in the course of a loading event, the maximum principal stress range may be used.

(4) If, at the same location, normal and shear stresses vary independently, the components of damage for normal and shear stresses shall be assessed separately using the Palmgren-Miner rule, then combined using the criterion:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \qquad (2.14)$$

in which $D_{d.\sigma} = \Sigma(n_i/N_i)$ for normal stress ranges $\Delta\sigma_i$

and $D_{d.\tau} = \Sigma(n_i/N_i)$ for shear stress ranges $\Delta\tau_i$

(5) When using equivalent constant amplitude stress ranges, this criterion generally becomes:

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_E}{\Delta\sigma_R/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_E}{\Delta\tau_R/\gamma_{Mf}}\right]^5 \leq 1 \qquad (2.15)$$

(6) Alternatively, an equivalent constant amplitude fatigue assessment may be made using the specific criterion:

$$\left[\frac{\gamma_{Ff}\ \Delta\sigma_{E.2}}{\Delta\sigma_C/\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\ \Delta\tau_{E.2}}{\Delta\tau_C/\gamma_{Mf}}\right]^5 \leq 1 \qquad (2.16)$$

(7) Stress ranges in welds shall be determined as specified in 2.7.4.3. The components of damage for normal and shear stresses shall be assessed separately using the Palmgren-Miner rule, then combined using the criterion:

$$D_{d.\sigma} + D_{d.\tau} \leq 1 \qquad (2.17)$$

in which $D_{d.\sigma} = \Sigma(n_i/N_i)$ for stress ranges of the normal stress σ_w defined in 2.7.4.3.

and $D_{d.\tau} = \Sigma(n_i/N_i)$ for stress ranges of the shear stress τ_w defined in 2.7.4.3.

2.7.5.3 Fatigue assessments based on geometric stress ranges

(1) The geometric stress is the maximum principal stress in the parent material adjacent to the weld toe taking into account only the overall geometry of the joint, excluding local stress concentration effects due to the weld geometry and discontinuities at the weld toe.

(2) The maximum value of the geometric stress range shall be found, investigating various locations at the weld toe around the welded joint or the stress concentration area.

(3) The geometric stresses may be determined using stress concentration factors obtained from parametric formulae within their domains of validity, a finite element analysis or an experimental model.

(4) A fatigue assessment based on the geometric stress range, shall be treated similarly to the assessments given in 2.7.5.2, but replacing the nominal stress range by the geometric stress

range.

(5) The fatigue strength to be used in assessments based on geometric stress ranges shall be determined by reference to 2.7.6.3.

2.7.6 Fatigue strength

2.7.6.1 General

(1) The fatigue strength is defined for normal stresses by a series of log $\Delta\sigma_R$ - log N curves, each applying to a typical detail category. Each detail category is designated by a number which represents, in N/mm², the reference value $\Delta\sigma_C$ of the fatigue strength at 2 million cycles, see figure 2.7.6.1. The values used are rounded values, corresponding to the detail categories given in table 2.7.6.1.

(2) The fatigue strength curves for nominal normal stresses are defined by:

$$\log N = \log a - m \log \Delta\sigma_R \qquad (2.18)$$

where

$\Delta\sigma_R$ is the fatigue strength

N is the number of stress range cycles

m is the slope constant of the fatigue strength curves, with values of 3 and/or 5.

log a is a constant which depends on the related part of the slope, see 2.7.6.2.1.

(3) Similar fatigue strength curves are used for shear stresses, see figure 2.7.6.2 and table 2.7.6.2.

(4) The curves are based on representative experimental investigations and thus include the effects of:

- *local stress concentrations due to the weld geometry,*
- *size and shape of acceptable discontinuities,*
- *the stress direction,*
- *residual stresses,*
- *metallurgical conditions,*
- *in some cases, the welding process and post-weld improvement procedures.*

(5) When test data are used to determine the appropriate detail category for a particular constructional detail, the value of the stress range $\Delta\sigma_R$ corresponding to a value of N of 2 million cycles shall be calculated for a 75% confidence interval of 95% probability of survival for log N, taking into account the standard deviation and the sample size. The number of data points (not lower than 10) shall be considered in the statistical analysis.

(6) Proper account shall be taken of the fact that residual stresses are low in small scale samples. The resulting fatigue strength curve shall be corrected to allow for the greater effect of residual stresses in full scale structures.

(7) The level of acceptable discontinuities are defined in Reference Standard 9, see section 1.6.

(8) Separate fatigue strength curve definitions are given for:

- Classified details, for which the nominal stress range procedure applies, see 2.7.6.2.
- Non-classified details, for which the geometrical stress range procedure applies, see 2.7.6.3.

Table 2.7.6.1 Numerical values for fatigue strength curves for normal stress ranges.

Detail category $\Delta\sigma_C$ (N/mm²)	log a for N < 10^8		Stress range at constant amplitude fatigue limit	Stress range at cut-off limit
	$N \leq 5 \times 10^6$ (m = 3)	$N \geq 5 \times 10^6$ (m = 5)	($N = 5 \times 10^6$) $\Delta\sigma_D$ (N/mm²)	($N = 10^8$) $\Delta\sigma_L$ (N/mm²)
160	12,901	17,036	117	64
140	12,751	16,786	104	57
125	12,601	16,536	93	51
112	12,451	16,286	83	45
100	12,301	16,036	74	40
90	12,151	15,786	66	36
80	12,001	15,536	59	32
71	11,851	15,286	52	29
63	11,701	15,036	46	26
56	11,551	14,786	41	23
50	11,401	14,536	37	20
45	11,251	14,286	33	18
40	11,101	14,036	29	16
36	10,951	13,786	26	14

2.7.6.2 Fatigue strength curves for classified details

2.7.6.2.1 Fatigue strength curves for open sections

(1) The detail categories to be used for various typical constructional details for open sections are given in 5 tables as follows:

[EC3: Table 9.8.1]:	Non-welded details.
[EC3: Table 9.8.2]:	Welded built-up sections.
[EC3: Table 9.8.3]::	Transverse butt welds.
[EC3: Table 9.8.4]:	Welded attachments with non-load carrying welds.
[EC3: Table 9.8.5]:	Welded joints with load-carrying welds.

(2) In [EC3: Table 9.8.1] onwards, the arrows in the diagrams indicate the location and direction of the stresses to which the relevant fatigue strengths apply.

(3) The detail category used to designate a particular fatigue strength curve corresponds to the reference value (in N/mm²) of the fatigue strength at 2 million cycles, $\Delta\sigma_C$ or $\Delta\tau_C$ as appropriate.

(4) Fatigue strength curves for nominal normal stress ranges for a number of typical detail categories are given in [EC3: figure 9.6.1]. The constant amplitude fatigue limit corresponds to the fatigue strength for 5 million cycles and the cut-off limit corresponds to the fatigue strength for 100 million cycles.

(5) The corresponding values for calculating the fatigue strength are given in table 2.7.6.1.(6) Fatigue strength curves for nominal shear stress ranges are given in [EC3: figure 9.6.2]. They have a single slope constant of m = 5. There is no constant amplitude fatigue limit for these curves but the cut-off limit at 100 million cycles applies as for nominal normal

stress ranges.

(7) The corresponding values for calculating the fatigue strength are given in table 2.7.6.2.

(8) Detail category 100 is for parent metal, full penetration butt welds and for bearing type fitted bolts in shear.

(9) Detail category 80 is for fillet welds and for partial penetration butt welds in shear.

Table 2.7.6.2 Numerical values for fatigue strength curves for shear stress ranges

Detail category $\Delta\tau_C$ (N/mm^2)	log a for $N < 10^8$ ($m = 5$)	Stress range at cut-off limit ($N = 10^8$) $\Delta\tau_L$ (N/mm^2)
100	16,301	46
80	15,801	36

2.7.6.2.2 Fatigue strength curves for hollow sections

(1) The fatigue strength curves to be used in conjunction with the hollow section details shown in [EC3: table 9.8.6], are those given in [EC3: figure 9.6.1]. They have double slope constants of m = 3 and m = 5.

(2) The fatigue strength curves to be used in conjunction with the hollow section joint details for lattice girders shown in [EC3: table 9.8.7], are given in [EC3: figure 9.6.3]. They have a single slope constant of m = 5.

(3) The corresponding values for numerical calculations of the fatigue strength are given in table 2.7.6.3.

(4) The throat thickness of a fillet weld shall not be less than the wall thickness of the hollow section member which it connects.

(5) The member forces may be analysed neglecting the effect of eccentricities and joint stiffness, assuming hinged connections, provided that the effects of secondary bending moments on stress ranges are considered.

(6) In the absence of rigorous stress analysis and modelling of the joint, the effects of secondary bending moments may be taken into account by multiplying the stress ranges due to axial member forces by appropriate coefficients as follows:

- for joints in lattice girders made from circular hollow sections, see table 2.7.6.4.
- for joints in lattice girders made from rectangular hollow sections, see table 2.7.6.5.

(7) For clarification of the terminology used in tables 2.7.6.4 and 2.7.6.5, see [EC3: table 9.8.7].

Table 2.7.6.3 Numerical values for fatigue strength curves for hollow sections		
Detail category $\Delta\sigma_C$ (N/mm²)	log a for N < 10^8 (m = 5)	Stress range at cut-off limit (N = 10^8) $\Delta\sigma_L$ (N/mm²)
90	16,051	41
71	15,551	32
56	15,051	26
50	14,801	23
45	14,551	20
36	14,051	16

Table 2.7.6.4 Coefficients to account for secondary bending moments in joints of lattice girders made from circular hollow sections

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,3
	N type	1,5	1,8	1,4
Overlap joints	K type	1,5	1,0	1,2
	N type	1,5	1,65	1,25

2.7.6.3 Fatigue strength curves for non-classified details

(1) The fatigue assessment of all constructional details not included in [EC3: tables 9.8.1 to 9.8.7] and of all hollow section members and tubular joints with wall thicknesses greater than 12,5 mm, shall be carried out using the procedure based on geometric stress ranges, given in 2.7.5.3.

(2) The fatigue strength curves to be used for fatigue assessments based on geometric stress ranges, shall be:

a) For full penetration butt welds:

- Category 90, in [EC3: figure 9.6.1], when both weld profile and permitted weld defects acceptance criteria are satisfied.
- Category 71, in [EC3: figure 9.6.1], when only permitted weld defects acceptance criteria are satisfied.

b) For load carrying partial penetration butt welds and fillet welds:

- Category 36, in [EC3: figure 9.6.1], or alternatively a fatigue strength curve obtained from adequate fatigue test results.

(3) For stress ranges in welds see 2.7.4.3.

Table 2.7.6.5 Coefficients to account for secondary bending moments in joints of lattice girders made from rectangular hollow sections

Type of joint		Chords	Verticals	Diagonals
Gap joints	K type	1,5	1,0	1,5
	N type	1,5	2,2	1,6
Overlap joints	K type	1,5	1,0	1,3
	N type	1,5	2,0	1,4

2.7.7 Fatigue strength modifications

2.7.7.1 Stress range in non-welded or stress relieved details

(1) In non-welded details or stress relieved welded details, the effective stress range to be used in the fatigue assessment shall be determined by adding the tensile portion of the stress range and 60% of the compressive portion of the stress range.

2.7.7.2 Influence of thickness

(1) The fatigue strength depends on the thickness of the parent metal in which a potential crack may initiate and propagate.

(2) The variation of fatigue strength with thickness shall be taken into account for material thicknesses greater than 25mm by reducing the fatigue strength using:

$$\Delta\sigma_{R,t} = \Delta\sigma_R \, (25/t)^{0,25} \qquad (2.19)$$

with $t > 25$ mm

(3) When the material thickness of the constructional detail is less than 25mm the fatigue strength shall be taken as that for a thickness of 25mm.

(4) This reduction for thickness shall be applied only to structural details with welds transverse to the direction of the normal stresses.

(5) Where the detail category in the classification tables already varies with thickness, the above correction for thickness shall not be applied.

2.7.7.3 Modified fatigue strength curves

(1) Test data for certain details do not fit the fatigue strength curves given in [EC3: figure 9.6.1]. In order to avoid any non-conservative conditions, such details are allocated to one detail category lower than their fatigue strength at 2 million cycles would otherwise indicate.

(2) These details are identified by an asterisk in [EC3: tables 9.8.1 to 9.8.5]. The classification of

such details may be increased by one detail category in table 2.7.6.1, provided that modified fatigue strength curves are adopted in which the constant amplitude fatigue limit is taken as the fatigue strength at 10 million cycles for m = 3, see [EC3:figure 9.7.1].

(3) The numerical values necessary for calculating a modified value of fatigue strength are given in table 2.7.7.1.

Table 2.7.7.1 Numerical values for modified fatigue strength curves for normal stress ranges.

Detail category (Nominal)	log a for $N < 10^8$		Stress range at constant amplitude fatigue limit	Stress range at cut-off limit
	$N \leq 10^7$ ($m = 3$)	$N \geq 10^7$ ($m = 5$)	($N = 10^7$) $\Delta\sigma_D$ (N/mm^2)	($N = 10^8$) $\Delta\sigma_L$ (N/mm^2)
50*	11,551	14,585	33	21
45*	11,401	14,335	29	18
36*	11,101	13,835	23	15

2.7.8 Classification tables

(1) The classification of the constructional details listed in [EC3: tables 9.8.1 to 9.8.7] has been established on the basis of stresses along the direction indicated by the arrow for potential cracks on the surface of the parent metal, or for the case of weld throat cracking, on the stress calculated in the weld throat.

(2) The stresses shall be calculated using the gross or net section of the loaded member as appropriate.

3 Materials

3.1 General

(1) The material properties given in this Chapter are nominal values to be adopted as characteristic values in design calculations.

(2) Other material properties are given in the relevant Reference Standards defined in 1.6.

3.2 Structural steel

3.2.1 Scope

(1) This Part 1.1 of Eurocode 3 covers the design of structures fabricated from steel material conforming to Reference Standard 1, see 1.6.

(2) It may also be used for other structural steels, provided that adequate data exist to justify the application of the relevant design and fabrication rules. Test procedures and test evaluation shall conform with Chapters 2 and 9 of this Part 1.1 and the test requirements shall align with those required in Reference Standard 1.

(3) For high strength steel refer to normative Annex D.[*]

3.2.2 Material properties for hot rolled steel

3.2.2.1 Nominal values

(1) The nominal values of the yield strength f_y and the ultimate tensile strength f_u for hot rolled steel are given in table 3.1 for steel grades Fe 360, Fe 430 and Fe 510 in accordance with EN 10025 and steel grades Fe E 275 and Fe E 355 in accordance with prEN 10113.

(2) The nominal values in table 3.1 may be adopted as characteristic values in calculations.

(3) As an alternative, the values specified in EN 10025 and prEN 10113 for a larger range of thicknesses may be used.

(4) Similar values may be adopted for hot finished structural hollow sections.

(5) For high strength steel refer to normative Annex D.[*]

3.2.2.2 Plastic analysis

(1) Plastic analysis (see 4.1.3) may be utilised in the global analysis of structures or their elements provided that the steel complies with the following additional requirements:

- the ratio of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y satisfies:

$f_u / f_y \geq 1{,}2$

- the elongation at failure on a gauge length of $5{,}65\sqrt{A_o}$ (where A_o is the original cross section area) is not less than 15%
- the stress-strain diagram shows that the ultimate strain ε_u corresponding to the ultimate tensile strength f_u is at least 20 times the yield strain ε_y corresponding to the yield strength f_y.

(2) *The steel grades listed in table 3.1 may be accepted as satisfying these requirements.*

Table 3.1 Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel to EN 10025 or prEN 10113.

Nominal steel grade	Thickness t mm*)			
	t ≤ 40 mm		40 mm < t ≤ 100 mm**)	
	f_y (N/mm²)	f_u (N/mm²)	f_y (N/mm²)	f_u (N/mm²)
EN 10025:				
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490
prEN 10113:				
Fe E 275	275	390	255	370
Fe E 355	355	490	335	470

*) t is the nominal thickness of the element.
**) 63 mm for plates and other flat products in steels of delivery condition TM to prEN 10113-3

3.2.2.3 Fracture toughness

(1) The material shall have sufficient fracture toughness to avoid brittle fracture at the lowest service temperature expected to occur within the intended life of the structure.

(2) *In normal cases of welded or non-welded members in building structures subject to static loading or fatigue loading (but not impact loading), no further check against brittle fracture is necessary if the conditions given in table 3.2 are satisfied.*

(3) *For high strength steel refer to normative Annex D.*

(4) *For all other cases reference should be made to informative Annex C.*

3.2.3 Material properties for cold formed steel

(1) The nominal values of the yield strength and the ultimate tensile strength (to be adopted as characteristic values in calculations) for cold formed steel are specified in ENV 1993-1-3 Eurocode 3: Part 1.3*).

(2) The average yield strength of cold finished structural hollow sections shall be determined as specified in figure 5.4.2.

Table 3.2 Maximum thickness for statically loaded structural elements without reference to informative Annex C

Steel grade and quality	Maximum thickness (mm) for lowest service temperature of					
	0°C		-10°C		-20°C	
Service condition	S1	S2	S1	S2	S1	S2
EN 10025 [(1)]:						
Fe 360 B	150	41	108	30	74	22
Fe 360 C	250	110	250	75	187	53
Fe 360 D	250	250	250	212	250	150
Fe 430 B	90	26	63	19	45	14
Fe 430 C	250	63	150	45	123	33
Fe 430 D	250	150	250	127	250	84
Fe 510 B	40	12	29	9	21	6
Fe 510 C	106	29	73	21	52	16
Fe 510 D	250	73	177	52	150	38
Fe 510 DD[(2)]	250	128	250	85	250	59
prEN 10113:[(3)]						
Fe E 275 KG[(4)]	250	250	250	192	250	150
Fe E 275 KT	250	250	250	250	250	250
Fe E 355 KG[(4)]	250	128	250	85	250	59
Fe E 355 KT	250	250	250	250	250	150

Service conditions [(5)]:

S1 Either:

- non-welded, or
- in compression

S2 As welded, in tension

In both cases this table assumes loading rate R1 and consequences of failure condition C2, see informative Annex C.

Notes:

(1) For rolled sections over 100 mm thick, the minimum Charpy V-notch energy specified in EN 10025 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J at the relevant specified test temperature is required and 23 J for thicknesses over 150 mm up to 250 mm.

(2) For steel grade Fe 510 DD to EN 10025, the specified minimum Charpy V-notch energy value is 40J at -20°C. The entries in this row assume an equivalent value of 27 J at -30°C.

(3) For steels of delivery condition N to prEN 10113-2 over 150 mm thick and for steels of delivery condition TM to prEN 10113-3 over 150 mm thick for long products and over 63 mm thick for flat products, the minimum Charpy V-notch energy specified in prEN 10113 is subject to agreement.
For thicknesses up to 150 mm, a minimum value of 27 J is required and 23 J for thicknesses over 150 mm up to 250 mm. The test temperature should be -30°C for KG quality steel and -50°C for KT quality steel.

(4) For steel of quality KG to prEN 10113, the specified minimum values of Charpy V-notch energy go down to 40 J at -20°C. The entries in this row assume an equivalent value of 27 J at -30°C.

(5) For full details of service conditions, refer to informative Annex C.

3.2.4 Dimensions, mass and tolerances

(1) The dimensions and mass of all rolled steel sections, plates and structural hollow sections, and their dimensional and mass tolerances, shall conform with Reference Standard 2, see normative Annex B.

3.2.5 Design values of material coefficients

(1) The material coefficients to be adopted in calculations for the steels covered by this Eurocode shall be taken as follows:

- modulus of elasticity $E = 210\ 000\ N/mm^2$
- shear modulus $G = E/2(1+\nu)$
- Poisson's ratio $\nu = 0,3$
- coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per °C
- unit mass $\rho = 7850\ kg/m^3$

3.3 Connecting devices

3.3.1 General

(1) Connecting devices shall be suitable for their specified use.

(2) Suitable connecting devices include bolts, friction grip fasteners, rivets and welds, each to the appropriate Reference Standard, see normative Annex B.

3.3.2 Bolts, nuts and washers

3.3.2.1 General

(1) Bolts, nuts and washers shall conform with Reference Standard 3, see normative Annex B.

(2) Bolts of grades lower than 4.6 or higher than 10.9 shall not be used unless test results prove their acceptability in a particular application.

(3) The nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} (to be adopted as characteristic values in calculations) are given in table 3.3.

3.3.2.2 Preloaded bolts

(1) High strength bolts may be used as preloaded bolts with controlled tightening, if they conform with the requirements for preloaded bolts in Reference Standard 3.

(2) Other suitable types of high strength bolts may also be used as preloaded bolts with controlled tightening, when agreed between the client, the designer and the competent authority.

3.3.3 Other types of preloaded fasteners

(1) Other suitable types of high strength fasteners (such as high strength swaged fasteners) may also be used as preloaded fasteners, when agreed between the client, the designer and the competent authority, provided that they have similar mechanical properties to those required for preloaded bolts and are capable of being reliably tightened to appropriate specified initial preloads.

Table 3.3 Nominal values of yield strength f_{yb} and ultimate tensile strength f_{ub} for bolts.

Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

3.3.4 Rivets

(1) The material properties, dimensions and tolerances of steel rivets shall conform with Reference Standard 5, see normative Annex B.

3.3.5 Welding consumables

(1) All welding consumables shall conform with Reference Standard 4, see normative Annex B.

(2) The specified yield strength, ultimate tensile strength, elongation at failure and minimum Charpy V-notch energy value of the filler metal, shall all be either equal to, or better than, the corresponding values specified for the steel grade being welded.

4 Analysis of structures

4.1 General

4.1.1 Structural systems

(1) The extent of global analysis required depends on the form of structure, as follows:

(a) **Simple structural elements:**

Single-span beams and individual tension or compression members are statically determinate. Triangulated frames may be statically determinate or statically indeterminate.

(b) **Continuous beams and non-sway frames:**

Continuous beams and frames in which sway effects are negligible, or are eliminated by suitable means (see 4.4.4), shall be analysed under appropriate arrangements of the variable loads to determine those combinations of internal forces and moments which are critical for verifying the resistance of the individual members and of the connections.

(c) **Sway frames:**

Sway frames (see 4.4.4) shall be analysed under those arrangements of the variable loads which are critical for failure in a sway mode. In addition, sway frames shall also be analysed for the non-sway mode as described in (b).

(2) The initial sway imperfections specified in 4.1.4.3 - and member imperfections where necessary, see 4.1.4.2(4) - shall be included in the global analysis of all frames.

4.1.1.1 Sub-frames

(1) For the global analysis, the structure may be sub-divided into a number of sub-frames, provided that:

- the structural interaction between the sub-frames is reliably modelled.
- the arrangement of the sub-frames is appropriate for the structural system used.
- account is taken of possible adverse effects of interaction between the sub-frames.

4.1.1.2 Connection requirements

(1) The assumptions made in the global analysis of the structure shall be consistent with the anticipated type of behaviour of the connections.

(2) The assumptions made in the design of the members shall be consistent with (or conservative in relation to) the method used for the global analysis and with the anticipated type of behaviour of the connections.

(3) The requirements for the various types of connections are given in section 6.

(4) For classification of beam-to-column connections as rigid or semi-rigid see section 6.

4.1.1.3 Stiffness of bases

(1) Account shall be taken of the deformation characteristics of the bases or other foundations to which columns have moment-resisting connections. Appropriate rotational stiffness values shall be adopted in all methods of global analysis other than the rigid-plastic method.

(2) Where an actual pin or rocker is used, the rotational stiffness of the foundation shall be taken as zero.

(3) Optionally, appropriate rotational stiffness values may also be adopted to represent the semi-rigid nature of nominally pinned bases.

4.1.2 Elastic global analysis

4.1.2.1 Basis

(1) Elastic global analysis shall be based on the assumption that the stress-strain behaviour of the material is linear, whatever the stress level.

(2) This assumption may be maintained for both first-order and second-order elastic analysis, even where the resistance of a cross-section is based on its plastic resistance, see 4.1.3.2.

(3) Following a first-order elastic analysis, the calculated bending moments may be modified by redistributing up to 15% of the peak calculated moment in any member, provided that:

(a) the internal forces and moments in the frame remain in equilibrium with the applied loads, and

(b) all the members in which the moments are reduced have Class 1 or Class 2 cross-sections (see 5.1.3).

(4) The design assumptions for the connections shall satisfy the requirements specified in 4.1.1.2.

4.1.2.2 Cross-section requirements when elastic global analysis is used

(1) When elastic global analysis is used, any class of cross-section may be used for the members, provided that the design of the members takes into account the possible limits on the resistance of cross-sections due to local buckling.

(2) When elastic global analysis is used, the role of cross-section classification is to identify the extent to which the resistance of a cross-section is limited by its local buckling resistance.

(3) When all the compression elements of a cross-section are Class 2, the cross-section may be taken as capable of developing its full plastic resistance moment.

(4) When all the compression elements of a cross-section are Class 3, its resistance may be based on an elastic distribution of stresses across the cross-section, limited to the yield strength at the extreme fibres.

(5) When yielding first occurs on the tension side of the neutral axis, the plastic reserves of the tension zone may be utilised when determining the resistance of a Class 3 cross-section, using the method given in ENV 1993-1-3 Eurocode 3: Part 1.3[1].

(6) The resistance of a cross-section with a Class 2 compression flange but a Class 3 web may alternatively be determined by treating the web as an effective Class 2 web with a reduced effective area, using the method given in ENV 1994-1-1 Eurocode 4: Part 1.1[7].

(7) When any of the compression elements of a cross-section is Class 4 the cross-section shall

4.1.3 Plastic global analysis

4.1.3.1 Basis

(1) Plastic global analysis may be carried out using either:

- Rigid-Plastic methods.
- Elastic-Plastic methods.

(2) The following methods of Elastic-Plastic analysis may be used:

- Elastic - Perfectly Plastic
- Elasto-plastic

(3) When plastic global analysis is used, lateral restraint shall be provided at all plastic hinge locations at which plastic hinge rotation may occur under any load case.

(4) The restraint should be provided within a distance along the member from the theoretical plastic hinge location not exceeding half the depth of the member.

(5) Rigid-Plastic methods should not be used for second-order analysis, except as specified in 4.4.5.3.

(6) In "Rigid-Plastic" analysis elastic deformations of the members and the foundations are neglected and plastic deformations are assumed to be concentrated at plastic hinge locations.

(7) In "Elastic - Perfectly Plastic" analysis, it is assumed that the cross-section remains fully elastic until the plastic resistance moment is reached and then becomes fully plastic. Plastic deformations are assumed to be concentrated at the plastic hinge locations.

(8) In "Elasto-plastic" analysis, the bi-linear stress-strain relationship indicated in [EC3: figure 5.2.1] may be used for the grades of structural steel specified in Chapter 3. Alternatively, a more precise relationship may be adopted. The cross-section remains fully elastic until the stress in the extreme fibres reaches the yield strength. As the moment continues to increase, the section yields gradually as plasticity spreads across the cross-section and plastic deformations extend partially along the member.

(9) To avoid possible computational difficulties when using a computer for elasto-plastic analysis, the alternative bi-linear stress-strain relationship indicated in [EC3: figure 5.2.2] may be used if necessary.

(10) When elastic-plastic analysis is carried out, it may be assumed to be sufficient, in the case of building structures, to apply the loads in a series of increments, stopping when the full design load is reached, and to use the resulting internal forces and moments to check the resistances of the cross-sections and the buckling resistances of the members.

(11) In the case of building structures, it is not normally necessary to consider the effects of alternating plasticity.

4.1.3.2 Cross-section requirements for plastic global analysis

(1) When plastic global analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moments to develop.

(2) At plastic hinge locations, the cross-section of the member which contains the plastic hinge

shall have an axis of symmetry in the plane of loading.

(3) At plastic hinge locations, the cross-section of the member which contains the plastic hinge shall have a rotation capacity of not less than the required rotation at that plastic hinge location.

(4) To satisfy the above requirement, the required rotations should be determined from a rotation analysis.

(5) For building structures in which the required rotations are not calculated, all members containing plastic hinges shall have Class 1 cross-sections at the plastic hinge location.

(6) Where the cross-sections of the members vary along their length, the following additional criteria should be satisfied:

(a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance along the beam from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location.

(b) Adjacent to plastic hinge locations, the compression flange should be Class 1 for a distance along the beam from the plastic hinge location of not less than the greater of:

- *2d, where d is as defined in (a)*
- *the distance to the point at which the moment in the beam has fallen to 0,8 times the plastic moment resistance at the point concerned.*

(c) Elsewhere the compression flange should be Class 1 or Class 2 and the web should be Class 1, Class 2 or Class 3.

4.1.3.3 Column requirements for plastic analysis

(1) In frames it is necessary to ensure that where plastic hinges are required to form in members which are also under compression, adequate rotation capacity is available.

(2) This criterion may be assumed to be satisfied when elastic-plastic global analysis is used, provided that the cross-sections satisfy the requirements given in 4.1.3.2.

(3) When plastic hinge locations occur in the columns of frames designed using first-order rigid-plastic analysis, the columns should satisfy the following:

- *in braced frames:*

$$\bar{\lambda} \leq 0{,}40\,[Af_y/N_{Sd}]^{0,5} \qquad (4.0)$$

- *in unbraced frames:*

$$\bar{\lambda} \leq 0{,}32\,[Af_y/N_{Sd}]^{0,5} \qquad (4.1)$$

where $\bar{\lambda}$ is the in-plane non-dimensional slenderness (see 5.4.3.2) calculated using a buckling length equal to the system length.

(4) In frames designed using first-order rigid-plastic global analysis, columns containing plastic hinge locations should also be checked for resistance to in-plane buckling, using buckling lengths equal to their system lengths.

(5) Except for the method outlined in 4.4.5.3 (3)(b), first-order rigid-plastic global analysis should not be used for unbraced frames with more than two storeys.

4.1.4 Allowance for imperfections

4.1.4.1 Basis

(1) Appropriate allowances shall be incorporated to cover the effects of practical imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of fit and the unavoidable minor eccentricities present in practical connections.

(2) Suitable equivalent geometric imperfections may be used, with values which reflect the possible effects of all types of imperfection.

(3) The effects of imperfections shall be taken into account in the following cases:

a) Global analysis

b) Analysis of bracing systems

c) Member design

4.1.4.2 Method of application

(1) Imperfections shall be allowed for in the analysis by including appropriate additional quantities, comprising frame imperfections, member imperfections and imperfections for analysis of bracing systems.

(2) The effects of the frame imperfections given in 4.1.4.3 shall be included in the global analysis of the structure. The resulting forces and moments shall be used for member design.

(3) The effects of the imperfections given in 4.1.4.4 shall be included in the analysis of bracing systems. The resulting forces shall be used for member design.

(4) The effects of member imperfections (see 4.1.4.5) may be neglected when carrying out the global analysis of frames, except in sway frames (see 4.4.4.2) in the case of members which are subject to axial compression, which have moment-resisting connections and in which:

$$\bar{\lambda} > 0{,}5\,[Af_y/N_{Sd}]^{0,5} \qquad (4.2)$$

where N_{Sd} is the design value of the compressive force

and $\bar{\lambda}$ is the in-plane non-dimensional slenderness (see 5.5.3.2) calculated using a buckling length equal to the system length.

4.1.4.3 Frame imperfections

(1) The effects of imperfections shall be allowed for in frame analysis by means of an equivalent geometric imperfection in the form of an initial sway imperfection ϕ determined from:

$$\phi = k_c\, k_s\, \phi_o \qquad (4.3)$$

with $\phi_o = 1/200$

$k_c = [0{,}5 + 1/n_c]^{0,5}$ but $k_c \leq 1{,}0$

and $k_s = [0{,}2 + 1/n_s]^{0,5}$ but $k_s \leq 1{,}0$

where n_c is the number of columns per plane

and n_s is the number of storeys.

(2) Columns which carry a vertical load N_{Sd} of less than 50% of the mean value of the vertical load per column in the plane considered, shall not be included in n_c.

(3) Columns which do not extend through all the storeys included in n_s shall not be included in n_c. Those floor levels and roof levels which are not connected to all the columns included in n_c shall not be included when determining n_s.

Note: Where more than one combination of n_c and n_s satisfies these conditions, any such combination can safely be used.

(4) These initial sway imperfections apply in all horizontal directions, but need only be considered in one direction at a time.

(5) The possible torsional effects on the structure of anti-symmetric sways, on two opposite faces, shall also be considered.

(6) If more convenient, the initial sway imperfection may be replaced by a closed system of equivalent horizontal forces, see [EC3: figure 5.2.3].

(7) In beam-and-column building frames, these equivalent horizontal forces should be applied at each floor and roof level and should be proportionate to the vertical loads applied to the structure at that level, see [EC3: figure 4.1.5].

(8) The horizontal reactions at each support should be determined using the initial sway imperfection and not the equivalent horizontal forces. In the absence of actual horizontal loads, the net horizontal reaction is zero.

4.1.4.4 Imperfections for analysis of bracing systems

(1) The effects of imperfections shall be allowed for in the analysis of bracing systems which are required to provide lateral stability within the length of beams or compression members, by means of an equivalent geometric imperfection of the members to be restrained, in the form of an initial bow imperfection:

$$e_o = k_r L/500. \qquad (4.4)$$

where L is the span of the bracing system

and $k_r = [0{,}2 + 1/n_r]^{0,5}$ but $k_r \le 1{,}0$

in which n_r is the number of members to be restrained.

(2) For convenience, the initial bow imperfections of the members to be restrained by a bracing system, may be replaced by the equivalent stabilizing force shown in [EC3: figure 5.2.5].

(3) Where the bracing system is required to stabilize a beam, the force N in [EC3: figure 5.2.5] should be obtained from:

$$N = M/h \qquad (4.5)$$

where M is the maximum moment in the beam

and h is the overall depth of the beam.

(4) At points where beams or compression members are spliced, it shall also be verified that the bracing system is able to resist a local force equal to $k_r N/100$ applied to it by each beam or compression member which is spliced at that point, and to transmit this force to the adjacent points at which that beam or compression member is restrained, see [EC3: figure 5.2.6].

(5) When checking for this local force, any external loads acting on the bracing system shall also be included, but the forces arising from the imperfection given in (1) may be omitted.

4.1.4.5 Member imperfections

(1) Normally the effects of imperfections on member design shall be incorporated by using the appropriate buckling formulae given in this Eurocode.

(2) Alternatively, for a compression member, the initial bow imperfection specified in 5.4.3.3 may be included in a second-order analysis of the member.

(3) Where it is necessary (according to 4.1.4.2) to allow for member imperfections in the global analysis, the imperfections specified in 5.4.3.3 shall be included and second-order global analysis shall be used.

4.1.5 Actions for ultimate limit states

4.1.5.1 Verification conditions

(1) When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \tag{4.6}$$

where $E_{d,dst}$ is the design effect of the destabilizing actions
and $E_{d,stb}$ is the design effect of the stabilizing actions.

(2) When considering a limit state of rupture or excessive deformation of a section, member or connection (fatigue excluded) it shall be verified that:

$$S_d \leq R_d \tag{4.7}$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments)

and R_d is the corresponding design resistance,

each taking account of the respective design values of all structural properties.

(3) When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties.

(4) When considering a limit state of stability induced by second-order effects, it shall be verified that instability does not occur unless actions exceed their design values, taking account of the respective design values of all structural properties. In addition, sections shall be verified according to (2) above.

(5) When considering a limit state of rupture induced by fatigue, it shall be verified that the design value of the damage indicator D_d does not exceed unity, see Chapter 2.7.

(6) When considering effects of actions, it shall be verified that:

$$E_d \leq C_d \tag{4.8}$$

where E_d is the design value of the particular effect of actions being considered

and C_d is the design capacity for that effect of actions.

4.1.5.2 Combinations of actions

(1) For each load case, design values E_d for the effects of actions shall be determined from combination rules involving the design values of actions given in table 4.1.

Table 4.1 Design values of actions for use in the combination of actions.

Design situation	Permanent actions G_d	Variable actions Q_d		Accidental actions A_d
		Leading variable action	Accompanying variable actions	
Persistent and Transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\psi_0 \gamma_Q Q_k$	-
Accidental (if not specified differently elsewhere)	$\gamma_{GA} G_k$	$\psi_1 Q_k$	$\psi_2 Q_k$	$\gamma_A A_k$ (if A_d is not specified directly)

(2) The design values given in table 4.1 shall be combined using the following rules (given in symbolic form):[2]

- Persistent and transient design situations for verifications other than those relating to fatigue (fundamental combinations):

$$SUM_j \ \{ \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} \ \} + SUM_{i>1} \{ \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i} \} \qquad (4.9)$$

- Accidental design situations (if not specified differently elsewhere):

$$SUM_j \ \{ \gamma_{GA,j}\, G_{k,j} + A_d \} + \ SUM_{i>1} \{ \psi_{1,1}\, Q_{k,1} + \psi_{2,i}\, Q_{k,i} \} \qquad (4.10)$$

where:

$G_{k,j}$ are the characteristic values of the permanent actions

$Q_{k,1}$ is the characteristic value of one of the variable actions

$Q_{k,i}$ are the characteristic values of the other variable actions

A_d is the design value (specified value) of the accidental action

$\gamma_{G,j}$ is the partial safety factor for the permanent action $G_{k,j}$

$\gamma_{GA,j}$ is as $\gamma_{G,j}$ but for accidental design situations

$\gamma_{Q,i}$ is the partial safety factor for the variable action $Q_{k,i}$

and ψ_0, ψ_1, ψ_2 are factors defined in 1.3.4.2.

(3) Combinations for accidental design situations either involve an explicit accidental action A or refer to a situation after an accidental event (A = 0). Unless specified otherwise, $\gamma_{GA} = 1.0$ may be used.

(4) In expressions (4.9) and (4.10), indirect actions shall be introduced where relevant.

(5) For fatigue, see 2.7.

(6) Simplified combinations for building structures are given in 4.1.5.5.1.

4.1.5.3 Design values of permanent actions

(1) In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values and those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values (see 1.3.4.3(3)).

(2) Where the results of a verification may be very sensitive to variations of the magnitude of a single permanent action from place to place in the structure, this action shall be treated as consisting of separate unfavourable and favourable parts. This applies in particular to the verification of static equilibrium, see 4.1.5.4.

(3) Where a single permanent action is treated as consisting of separate unfavourable and favourable parts, allowance may be made for the relationship between these parts by adopting special design values (see 4.1.5.5 (3) for building structures).

(4) Except for the cases mentioned in (2), the whole of each permanent action should be represented throughout the structure by either its lower or its upper design value, whichever gives the more unfavourable effect.

(5) For continuous beams and frames, the same design value of the self-weight of the structure (evaluated as in 1.3.4.1(3)) may be applied to all spans, except for cases involving the static equilibrium of cantilevers (see 4.1.5.4).

4.1.5.4 Verification of static equilibrium

(1) For the verification of static equilibrium, destabilizing (unfavourable) actions shall be represented by upper design values and stabilizing (favourable) actions by lower design values (see 4.1.5.1(1)).

(2) For stabilizing effects, only those actions which can reliably be assumed to be present in the situation considered shall be included in the relevant combination.

(3) Variable actions should be applied where they increase the destabilizing effects but omitted where they would increase the stabilizing effects.

(4) Account should be taken of the possibility that non-structural elements might be omitted or removed.

(5) Permanent actions shall be represented by appropriate design values, depending on whether the destabilizing and stabilizing effects result from:

- the unfavourable and the favourable parts of a single permanent action, see (9) below, and/or
- different permanent actions, see (10) below.

(6) The self-weights of any unrelated structural or non-structural elements made of different construction materials should be treated as different permanent actions.

(7) The self-weight of a homogeneous structure should be treated as a single permanent action consisting of separate unfavourable and favourable parts.

(8) The self-weights of essentially similar parts of a structure (or of essentially uniform non-structural elements) may also be treated as separate unfavourable and favourable parts of a single permanent action.

(9) For building structures, the special partial safety factors given in 4.1.5.5.1 (3) apply to the unfavourable and the favourable parts of each single permanent action, as envisaged in 4.1.5.3 (2).

(10) For building structures, the normal partial safety factors given in 4.1.5.5.1 (1) apply to permanent actions other than those covered by (9).

(11) For closely bounded or closely controlled permanent actions, smaller ratios of partial safety factors may apply in the other Parts of Eurocode 3.

(12) Where uncertainty of the value of a geometrical dimension significantly affects the verification of static equilibrium, this dimension shall be represented in this verification by the most unfavourable value that it is reasonably possible for it to reach.

4.1.5.5 Partial safety factors for ultimate limit states

4.1.5.5.1 Partial safety factors for actions on building structures

(1) For the persistent and transient design situations the partial safety factors given in table 4.2 shall be used.

Table 4.2 Partial safety factors for actions on building structures for persistent and transient design situations

	Permanent actions (γ_G)	Variable actions (γ_Q)	
		Leading variable action	Accompanying variable actions
Favourable effect $\gamma_{F,inf}$	$\boxed{1{,}0^{*)}}$	-**)	-**)
Unfavourable effect $\gamma_{F,sup}$	$\boxed{1{,}35^{*)}}$	$\boxed{1{,}5}$	$\boxed{1{,}5}$

*) See also 2.3.3.1(3)
**) See Eurocode 1; in normal cases for building structures $\gamma_{Q,inf} = 0$.

(2) For accidental design situations to which expression (4.10) applies, the partial safety factors for the variable actions are taken as equal to 1,0.

(3) Where, according to 4.1.5.3(2), a single permanent action needs to be considered as consisting of unfavourable and favourable parts, the favourable part may, as an alternative, be multiplied by:

$$\gamma_{G,inf} = 1.1$$

and the unfavourable part by:

$$\gamma_{G,sup} = 1.35$$

provided that applying $\gamma_{G,inf} = 1.0$ both to the favourable part and to the unfavourable part does not give a more unfavourable effect.

(4) Where the components of a vectorial effect can vary independently, favourable components (eg. the longitudinal force) should be multiplied by a reduction factor:

$$\psi_{vec} = 0.8$$

(5) For building structures, as a simplification, expression (4.9) may be replaced by whichever of the following combinations gives the larger value:

- *considering only the most unfavourable variable action:*

$$\mathrm{SUM}_j \{ \gamma_{G,j} G_{k,j} \} + \gamma_{Q,1} Q_{k,1} \tag{4.11}$$

- *considering all unfavourable variable actions:*

$$\mathrm{SUM}_j \{\gamma_{G,j} G_{k,j} \} + \mathrm{SUM}_{i>=1} \{ 0{,}9\gamma_{Q,i} Q_{k,i} \} \tag{4.12}$$

4.1.5.5.2 Partial safety factors for resistances

(1) Partial safety factors for resistances are given in the relevant clauses in Chapters 5 and 6.

(2) Where structural properties are determined by testing see Chapter 9.

(3) For fatigue verifications see Chapter 2.7.

4.1.6 Actions for serviceability limit states

(1) It shall be verified that:

$$E_d \leq C_d \quad \text{or} \quad E_d \leq R_d \tag{4.13}$$

where:

C_d is a nominal value or a function of certain design properties of materials related to the design effect of actions considered, and

E_d is the design effect of actions, determined on the basis of one of the combinations defined below.

The required combination is identified in the particular clause for each serviceability verification, see 2.2.1(4) and 2.3.1(4).

(2) Three combinations of actions for serviceability limit states are defined by the following expressions:

Rare combination:

$$\mathrm{SUM}_j \{ G_{k,j} \} + \mathrm{SUM}_{i>1} \{ Q_{k,1} + \psi_{0,i} Q_{k,i} \} \tag{4.14}$$

Frequent combination:

$$\mathrm{SUM}_j \{ G_{k,j} \} + \mathrm{SUM}_{i>1} \{\psi_{1,1} Q_{k,1} + \psi_{2,i} Q_{k,i} \} \tag{4.15}$$

Quasi-permanent combination:

$$\mathrm{SUM}_j \{ G_{k,j} \} + \mathrm{SUM}_{i>=1} \{\psi_{2,i} Q_{k,i} \} \tag{4.16}$$

where the notation is defined in 4.1.5.2(2)

(3) Where simplified compliance rules are given in the relevant clauses dealing with serviceability limit states, detailed calculations using combinations of actions are not required.

(4) Where the design considers compliance of serviceability limit states by detailed calculations, simplified expressions may be used for building structures.

(5) For building structures, as a simplification, expression (4.14) for the rare combination may be replaced by whichever of the following combinations gives the larger value:

- *considering only the most unfavourable variable action:*

$$SUM_j \{ G_{k,j} \} + Q_{k,1} \qquad (4.17)$$

- *considering all unfavourable variable actions:*

$$SUM_j \{ G_{k,j} \} + 0,9\ SUM_{i>=1} \{ Q_{k,i} \} \qquad (4.18)$$

These two expressions may also be used as a substitute for expression (4.15) for the frequent combination.

(6) Values of γ_M shall be taken as 1,0 for all serviceability limit states, except where stated otherwise in particular clauses.

4.2 Simple frames

4.2.1 Methods of analysis

(1) The internal forces and moments in a statically determinate structure shall generally be obtained using statics.

4.2.2 Connection requirements

(1) In simple framing the connections between the members may be assumed not to develop moments. In the global analysis, members may be assumed to be effectively pin connected.

(2) The connections should satisfy the requirements for nominally pinned connections, (see section 6 and 7)

4.2.3 Bracing requirements

(1) All structures shall have sufficient stiffness to limit lateral sway. This may be supplied by the sway stiffness of bracing systems, which may be:

- triangulated frames
- rigid-jointed frames
- shear walls, cores and the like

(2) A frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system.

(3) A steel frame may be classified as braced if the bracing system reduces its horizontal displacements by at least 80%.

(4) A braced frame may be treated as fully supported laterally.

(5) The effects of the initial sway imperfections (see 4.1.4.3) in the braced frame shall be taken into account in the design of the bracing system.

(6) The initial sway imperfections (or the equivalent horizontal forces, see 4.1.4.3) plus any horizontal loads applied to a braced frame, may be treated as affecting only the bracing system.

(7) The bracing system should be designed to resist:

- *any horizontal loads applied to the frames which it braces,*
- *any horizontal or vertical loads applied directly to the bracing system,*
- *the effects of the initial sway imperfections (or the equivalent horizontal forces) from the bracing system itself and from all the frames which it braces.*

(8) *Where the bracing system is a frame or sub-frame, it may itself be either sway or non-sway, see 4.4.4.2.*

(9) *When applying the criterion given in 4.4.4.2 (3) to a frame or sub-frame acting as a bracing system, the total vertical load acting on all the frames which it braces should also be included.*

(10) *When applying the criterion given in 4.4.4.2 (4) to a frame or sub-frame acting as a bracing system, the total horizontal and vertical load acting on all the frames which it braces should also be included, plus the initial sway imperfection applied in the form of the equivalent horizontal forces from the bracing system itself and from all the frames which it braces.*

4.3 Continuous braced frames

4.3.1 Methods of analysis

(1) The internal forces and moments in a statically indeterminate structure may generally be determined using either:

a) elastic global analysis (4.1.2)

b) plastic global analysis (4.1.3)

(3) Elastic global analysis may be used in all cases.

(4) Plastic global analysis may be used only where the member cross-sections satisfy the requirements specified in 4.1.3.3 and 4.1.3.2 and the steel material satisfies the requirements specified in 3.2.2.2.

(5) *When the global analysis is carried out by applying the loads in a series of increments, it may be assumed to be sufficient, in the case of building structures, to adopt simultaneous proportional increases of all loads.*

4.3.2 Connection requirements

(1) Elastic analysis should be based on the assumption of full continuity, with rigid connections which satisfy the requirements given in Section 6.

(2) Rigid-Plastic analysis should be based on the assumption of full continuity, with full strength connections which satisfy the requirements given in Section 6.

(3) Elastic-Plastic analysis should be based on the assumption of full continuity, with rigid full-strength connections which satisfy the requirements given in Section 6.

4.3.3 Bracing requirements

(1) The same bracing requirements specified in 4.2.3 apply to continuous braced frames

4.4 Continuous unbraced frames

4.4.1 Methods of analysis

(1) As for continuous braced frames, refer to 4.3.1 .

4.4.2 Effects of deformations

(1) The internal forces and moments may generally be determined using either:

a) first-order theory, using the initial geometry of the structure using design methods which make indirect allowances for second-order effects (4.4.5).

b) second-order theory, taking into account the influence of the deformation of the structure.

4.4.3 Connection requirements

(1) As for continuous braced frames, refer to 4.3.2 .

4.4.4 Sway stability

4.4.4.1 Sway stiffness

(1) All structures shall have sufficient stiffness to limit lateral sway. This may be supplied by the sway stiffness of the frames, which may be supplied by one or more of the following:

- triangulation
- the stiffness of the connections
- cantilever columns

4.4.4.2 Classification as sway or non-sway

(1) A frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.

(2) Any other frame shall be classified as a sway frame and the effects of the horizontal displacements of its nodes taken into account in its design, see 4.4.2.

(3) A frame may be classified as non-sway for a given load case if the elastic critical load ratio V_{Sd}/V_{cr} for that load case satisfies the criterion:

$$V_{Sd}/V_{cr} \leq 0{,}1 \qquad (4.19)$$

where V_{Sd} is the design value of the total vertical load

and V_{cr} is its elastic critical value for failure in a sway mode.

(4) Beam-and-column type plane frames in building structures with beams connecting each column at each storey level see [EC3: figure 5.2.7] may be classified as non-sway for a given load case if the following criterion is satisfied. When first-order theory is used, the horizontal displacements in each storey due to the design loads (both horizontal and vertical), plus the initial sway imperfection (see 4.1.4.3) applied in the form of equivalent horizontal forces, should satisfy the criterion:

$$\left(\frac{\delta}{h}\right)\left(\frac{V}{H}\right) \leq 0{,}1 \qquad (4.20)$$

(5) For sway frames, the requirements for frame stability given in 4.4.5 should also be satisfied.

4.4.5 Frame stability

4.4.5.1 General

(1) All frames shall have adequate resistance to failure in a sway mode. However, where the frame is shown to be a non-sway frame, see 4.4.4.2, no further sway mode verification is required.

(2) All frames, including sway frames, shall also be checked for adequate resistance to failure in non-sway modes.

(3) A check should be included for the possibility of local storey-height failure modes.

(4) Frames with non-triangulated pitched roofs shall also be checked for snap-through buckling.

(5) The use of rigid-plastic analysis with plastic hinge locations in the columns shall be limited to cases where it can be demonstrated that the columns are able to form hinges with sufficient rotation capacity, see 4.1.3.3.

4.4.5.2 Elastic analysis of sway frames

(1) When elastic global analysis is used, the second-order effects in the sway mode shall be included, either directly by using second-order elastic analysis, or indirectly by using one of the following alternatives:

(a) first-order elastic analysis, with amplified sway moments.

(b) first-order elastic analysis, with sway-mode buckling lengths.

(2) When second-order elastic global analysis is used, in-plane buckling lengths for the non-sway mode may be used for member design.

(3) In the amplified sway moments method, the sway moments found by a first-order elastic analysis should be increased by multiplying them by the ratio:

$$1 / (1 - V_{sd} / V_{cr}) \qquad (4.21)$$

where V_{sd} is the design value of the total vertical load

and V_{cr} is its elastic critical value for failure in a sway mode.

(4) The amplified sway moments method should not be used when the elastic critical load ratio V_{sd} / V_{cr} is more than 0,25.

(5) Sway moments are those associated with the horizontal translation of the top of a storey relative to the bottom of that storey. They arise from horizontal loading and may also arise from vertical loading if either the structure or the loading is asymmetrical.

(6) As an alternative to determining V_{sd} / V_{cr} direct the following approximation may be used in beam-and-column type frames as described in 4.4.4.2(4):

$$\frac{V_{sd}}{V_{cr}} = \left(\frac{\delta}{h}\right)\left(\frac{V}{H}\right) \qquad (4.22)$$

(7) When the amplified sway moments method is used, in-plane buckling lengths for the non-sway

mode may be used for member design.

(8) When first-order elastic analysis, with sway-mode in-plane buckling lengths, is used for column design, the sway moments in the beams and the beam-to-column connections should be amplified by at least 1,2 unless a smaller value is shown to be adequate by analysis.

4.4.5.3 Plastic analysis of sway frames

(1) When plastic global analysis is used, allowance shall be made for the second-order effects in the sway mode.

(2) This should generally be done by using second-order elastic-plastic analysis, see 4.1.3.

(3) However, as an alternative, rigid-plastic analysis with indirect allowance for second-order effects, as given in (4) below, may be adopted in the following cases:

(a) Frames one or two storeys high in which either:

- *no plastic hinge locations occur in the columns, or*
- *the columns satisfy 4.1.3.3.*

(b) Frames with fixed bases, in which the sway failure mode involves plastic hinge locations in the columns at the fixed bases only, see [EC3: figure 5.2.8], and the design is based on an incomplete mechanism in which the columns are designed to remain elastic at the calculated plastic hinge moment.

(4) In the cases given in (3), V_{Sd}/V_{cr} should not exceed 0,20 and all the internal forces and moments should be amplified by the ratio given in 4.4.5.2 (3).

(5) In-plane buckling lengths for the non-sway mode may be used for member design. These should be determined with due allowance for the effects of plastic hinges.

5 Member design

5.1 General

5.1.1 Basis

(1) Steel structures and components shall be so proportioned that the basic design requirements for the ultimate limit state given in Chapter 4 are satisfied.

(2) *When checking the resistance of cross-sections and members of a frame, each member may be treated as isolated from the frame, with forces and moments applied to each end as determined from the frame analysis. The conditions of restraint at each end should be determined by considering the member as part of the frame and should be consistent with the type of analysis and mode of failure .*

(3) The partial safety factor γ_M shall be taken as follows:

- resistance of Class 1, 2 or 3 cross-section:*) γ_{M0} = $\boxed{1,1}$
- resistance of Class 4 cross-section:*) γ_{M1} = $\boxed{1,1}$
- resistance of member to buckling: γ_{M1} = $\boxed{1,1}$
- resistance of net section at bolt holes: γ_{M2} = $\boxed{1,25}$
- resistance of connections: see Chapters 6 & 7

5.1.2 Section properties

5.1.2.1 Gross cross-section

(1) The properties of the gross cross-section shall be determined using the specified dimensions. Holes for fasteners need not be deducted, but allowance shall be made for larger openings. Splice materials and battens shall not be included.

5.1.2.2 Net area

(1) The net area of a member or element cross-section shall be taken as its gross area less appropriate deductions for all holes and other openings.

(2) When calculating net section properties, the deduction for a single fastener hole shall be the gross cross-sectional area of the hole in the plane of its axis. For countersunk holes, appropriate allowance shall be made for the countersunk portion.

(3) Provided that the fastener holes are not staggered, the total area to be deducted for fastener

holes shall be the maximum sum of the sectional areas of the holes in any cross-section perpendicular to the member axis.

(4) When the fastener holes are staggered, the total area to be deducted for fastener holes shall be the greater of:

a) the deduction for non-staggered holes given in (3)

b) the sum of the sectional areas of all holes in any diagonal or zig-zag line extending progressively across the member or part of the member, less $s^2t/(4p)$ for each gauge space in the chain of holes, see [EC3: figure 5.4.1].

where s is the staggered pitch, the spacing of the centres of two consecutive holes in the chain measured parallel to the member axis.

p is the spacing of the centres of the same two holes measured perpendicular to the member axis.

and t is the thickness.

(5) In an angle or other member with holes in more than one plane, the spacing p shall be measured along the centre of thickness of the material, see [EC3: figure 5.4.2].

5.1.2.3 Shear lag effects

(1) Shear lag effects in flanges may be neglected provided that:

- *for outstand elements:* $c \leq L_0/20$
- *for internal elements:* $b \leq L_0/10$

where L_0 *is the length between points of zero moment.*

b *is the breadth*

and c *is the outstand*

(2) When these limits are exceeded an effective breadth of flange should be taken.

(3) The calculation of effective breadths of flanges is covered in ENV 1993-1-3 Eurocode 3: Part 1.3) and ENV 1993-2 Eurocode 3: Part 2*).*

5.1.3 Classification of cross-sections

5.1.3.1 General

(1) Four classes of cross-sections are defined, as follows:

- **Class 1 cross-sections** are those which can form a plastic hinge with the rotation capacity required for plastic analysis.
- **Class 2 cross-sections** are those which can develop their plastic moment resistance, but have limited rotation capacity.
- **Class 3 cross-sections** are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- **Class 4 cross-sections** are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or

compression resistance.

(2) Effective widths may be used in Class 4 cross-sections to make the necessary allowances for reductions in resistance due to the effects of local buckling, see 5.1.3.2.

(3) The classification of a cross-section depends on the proportions of each of its compression elements.

(4) Compression elements include every element of a cross-section which is either totally or partially in compression, due to axial force or bending moment, under the load combination considered.

(5) The various compression elements in a cross-section (such as a web or a flange) can, in general, be in different classes.

(6) A cross-section is normally classified by quoting the highest (least favourable) class of its compression elements.

(7) Alternatively the classification of a cross-section may be defined by quoting both the flange classification and the web classification.

(8) The limiting proportions for Class 1, 2 and 3 compression elements should be obtained from [EC3: table 5.3.1]. An element which fails to satisfy the limits for Class 3 should be taken as Class 4.

5.1.3.2 Effective cross-section properties of Class 4 cross-sections

(1) The effective cross-section properties of Class 4 cross-sections shall be based on the effective widths of the compression elements (see 5.1.3.1 (2)).

(2) The effective widths of flat compression elements should be obtained using [EC3: table 5.3.2] for internal elements and [EC3: table 5.3.3] for outstand elements.

(3) As an approximation, the reduction factor ρ may be obtained as follows:

- *when* $\bar{\lambda}_p \leq 0{,}673$: $\rho = 1$
- *when* $\bar{\lambda}_p > 0{,}673$: $\rho = (\bar{\lambda}_p - 0{,}22)/\bar{\lambda}_p^2$ *(5.0)*

where $\bar{\lambda}_p$ *is the plate slenderness given by:*

$$\bar{\lambda}_p = [f_y / \sigma_{cr}]^{0,5} = (\bar{b}/t) / (28{,}4\, \varepsilon \sqrt{k_\sigma})$$

in which t *is the relevant thickness*

σ_{cr} *is the critical plate-buckling stress*

k_σ *is the buckling factor corresponding to the stress ratio ψ from [EC3: table 5.3.2 or table 5.3.3] as appropriate*

and $\bar{b}$ *is the appropriate width, see [EC3: table 5.3.1] as follows:*

$\bar{b} = d$ *for webs*

$\bar{b} = b$ *for internal flange elements (except RHS)*

$\bar{b} = b - 3t$ *for flanges of RHS*

$\bar{b} = c$ *for outstand flanges*

$\bar{b} = (b + h)/2$ for equal-leg angles

$\bar{b} = h$ or $(b + h)/2$ for unequal-leg angles

(4) *To determine the effective widths of flange elements, the stress ratio ψ used in table 5.3.2 or table 5.3.3 may be based on the properties of the gross cross-section.*

(5) *To determine the effective width of a web, the stress ratio ψ used in table 5.3.2 may be obtained using the effective area of the compression flange but the gross area of the web.*

(6) *Generally the centroidal axis of the effective cross-section will shift by a dimension e compared to the centroidal axis of the gross cross-section, see figures 5.3.1 and 5.3.2. This should be taken into account when calculating the properties of the effective cross-section.*

(7) *When the cross-section is subject to an axial force, the method given in 5.4.1.3 should be used to take account of the additional moment ΔM given by:*

$$\Delta M = N\, e_N \qquad (5.12)$$

where e_N is the shift of the centroidal axis when the effective cross-section is subject to uniform compression, see figure 5.3.1.

and N is positive for compression.

(8) *Except as given in (9), for greater economy the plate slenderness $\bar{\lambda}_p$ of an element may be determined using the maximum calculated compressive stress $\sigma_{com.Ed}$ in that element in place of the yield strength f_y, provided that $\sigma_{com.Ed}$ is determined using the effective widths b_{eff} of all the compression elements. This procedure generally requires an iterative calculation in which ψ is determined again at each step from the stresses calculated on the effective cross-section defined at the end of the previous step, including the stresses from the additional moment ΔM.*

(9) *However, when verifying the design buckling resistance of a member using section 5.5, the plate slenderness $\bar{\lambda}_p$ of an element should always be based on its yield strength f_y when calculating the values of A_{eff}, e_N and W_{eff}.*

5.1.3.3 Effects of transverse forces on webs

(1) The effects of significant transverse compressive stresses on the local buckling resistance of a web shall be taken into account in design. Such stresses may arise from transverse forces on a member and at member intersections.

(2) *The presence of significant transverse compressive stresses may effectively reduce the maximum values of the depth-to-thickness ratios d/t_w for Class 1, Class 2 and Class 3 webs below those given in [EC3: table 5.3.1], depending on the spacing of any web stiffeners.*

(3) *A recognised method of verification should be used. Reference may be made to the application rules for stiffened plating given in ENV 1993-2 Eurocode 3: Part 2*[1]*.*

5.2 Laterally restrained beams

5.2.1 Bending

5.2.1.1 Basis

(1) In the absence of shear force, the design value of the bending moment M_{Sd} at each cross-section shall satisfy:

$$M_{Sd} \leq M_{c.Rd} \qquad (5.2)$$

where $M_{c.Rd}$ is the design moment resistance of the cross- section, taken as the smallest of:

a) the design plastic resistance moment of the gross section

$M_{pl.Rd} = W_{pl} f_y / \gamma_{M0}$

b) the design local buckling resistance moment of the gross section

$M_{o.Rd} = W_{eff} f_y / \gamma_{M1}$

where W_{eff} is the effective section modulus (see 5.3.5).

c) the design ultimate resistance moment of the net section at bolt holes $M_{u.Rd}$, see 5.2.1.4.

(2) For a Class 3 cross-section the design moment resistance of the gross section shall be taken as the design elastic resistance moment given by:

$$M_{el.Rd} = W_{el} f_y / \gamma_{M0} \qquad (5.3)$$

5.2.1.2 Bending with low shear ($V_{sd} \leq 0.5\ V_{pl.rd}$).

(1) When shear is low ($V_{sd} \leq 0.5\ V_{pl.rd}$), the design moment resistance of a cross-section without holes for fasteners may be determined as follows:

Class 1 or 2 cross-sections:	$M_{c.Rd} = W_{pl} f_y / \gamma_{M0}$
Class 3 cross-sections:	$M_{c.Rd} = W_{el} f_y / \gamma_{M0}$
Class 4 cross-sections:	$M_{c.Rd} = W_{eff} f_y / \gamma_{M1}$

5.2.1.3 Bending with high shear ($V_{sd} \geq 0.5\ V_{pl.rd}$).

(1) The theoretical plastic resistance moment of a cross-section is reduced by the presence of shear. For small values of the shear force this reduction is so small that it is counter-balanced by strain hardening and may be neglected. However, when the shear force exceeds half the plastic shear resistance, allowance shall be made for its effect on the plastic resistance moment.

(2) Provided that the design value of the shear force V_{Sd} does not exceed 50% of the design plastic shear resistance $V_{pl.Rd}$ no reduction need be made in the resistance moments given by 5.2.1.2.

(3) When V_{Sd} exceeds 50% of $V_{pl.Rd}$ the design resistance moment of the cross-section should be reduced to $M_{V.Rd}$ the reduced design plastic resistance moment allowing for the shear force, obtained as follows:

(a) for cross-sections with equal flanges, bending about the major axis:

$$M_{V.Rd} = \left(W_{pl} - \frac{\rho A_v^2}{4\ t_w} \right) f_y / \gamma_{M0} \quad \text{but } M_{V.Rd} \leq M_{c.Rd} \qquad (5.4)$$

where $\rho = (2V_{Sd} / V_{pl.Rd} - 1)^2$

(b) for other cases $M_{V.Rd}$ should be taken as the design plastic resistance moment of the cross-section, calculated using a reduced strength $(1 - \rho) f_y$ for the shear area, but not more than $M_{c.Rd}$.

Note: Paragraph (3) applies to Class 1, 2, 3 and 4 cross-sections. The appropriate value of $M_{c.Rd}$ should be used, see 5.2.1.2.

5.2.1.4 Holes for fasteners

(1) Fastener holes in the tension flange need not be allowed for, provided that for the tension flange:

$$0{,}9\ [A_{f.net}/A_f] \geq [f_y/f_u]\ [\gamma_{M2}/\gamma_{M0}] \tag{5.5}$$

(2) When $A_{f.net}/A_f$ is less than this limit, a reduced flange area may be assumed which satisfies the limit.

(3) Fastener holes in the tension zone of the web need not be allowed for, provided that the limit given in (1) is satisfied for the complete tension zone comprising the tension flange plus the tension zone of the web.

(4) Fastener holes in the compression zone of the cross-section need not be allowed for, except for oversize and slotted holes.

5.2.2 Shear

(1) The design value of the shear force V_{Sd} at each cross-section shall satisfy:

$$V_{Sd} \leq V_{pl.Rd} \tag{5.6}$$

where $V_{pl.Rd}$ is the design plastic shear resistance given by:

$$V_{pl.Rd} = A_v\ (f_y\ /\ \sqrt{3})\ /\ \gamma_{M0}$$

where A_v is the shear area.

(2) The shear area A_v may be taken as follows:

a) rolled I and H sections, load parallel to web — $A-2bt_f+(t_w+2r)t_f$

b) rolled channel sections, load parallel to web — $A-2bt_f+(t_w+r)t_f$

c) welded I, H and box sections, load parallel to web — $\Sigma(dt_w)$

d) welded I, H, channel and box sections, load parallel to flanges — $A-\Sigma(dt_w)$

e) rolled rectangular hollow sections of uniform thickness:
   - load parallel to depth — $Ah/(b+h)$
   - load parallel to breadth — $Ab/(b+h)$

f) circular hollow sections and tubes of uniform thickness — $2A/\pi$

g) plates and solid bars — A

where A is the cross-section area
b is the overall breadth
d is the depth of the web
h is the overall depth
r is the root radius
t_f is the flange thickness
and t_w is the web thickness

(3) *For other cases A_v should be determined analogously.*

(4) *For simplicity, the value of A_v for a rolled I, H or channel section, load parallel to web, may be taken as $1{,}04ht_w$.*

(5) *In appropriate cases the formulae in (2) may be applied to components of a built-up section.*

(6) *If the web thickness is not constant, t_w should be taken as the minimum thickness.*

(7) In addition the shear buckling resistance shall also be verified as specified in 5.6 when:

- for an unstiffened web:

  $d/t_w > 69\varepsilon$
- for a stiffened web:

  $d/t_w > 30\varepsilon \sqrt{k_\tau}$

where k_τ is the buckling factor for shear, see 5.6

and $\varepsilon = [235/f_y]^{0,5}$ (f_y in N/mm²)

(8) Fastener holes need not be allowed for in shear verifications provided that:

$$A_{v.net} \geq (f_y/f_u)\, A_v \qquad (5.7)$$

When $A_{v.net}$ is less than this limit, an effective shear area of $(f_u/f_y)\, A_{v.net}$ may be assumed.

(9) The block shear criterion given in 6.5 shall also be verified at the ends of a member.

5.2.3 Resistance of webs to transverse forces

5.2.3.1 Basis

(1) The resistance of an unstiffened web to transverse forces applied through a flange, is governed by one of the following modes of failure:

- crushing of the web close to the flange, accompanied by plastic deformation of the flange,
- crippling of the web in the form of localised buckling and crushing of the web close to the flange, accompanied by plastic deformation of the flange.
- buckling of the web over most of the depth of the member

(2) A distinction is made between two types of load application, as follows:

- Forces applied through one flange and resisted by shear forces in the web, see [EC3: figure 5.7.1(a)].
- Forces applied to one flange and transferred through the web directly to the other flange, see [EC3: figure 5.7.1(b)].

(3) *Where forces are applied through one flange and resisted by shear forces in the web, the resistance of the web to transverse forces should be taken as the smaller of:*

- *the crushing resistance (see 5.2.3.3).*
- *the crippling resistance (see 5.2.3.4).*

(4) *Where forces are applied to one flange and transferred through the web directly to the other flange, the resistance of the web to transverse forces should be taken as the smaller of:*

- *the crushing resistance (see 5.2.3.3).*
- *the buckling resistance (see 5.2.3.5).*

(5) *Where, in a practical case, the details are such that there is doubt over which mode governs, all three modes should be considered.*

(6) *In addition the effect of the transverse force on the moment resistance of the member should be considered, see 5.1.3.3.*

(7) *The crippling resistance of a stiffened web between the locations of transverse web stiffeners, is basically similar to that of an unstiffened web, with some increase due to the presence of the stiffeners.*

5.2.3.2 Length of stiff bearing

(1) The length of stiff bearing on the flange is the distance over which the applied force is effectively distributed.

(2) The resistance of the web to transverse forces is influenced by the length of stiff bearing.

(3) *The length of stiff bearing s_s should be determined by dispersion of load through solid steel material which is properly fixed in place at a slope of 1:1, see [EC3: figure 5.7.2]. No dispersion should be taken through loose packs.*

5.2.3.3 Crushing resistance

(1) *The design crushing resistance $R_{y.Rd}$ of the web of an I, H or U section should be obtained from:*

$$R_{y.Rd} = (s_s + s_y)\, t_w\, f_{yw} / \gamma_{M1} \tag{5.8}$$

in which s_y is given by:

$$s_y = 2t_f\,(b_f / t_w)^{0,5}\, [f_{yf} / f_{yw}]^{0,5}\, [1 - (\gamma_{M0}\, \sigma_{f.Ed} / f_{yf})^2]^{0,5} \tag{5.9}$$

but b_f should not be taken as more than $25t_f$

where $\sigma_{f.Ed}$ is the longitudinal stress in the flange.

(2) *For a rolled I, H or U section s_y may alternatively be obtained from:*

$$s_y = \frac{2,5\,(h - d)\,[1 - (\gamma_{M0}\, \sigma_{f.Ed} / f_{yf})^2]^{0,5}}{(1 + 0,8\, s_s/(h - d))} \tag{5.10}$$

(3) *At the end of a member s_y should be halved.*

(4) *For wheel loads from cranes, transmitted through a crane rail bearing on a flange but not welded to it, the design crushing resistance of the web $R_{y.Rd}$ should be taken as:*

$$R_{y.Rd} = s_y\, t_w\, f_{yw} / \gamma_{M1} \tag{5.11}$$

in which:

$$s_y = k_R \left[\frac{I_f + I_R}{t_w}\right]^{1/3} [1 - (\gamma_{M0}\, \sigma_{f.Ed} / f_{yf})^2\,]^{0,5} \quad (5.12)$$

or more approximately:

$$s_y = 2(h_R + t_f)\,[1 - (\gamma_{M0}\, \sigma_{f.Ed} / f_{yf})^2\,]^{0,5} \quad (5.13)$$

where h_R is the height of the crane rail

I_f is the second moment of area of the flange about its horizontal centroidal axis

I_R is the second moment of area of the crane rail about its horizontal centroidal axis

and k_R is a constant taken as follows:

- *when the crane rail is mounted directly on the flange, $k_R = 3,25$*
- *when a suitable resilient pad not less than 5 mm thick is interposed between the crane rail and the beam flange: $k_R = 4,0$*

5.2.3.4 Crippling resistance

(1) The design crippling resistance $R_{a.Rd}$ of the web of an I, H or U section should be obtained from:

$$R_{a.Rd} = 0,5t_w^2(Ef_{yw})^{0,5}\,[(t_f / t_w)^{0,5} + 3(t_w / t_f)(s_s / d)] / \gamma_{M1} \quad (5.14)$$

where s_s is the length of stiff bearing from 5.2.3.2(3)

but s_s/d should not be taken as more than 0,2.

(2) Where the member is also subject to bending moments, the following criteria should be satisfied:

$$F_{Sd} \le R_{a.Rd} \quad (5.15a)$$

$$M_{Sd} \le M_{c.Rd} \quad (5.15b)$$

and

$$\frac{F_{Sd}}{R_{a.Rd}} + \frac{M_{Sd}}{M_{c.Rd}} \le 1,5 \quad (5.15c)$$

5.2.3.5 Buckling resistance

(1) The design buckling resistance $R_{b.Rd}$ of the web of an I, H or U section should be obtained by considering the web as a virtual compression member with an effective breadth b_{eff} obtained from:

$$b_{eff} = [h^2 + s_s^2]^{0,5} \quad (5.16)$$

(2) Near the ends of a member (or at openings in the web) the effective breadth b_{eff} should not be taken as greater than the breadth actually available, measured at mid-depth, see [EC3: figure 5.7.3].

(3) The buckling resistance should be determined from 5.4.3 using buckling curve c and $\beta_A = 1$.

(4) The buckling length of the virtual compression member should be determined from the

conditions of lateral and rotational restraint at the flanges at the point of load application.

(5) *The flange through which the load is applied should normally be restrained in position at the point of load application. Where this is not practicable, a special buckling investigation should be carried out.*

5.2.3.6 Transverse stiffeners

(1) *When checking the buckling resistance, the effective cross-section of a stiffener should be taken as including a width of web plate equal to $30\varepsilon t_w$, arranged with $15\varepsilon t_w$ each side of the stiffener, see [EC3: figure 5.7.4]. At the ends of the member (or openings in the web) the dimension of $15\varepsilon t_w$ should be limited to the actual dimension available.*

(2) *The out-of-plane buckling resistance should be determined from 5.4.3, using buckling curve c and a buckling length ℓ of not less than 0,75d, or more if appropriate for the conditions of restraint.*

(3) *End stiffeners and stiffeners at internal supports should normally be double sided and symmetric about the centreline of the web.*

(4) *Stiffeners at locations where significant external forces are applied should preferably be symmetric.*

(5) *Where single sided or other asymmetric stiffeners are used, the resulting eccentricity should be allowed for, using clause 5.5.4.*

(6) *In addition to checking the buckling resistance, the cross-section resistance of a load bearing stiffener should also be checked adjacent to the loaded flange. The width of web plate included in the effective cross-section should be limited to s_y (see 5.2.3.3) and allowance should be made for any openings cut in the stiffener to clear the web-to-flange welds.*

(7) *For intermediate transverse stiffeners it is only necessary to check the buckling resistance, provided that they are not subject to external loads.*

5.2.3.7 Flange induced buckling

(1) To prevent the possibility of the compression flange buckling in the plane of the web, the ratio d/t_w of the web shall satisfy the following criterion:

$$d/t_w \leq k\,(E/f_{yf})\,[A_w/A_{fc}]^{0,5} \qquad (5.17)$$

where A_w is the area of the web

A_{fc} is the area of the compression flange

and f_{yf} is the yield strength of the compression flange.

(2) *The value of the factor k should be taken as follows:*

Class 1 flanges : 0,3

Class 2 flanges : 0,4

Class 3 or Class 4 flanges : 0,55

(3) *When the girder is curved in elevation, with the compression flange on the concave face, the criterion should be modified to:*

$$d/t_w \leq \frac{k(E/f_{yf})\,[A_w/A_{fc}]^{0,5}}{[1 + dE/(3rf_{yf})]^{0,5}} \qquad (5.18)$$

where r is the radius of curvature of the compression flange.

(4) When the girder has transverse web stiffeners, the limiting value of d/t_w may be increased accordingly.

5.3 Laterally unrestrained beams

5.3.1 Elastic critical moment

5.3.1.1 Uniform cross-sections symmetrical about both axes

(1) Elastic critical moment is given by:

$$M_{cr} = \frac{\pi^2 E I_z}{L^2}\left[\frac{I_w}{I_z} + \frac{L^2 G I_t}{\pi^2 E I_z}\right]^{0,5} \qquad (5.19)$$

This formula may only be used if the beam satisfies the following conditions:

- loading applied through shear centre
- restrained at each end against lateral movement
- restrained at each end against rotation about the longitudinal axis
- ends free to rotate in plan
- subject to uniform moment, alternatively the maximum moment should be assumed to be applied along entire length of beam

(2) Alternatively elastic critical moment is given by the general formula:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2}\left\{\left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + [C_2 z_g]^2\right]^{0,5} - C_2 z_g\right\} \qquad (5.20)$$

where $G = \dfrac{E}{2(1+\nu)}$

$z_g = z_a - z_s$

z_a is the coordinate of the point of load application

z_s is the coordinate of the shear centre

I_t is the torsion constant

I_w is the warping constant

I_z is the second moment of area about the minor axis

L is the length of the beam between points which have lateral restraint

(3) The sign convention for determining z_g is:

- for gravity loads z_g is positive for loads applied above the shear centre
- in the general case z_g is positive for loads acting towards the shear centre from their point of application.

(4) The effective length factors k and k_w vary from 0,5 for full fixity to 1,0 for no fixity, with 0,7 for one end fixed and one end free.

(5) The factor k refers to end rotation on plan. It is analogous to the ratio ℓ/L for a compression member.

(6) The factor k_w refers to end warping. Unless special provision for warping fixity is made, k_w should be taken as 1,0.

(7) Values of C_1, C_2 and C_3 are given in [EC3: tables F.1.1 and F.1.2] for various load cases, as indicated by the shape of the bending moment diagram over the length L between lateral

restraints. Values are given corresponding to various values of k.

(8) *For cases with $k = 1{,}0$ the value of C_1 for any ratio of end moment loading as indicated in [EC3: table F.1.1], is given approximately by:*

$$C_1 = 1{,}88 - 1{,}40\,\psi + 0{,}52\psi^2 \qquad \text{but } C_1 \le 2{,}70 \tag{5.21}$$

5.3.1.2 Uniform cross-sections symmetrical about the minor axis

(1) Elastic critical moment is given by the general formula:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \left[\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + [C_2 z_g - C_3 z_j]^2 \right]^{0,5} - [C_2 z_g - C_3 z_j] \right\} \tag{5.22}$$

See 5.3.2.1 for details on how to determine k, C and z_g factors

(2) The general formula for deriving z_j is given by:

$$z_j = z_s - 0{,}5 \int_A (y^2 + z^2)\,z\,dA\,/\,I_y$$

The sign convention for determining z_j, see [EC3: figure F.1.1], is:

- z is positive for the compression flange
- z_j is positive when the flange with the larger value of I_z is in compression at the point of largest moment.

(3) *The following approximations for z_j can be used:*

when $\beta_f > 0{,}5$:

$$z_j = 0{,}8\,(2\beta_f - 1)\,h_s/2 \tag{5.23}$$

when $\beta_f < 0{,}5$:

$$z_j = 1{,}0\,(2\beta_f - 1)\,h_s/2 \tag{5.24}$$

(4) *for sections with a lipped compression flange:*

$$z_j = 0{,}8\,(2\beta_f - 1)(1 + h_L/h)\,h_s/2 \text{ when } \beta_f > 0{,}5 \tag{5.25}$$

$$z_j = 1{,}0\,(2\beta_f - 1)(1 + h_L/h)\,h_s/2 \text{ when } \beta_f < 0{,}5 \tag{5.26}$$

where h_L is the depth of the lip

5.3.2 Buckling resistance

5.3.2.1 Slenderness ratio

(1) The slenderness ratio $\bar{\lambda}_{LT}$ is given by:

$$\bar{\lambda}_{LT} = [\beta_W W_{pl.y} f_y / M_{cr}]^{0,5}$$

(2) If $\bar{\lambda}_{LT} \le 0{,}4$ no allowance for lateral-torsional buckling is necessary.

5.3.2.2 Reduction factor for lateral torsional buckling

(1) The value of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be determined from:

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \bar{\lambda}_{LT}^2]^{0,5}} \qquad \text{but } \chi_{LT} \le 1 \tag{5.27}$$

in which:

$$\phi_{LT} = 0{,}5\,[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0{,}2) + \bar{\lambda}_{LT}^2]$$

(2) *The imperfection factor α_{LT} should be taken as:*

0,21 for rolled sections
0,49 for welded sections

(3) *Values of the reduction factor χ_{LT} for the appropriate non-dimensional slenderness $\bar{\lambda}_{LT}$ may be obtained from table 5.4.2 with $\bar{\lambda} = \bar{\lambda}_{LT}$ and $\chi = \chi_{LT}$, using:*

- *curve a for rolled sections*
- *curve c for welded sections*

5.3.2.3 Buckling resistance

(1) The design buckling resistance moment shall be taken as:

$$M_{b.Rd} = \chi_{LT}\, \beta_W\, W_{pl.y} f_y/\gamma_{M1} \qquad (5.28)$$

where β_W = 1 for Class 1 or Class 2 cross-sections
β_W = $W_{el.y}/W_{pl.y}$ for Class 3 cross-sections
β_W = $W_{eff.y}/W_{pl.y}$ for Class 4 cross-sections

5.3.3 Shear

(1) Design as for laterally restrained beams, see 5.2.2

5.3.4 Resistance of webs to transverse forces

(1) Design as for laterally restrained beams, see section 5.2.3.

5.4 Struts and ties

5.4.1 Ties

(1) For members in axial tension, the design value of the tensile force N_{Sd} at each cross-section shall satisfy:

$$N_{Sd} \leq N_{t.Rd} \qquad (5.29)$$

where $N_{t.Rd}$ is the design tension resistance of the cross-section, taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{pl.Rd} = A f_y/\gamma_{M0}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u.Rd} = 0{,}9\, A_{net}\, f_u/\gamma_{M2}$$

(2) In Category C connections designed to be slip-resistant at the ultimate limit state [see EC3: 6.5.3.1], the design plastic resistance of the net section at holes for fasteners $N_{net.Rd}$ shall not be taken as more than:

$$N_{net.Rd} = A_{net}\, f_y/\gamma_{M0} \qquad (5.30)$$

(3) *For angles connected through one leg, see also [EC3: 6.5.2.3 and 6.6.10]. Similar consideration should also be given to other types of sections connected through outstands such as T-sections and channels.*

(4) Where ductile behaviour is required, the design plastic resistance $N_{pl.Rd}$ shall be less than the design ultimate resistance of the net section at fastener holes $N_{u.Rd}$, that is:

$$N_{u.Rd} \geq N_{pl.Rd} \tag{5.31}$$

This will be satisfied if:

$$0{,}9[A_{net}/A] \geq [f_y/f_u]\,[\gamma_{M2}/\gamma_{M0}]$$

5.4.2 Cross-section resistance of struts

(1) For members in axial compression, the design value of the compressive force N_{Sd} at each cross-section shall satisfy:

$$N_{Sd} \leq N_{c.Rd} \tag{5.32}$$

where $N_{c.Rd}$ is the design compression resistance of the cross-section, taken as the smaller of:

a) the design plastic resistance of the gross section

$N_{pl.Rd} = Af_y/\gamma_{M0}$

b) the design local buckling resistance of the gross section

$N_{o.Rd} = A_{eff}f_y/\gamma_{M1}$

where A_{eff} is the effective area of the cross-section, see 5.1.3.1.

(2) The design compression resistance of the cross-section $N_{c.Rd}$ may be determined as follows:

Class 1, 2 or 3 cross-sections: $N_{c.Rd} = Af_y/\gamma_{M0}$

Class 4 cross-sections: $N_{c.Rd} = A_{eff}f_y/\gamma_{M1}$

(3) In the case of unsymmetrical Class 4 sections, the method given in 5.5.1.3 should be used to allow for the additional moment ΔM due to the eccentricity of the centroidal axis of the effective section, see 5.1.3.1.(7).

(4) Fastener holes need not be allowed for in compression members, except for oversize and slotted holes.

5.4.3 Buckling resistance of struts

5.4.3.1 General

(1) The design buckling resistance of a compression member shall be taken as:

$$N_{b.Rd} = \chi\,\beta_A A f_y/\gamma_{M1} \tag{5.33}$$

where $\beta_A = 1$ for Class 1, 2 or 3 cross-sections

$\beta_A = A_{eff}/A$ for Class 4 cross-sections

and χ is the reduction factor for the relevant buckling mode.

(2) For hot rolled steel members with the types of cross-section commonly used for compression members, the relevant buckling mode is generally "flexural" buckling.

(3) In some cases the "torsional" or "flexural-torsional" modes may govern. Reference may be

made to ENV 1993-1-3 Eurocode 3: Part 1.3[*].

5.4.3.2 Uniform members

(1) For constant axial compression in members of constant cross-section, the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$, may be determined from:

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} \quad \text{but } \chi \leq 1 \tag{5.34}$$

where $\phi = 0,5\,[1 + \alpha(\bar{\lambda} - 0,2) + \bar{\lambda}^2]$

α is an imperfection factor

$\bar{\lambda} = [\beta_A\, Af_y/N_{cr}]^{0,5} = (\lambda/\lambda_1)\,[\beta_A\,]^{0,5}$

λ is the slenderness for the relevant buckling mode

$\lambda_1 = \pi\,[E/f_y]^{0,5} = 93,9\varepsilon$

$\varepsilon = [235/f_y]^{0,5}$ (f_y in N/mm^2)

and N_{cr} is the elastic critical force for the relevant buckling mode.

(2) The imperfection factor α corresponding to the appropriate buckling curve shall be obtained from table 5.4.1.

Table 5.4.1 Imperfection factors

Bucking curve	a	b	c	d
Imperfection factor α	0,21	0,34	0,49	0,76

(3) Values of the reduction factor χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ may be obtained from table 5.4.2.

(4) Alternatively, uniform members may be verified using second-order analysis, see 5.4.3.3 (4) and 5.4.3.3 (6).

5.4.3.3 Non-uniform members

(1) Tapered members and members with changes of cross-section within their length may be verified using second-order analysis, see (4) and (6).

(2) Alternatively, simplified methods of analysis may be based on modifications of the basic procedure for uniform members.

(3) No one method is preferred. Any recognised method may be used provided that it can be demonstrated to be conservative.

(4) Second-order analysis of a member shall incorporate the appropriate equivalent initial bow imperfection given in figure 5.4.1 corresponding to the relevant buckling curve, depending on the method of analysis and type of cross-section verification.

(5) The equivalent initial bow imperfections given in figure 5.4.1 shall also be used where it is necessary (according to 4.1.4.5) to include member imperfections in the global analysis.

(6) When the imperfections given in figure 5.4.1 are used, the resistances of the cross-sections shall be verified, but using γ_{M1} in place of γ_{M0}.

Table 5.4.2 Reduction factors χ

$\bar{\lambda}$	Buckling curve			
	a	b	c	d
0,2	1,0000	1,0000	1,0000	1,0000
0,3	0,9775	0,9641	0,9491	0,9235
0,4	0,9528	0,9261	0,8973	0,8504
0,5	0,9243	0,8842	0,8430	0,7793
0,6	0,8900	0,8371	0,7854	0,7100
0,7	0,8477	0,7837	0,7247	0,6431
0,8	0,7957	0,7245	0,6622	0,5797
0,9	0,7339	0,6612	0,5998	0,5208
1,0	0,6656	0,5970	0,5399	0,4671
1,1	0,5960	0,5352	0,4842	0,4189
1,2	0,5300	0,4781	0,4338	0,3762
1,3	0,4703	0,4269	0,3888	0,3385
1,4	0,4179	0,3817	0,3492	0,3055
1,5	0,3724	0,3422	0,3145	0,2766
1,6	0,3332	0,3079	0,2842	0,2512
1,7	0,2994	0,2781	0,2577	0,2289
1,8	0,2702	0,2521	0,2345	0,2093
1,9	0,2449	0,2294	0,2141	0,1920
2,0	0,2229	0,2095	0,1962	0,1766
2,1	0,2036	0,1920	0,1803	0,1630
2,2	0,1867	0,1765	0,1662	0,1508
2,3	0,1717	0,1628	0,1537	0,1399
2,4	0,1585	0,1506	0,1425	0,1302
2,5	0,1467	0,1397	0,1325	0,1214
2,6	0,1362	0,1299	0,1234	0,1134
2,7	0,1267	0,1211	0,1153	0,1062
2,8	0,1182	0,1132	0,1079	0,0997
2,9	0,1105	0,1060	0,1012	0,0937
3,0	0,1036	0,0994	0,0951	0,0882

5.4.3.4 Flexural buckling

(1) For flexural buckling the appropriate buckling curve shall be determined from [EC3: table 5.5.3].

(2) Sections not contained in [EC3: table 5.5.3] shall be classified analogously.

(3) The slenderness λ shall be taken as follows:

$$\lambda = \ell / i \tag{5.25}$$

where i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section.

(4) Cold formed structural hollow sections shall be verified using either:

(a) the basic yield strength f_{yb} of the flat sheet material out of which the member is madeby cold-forming, with buckling curve b.

(b) The average yield strength f_{ya} of the member after cold-forming, determined in conformity with the definition given in figure 5.4.2, with buckling curve c.

5.4.3.5 Buckling length

(1) The buckling length ℓ of a compression member with both ends effectively held in position laterally, may conservatively be taken as equal to its system length L.

(2) Alternatively the buckling length ℓ may be determined using 5.6.6.

Cross-section		Method of global analysis	
Method used to verify resistance	Section type and axis	Elastic or Rigid - Plastic or Elastic - Perfectly plastic	Elasto-plastic (plastic zone method)
Elastic [5.4.8.2]	Any	$\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{el}/A$	-
Linear plastic [5.4.8.1(12)]	Any	$\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{pl}/A$	-
Non-linear plastic [5.4.8.1(1) to (11)]	I-section yy-axis	$1{,}33\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{pl}/A$	$\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{pl}/A$
	I-section zz-axis	$2{,}0\, k_\gamma e_{eff}/\varepsilon$	$k_\gamma e_{eff}/\varepsilon$
	Rectangular hollow section	$1{,}33\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{pl}/A$	$\alpha(\bar{\lambda} - 0{,}2)\, k_\gamma W_{pl}/A$
	Circular hollow section	$1{,}5\, k_\gamma e_{eff}/\varepsilon$	$k_\gamma e_{eff}/\varepsilon$

$k_\gamma = (1 - k_\delta) + 2\, k_\delta \bar{\lambda}$ but $k_\gamma \geq 1{,}0$

Buckling curve	α	e_{eff}	k_δ			
			$\gamma_{M1} = 1{,}05$	$\gamma_{M1} = 1{,}10$	$\gamma_{M1} = 1{,}15$	$\gamma_{M1} = 1{,}20$
a	0,21	$\ell/600$	0,12	0,23	0,33	0,42
b	0,34	$\ell/380$	0,08	0,15	0,22	0,28
c	0,49	$\ell/270$	0,06	0,11	0,16	0,20
d	0,76	$\ell/180$	0,04	0,08	0,11	0,14

Non-uniform members:

Use value of W_{el}/A or W_{pl}/A at centre of buckling length ℓ

Figure 5.4.1 Design values of equivalent initial bow imperfection $e_{o,d}$

Average yield strength:

The average yield strength f_{ya} may be determined from full size section tests or as follows:

$$f_{ya} = f_{yb} + (knt^2 / A_g)(f_u - f_{yb})$$

where:

f_{yb} is the tensile yield strength of the basic material as defined below (N/mm²)

f_u is the tensile ultimate strength of the basic material (N/mm²)

t is the material thickness (mm)

A_g is the gross cross-sectional area (mm²)

k is a coefficient depending on the type of forming:

- $k = 7$ for cold rolling
- $k = 5$ for other methods of forming

n is the number of 90° bends in the section with an internal radius < 5t (fractions of 90° bends should be counted as fractions of n)

and f_{ya} should not exceed f_u or $1{,}2\ f_{yb}$

The increase in yield strength due to cold working should not be utilised for members which are welded, annealed, galvanised (after forming) or subject to heat treatment after forming which may produce softening.

Basic material:

Basic material is the flat sheet material out of which sections are made by cold-forming.

Figure 5.4.2 Average yield strength f_{ya} of cold formed structural hollow sections

5.5 Columns

5.5.1 Cross-section resistance to bending and axial force

5.5.1.1 Class 1 and 2 cross-sections

(1) For class 1 and 2 cross-sections, the criterion to be satisfied in the absence of shear force is:

$$M_{Sd} \leq M_{N.Rd} \tag{5.36}$$

where $M_{N.Rd}$ is the reduced design plastic resistance moment allowing for the axial force.

(2) For a plate without bolt holes, the reduced design plastic resistance moment is given by:

$$M_{N.Rd} = M_{pl.Rd}\,[1 - (N_{Sd}/N_{pl.Rd})^2]$$

and the criterion becomes:

$$\frac{M_{Sd}}{M_{pl.Rd}} + \left[\frac{N_{Sd}}{N_{pl.Rd}}\right]^2 \leq 1 \qquad (5.37)$$

(3) In flanged sections, the reduction of the theoretical plastic resistance moment by the presence of small axial forces is counter-balanced by strain hardening and may be neglected. However, for bending about the y-y-axis, allowance shall be made for the effect of the axial force on the plastic resistance moment when the axial force exceeds half the plastic tension resistance of the web, or a quarter of the plastic tension resistance of the cross-section, whichever is smaller. Similarly, for bending about the z-z-axis, allowance shall be made for the effect of the axial force when it exceeds the plastic tension resistance of the web.

(4) For cross-sections without bolt holes, the following approximations may be used for standard rolled I or H sections:

$$M_{Ny.Rd} = M_{pl.y.Rd}(1 - n)/(1 - 0{,}5a) \quad \text{but } M_{Ny.Rd} \leq M_{pl.y.Rd} \qquad (5.38)$$

for $n \leq a$: $\quad M_{Nz.Rd} = M_{pl.z.Rd}$

for $n > a$: $$M_{Nz.Rd} = M_{pl.z.Rd}\left[1 - \left[\frac{n - a}{1 - a}\right]^2\right] \qquad (5.39)$$

where $n = N_{Sd} / N_{pl.Rd}$

and $a = (A\text{-}2bt_f)/A$ but $a \leq 0{,}5$

(5) The expressions given in (4) may also be used for welded I or H sections with equal flanges.

(6) The approximations given in (4) may be further simplified (for standard rolled I or H sections only) to:

$$M_{Ny.Rd} = 1{,}11 M_{pl.y.Rd}(1 - n) \quad \text{but } M_{Ny.Rd} \leq M_{pl.y.Rd} \qquad (5.40)$$

for $n \leq 0{,}2$: $\quad M_{Nz.Rd} = M_{pl.z.Rd}$

for $n > 0{,}2$: $$M_{Nz.Rd} = 1{,}56 M_{pl.z.Rd}(1 - n)(n + 0{,}6) \qquad (5.41)$$

(7) For cross-sections without bolt holes, the following approximations may be used for rectangular structural hollow sections of uniform thickness:

$$M_{Ny.Rd} = M_{pl.y.Rd}(1 - n)/(1 - 0{,}5a_w) \quad \text{but } M_{Ny.Rd} \leq M_{pl.y.Rd} \qquad (5.42)$$

$$M_{Nz.Rd} = M_{pl.z.Rd}(1 - n)/(1 - 0{,}5a_f) \quad \text{but } M_{Nz.Rd} \leq M_{pl.z.Rd} \qquad (5.43)$$

where $a_w = (A - 2bt)/A$ but $a_w \leq 0{,}5$

and $a_f = (A - 2ht)/A$

(8) The expressions given in (7) may also be used for welded box sections with equal flanges and equal webs, by taking:

$a_w = (A\text{-}2bt_f)/A$ but $a_w \leq 0{,}5$

$a_f = (A\text{-}2ht_w)/A$ but $a_f \leq 0{,}5$

(9) The approximations given in (7) may be further simplified for standard rectangular structural hollow sections of uniform thickness, as follows:

- *for a square section:*

$$M_{N.Rd} = 1{,}26 M_{pl.Rd}(1 - n) \quad \text{but } M_{N.Rd} \leq M_{pl.Rd} \qquad (5.44)$$

- *for a rectangular section:*

$$M_{Ny.Rd} = 1{,}33 M_{pl.y.Rd}\,(1 - n) \quad \text{but } M_{Ny.Rd} \leq M_{pl.y.Rd} \tag{5.45}$$

$$M_{Nz.Rd} = M_{pl.z.Rd}\,(1 - n)/(0{,}5 + ht/A) \quad \text{but } M_{Nz.Rd} \leq M_{pl.z.Rd} \tag{5.46}$$

(10) *For cross-sections without bolt holes, the following approximation may be used for circular tubes of uniform thickness:*

$$M_{N.Rd} = 1{,}04 M_{pl.Rd}\,(1 - n^{1,7}) \quad \text{but } M_{N.Rd} \leq M_{pl.Rd} \tag{5.47}$$

(11) *For bi-axial bending the following approximate criterion may be used:*

$$\left[\frac{M_{y.Sd}}{M_{Ny.Rd}}\right]^{\alpha} + \left[\frac{M_{z.Sd}}{M_{Nz.Rd}}\right]^{\beta} \leq 1 \tag{5.48}$$

in which α *and* β *are constants, which may conservatively be taken as unity, otherwise as follows:*

- *I and H sections:*

$$\alpha = 2\,;\ \beta = 5n \quad \text{but } \beta \geq 1$$

- *circular tubes:*

$$\alpha = 2\,;\ \beta = 2$$

- *rectangular hollow sections:*

$$\alpha = \beta = \frac{1{,}66}{1 - 1{,}13n^2} \quad \text{but } \alpha = \beta \leq 6$$

- *solid rectangles and plates:*

$$\alpha = \beta = 1{,}73 + 1{,}8n^3$$

where $n = N_{Sd}/N_{pl.Rd}$

(12) *As a further conservative approximation, the following criterion may be used:*

$$\frac{N_{Sd}}{N_{pl.Rd}} + \frac{M_{y.Sd}}{M_{pl.y.Rd}} + \frac{M_{z.Sd}}{M_{pl.z.Rd}} \leq 1 \tag{5.49}$$

5.5.1.2 Class 3 cross-sections

(1) In the absence of shear force, Class 3 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x.Ed}$ satisfies the criterion:

$$\sigma_{x.Ed} \leq f_{yd} \tag{5.50}$$

where $f_{yd} = f_y/\gamma_{M0}$

(2) *For cross-sections without holes for fasteners, the above criterion becomes:*

$$\frac{N_{Sd}}{A f_{yd}} + \frac{M_{y.Sd}}{W_{el.y}\,f_{yd}} + \frac{M_{z.Sd}}{W_{el.z}\,f_{yd}} \leq 1 \tag{5.51}$$

5.5.1.3 Class 4 cross-sections

(1) In the absence of shear force, Class 4 cross-sections will be satisfactory if the maximum longitudinal stress $\sigma_{x.Ed}$ calculated using the effective widths of the compression elements (see 5.1.3.1.(2)) satisfies the criterion:

$$\sigma_{x.Ed} \leq f_{yd} \qquad (5.52)$$

where $f_{yd} = f_y/\gamma_{M1}$

(2) For cross-sections without holes for fasteners, the above criterion becomes:

$$\frac{N_{Sd}}{A_{eff}\, f_{yd}} + \frac{M_{y.Sd} + N_{Sd}\, e_{Ny}}{W_{eff.y}\, f_{yd}} + \frac{M_{z.Sd} + N_{Sd}\, e_{Nz}}{W_{eff.z}\, f_{yd}} \leq 1 \qquad (5.53)$$

where A_{eff} is the effective area of the cross-section when subject to uniform compression.

W_{eff} is the effective section modulus of the cross-section when subject only to moment about the relevant axis.

e_N is the shift of the relevant centroidal axis when the cross-section is subject to uniform compression, see 5.1.3.2(7).

5.5.2 Cross-section resistance to bending, shear and axial force

(1) When the shear force exceeds half the plastic shear resistance, allowance shall be made for the effect of both shear force and axial force on the reduced plastic resistance moment.

(2) Provided that the design value of the shear force V_{Sd} does not exceed 50% of the design plastic shear resistance V_{plRd} no reduction need be made in combinations of moment and axial force that meet the criteria in 5.5.1.

(3) When V_{Sd} exceeds 50% of V_{plRd} the design resistance of the cross-section to combinations of moment and axial force should be calculated using a reduced yield strength $(1 - \rho)f_y$ for the shear area, where $\rho = (2V_{Sd} / V_{plRd} - 1)^2$.

5.5.3 Buckling resistance with tension and moments

(1) Members subject to combined bending and axial tension shall be checked for resistance to lateral-torsional buckling, treating the axial force and bending moment as a vectorial effect, see 4.1.5.5.1(4).

(2) Where the axial force and bending moment can vary independently, the design value of the axial tension should be multiplied by a reduction factor for vectorial effects:

$$\psi_{vec} = \boxed{0,8}$$

(3) The net calculated stress $\sigma_{com.Ed}$ (which can exceed f_y) in the extreme compression fibre due to the vectorial effects should be determined from:

$$\sigma_{com.Ed} = M_{Sd} / W_{com} - \psi_{vec}\, N_{t.Sd} / A \qquad (5.54)$$

where W_{com} is the elastic section modulus for the extreme compression fibre

and $N_{t.Sd}$ is the design value of the axial tension

(4) The verification should be carried out using an effective design internal moment $M_{eff.Sd}$ obtained

from:

$$M_{eff.Sd} = W_{com}\sigma_{com.Ed}$$

(5) *The design buckling resistance moment $M_{b.Rd}$ should be obtained using section 5.3.*

5.5.4 Buckling resistance with compression and moments

5.5.4.1 General

5.5.4.1.1 Equivalent uniform moments

(1) The equivalent uniform moment factors $\beta_{M.y}$, $\beta_{M.z}$ and $\beta_{M.LT}$ shall be obtained from [EC3: figure 5.5.3] according to the shape of the bending moment diagram between the relevant braced points as follows:

factor:	moment about axis:	points braced in direction:
$\beta_{M.y}$	y-y	z-z
$\beta_{M.z}$	z-z	y-y
$\beta_{M.LT}$	y-y	y-y

5.5.4.1.2 Buckling length

(1) The buckling length ℓ of a compression member is the length of an otherwise similar member with "pinned ends" (ends restrained against lateral movement but free to rotate in the plane of buckling) which has the same buckling resistance.

(2) In the absence of better information, the theoretical buckling length for elastic critical buckling may conservatively be adopted.

(3) An equivalent buckling length may be used to relate the buckling resistance of a member subject to non-uniform loading to that of an otherwise similar member subject to uniform loading.

(4) An equivalent buckling length may also be used to relate the buckling resistance of a non-uniform member to that of a uniform member under similar conditions of loading and restraint.

5.5.4.1.2 Buckling length for columns in building frames

(1) The buckling length ℓ of a column in a non-sway mode may be obtained from [EC3: figure E.2.1].

(2) The buckling length ℓ of a column in a sway mode may be obtained from [EC3: figure E.2.2].

(3) For the theoretical models shown in [EC3: figure E.2.3] the distribution factors η_1 and η_2 are obtained from:

$$\eta_1 = K_c/(K_c + K_{11} + K_{12}) \qquad (5.55)$$

$$\eta_2 = K_c/(K_c + K_{21} + K_{22}) \qquad (5.56)$$

where K_c is the column stiffness coefficient I/L

and K_{ij} is the effective beam stiffness coefficient

(4) These models may be adapted to the design of continuous column, by assuming that each length of column is loaded to the same value of the ratio (N/N_{cr}). In the general case where (N/N_{cr}) varies, this leads to a conservative value of ℓ/L for the most critical length of column.

(5) For each length of a continuous column the assumption made in (4) may be introduced by using the model shown in [EC3: figure E.2.4] and obtaining the distribution factors η_1 and η_2 from:

$$\eta_1 = \frac{K_c + K_1}{K_c + K_1 + K_{11} + K_{12}} \qquad (5.57)$$

$$\eta_2 = \frac{K_c + K_2}{K_c + K_2 + K_{21} + K_{22}} \qquad (5.58)$$

where K_1 and K_2 are the stiffness coefficients for the adjacent lengths of column.

(6) Where the beams are not subject to axial forces, their effective stiffness coefficients may be determined by reference to table 5.5.1, provided that they remain elastic under the design moments.

Table 5.5.1 Effective stiffness coefficient for a beam

Conditions of rotational restraint at far end of beam	Effective beam stiffness coefficient K (provided that beam remains elastic)
Fixed at far end	1,0 I/L
Pinned at far end	0,75 I/L
Rotation as at near end (double curvature)	1,5 I/L
Rotation equal and opposite to that at near end (single curvature)	0,5 I/L
General case. Rotation θ_a at near end and θ_b at far end	$(1 + 0,5\ \theta_b / \theta_a)$ I/L

(7) *For building frames with concrete floor slabs, provided that the frame is of regular layout and the loading is uniform, it is normally sufficiently accurate to assume that the effective stiffness coefficients of the beams are as shown in table 5.5.2.*

Table 5.5.2 Effective stiffness coefficient for a beam in a building frame with concrete floor slabs.

Loading conditions for the beam	Non-sway mode	Sway mode
Beams directly supporting concrete floor slabs	1,0 I/L	1,0 I/L
Other beams with direct loads	0,75 I/L	1,0 I/L
Beams with end moments only	0,5 I/L	1,5 I/L

(8) *Where, for the same load case, the design moment in any of the beams exceeds $W_e f_y / \gamma_{M0}$, the beam should be assumed to be pinned at the point or points concerned.*

(9) *Where a beam has nominally pinned connections, it should be assumed to be pinned at the point or points concerned.*

(10) *Where a beam has semi-rigid connections, its effective stiffness coefficient should be reduced accordingly.*

(11) *Where the beams are subject to axial forces, their effective stiffness coefficients should be adjusted accordingly. Stability functions may be used. As a simple alternative, the increased stiffness coefficient due to axial tension may be neglected and the effects of axial compression may be allowed for by using the conservative approximations given in table 5.5.3.*

Table 5.5.3 *Approximate formulae for reduced beam stiffness coefficients due to axial compression*

Conditions of rotational restraint at far end of beam	*Effective beam stiffness coefficient K (provided that beam remains elastic)*
Fixed	$1{,}0\ I/L\ (1 - 0{,}4\ N/N_E)$
Pinned	$0{,}75\ I/L\ (1 - 1{,}0\ N/N_E)$
Rotation as at near end (double curvature)	$1{,}5\ I/L\ (1 - 0{,}2\ N/N_E)$
Rotation equal and opposite to that at near end (single curvature)	$0{,}5\ I/L\ (1 - 1{,}0\ N/N_E)$
In this table $N_E = \pi^2 EI/L^2$	

(12) *The following empirical expressions may be used as conservative approximations instead of reading values from [EC3: figures E.2.1 and E.2.2]:*

(a) ***non-sway mode*** *[EC3: figure E.2.1]*

$$\ell/L = 0{,}5 + 0{,}14\,(\eta_1 + \eta_2) + 0{,}055\,(\eta_1 + \eta_2)^2 \qquad (5.59)$$

or alternatively:

$$\ell/L = \left[\frac{1 + 0{,}145\,(\eta_1 + \eta_2) - 0{,}265\,\eta_1\,\eta_2}{2 - 0{,}364\,(\eta_1 + \eta_2) - 0{,}247\,\eta_1\,\eta_2}\right] \qquad (5.60)$$

(b) ***sway mode*** *[EC3: figure E.2.2]*

$$\ell/L = \left[\frac{1 - 0{,}2\,(\eta_1 + \eta_2) - 0{,}12\,\eta_1\,\eta_2}{1 - 0{,}8\,(\eta_1 + \eta_2) + 0{,}6\,\eta_1\,\eta_2}\right]^{0{,}5} \qquad (5.61)$$

Table 3.2 Maximum thickness for statically loaded structural elements without reference to informative Annex C

Steel grade and quality	Maximum thickness (mm) for lowest service temperature of					
	0°C		-10°C		-20°C	
Service condition	*S1*	*S2*	*S1*	*S2*	*S1*	*S2*
EN 10025 (1):						
Fe 360 B	150	41	108	30	74	22
Fe 360 C	250	110	250	75	187	53
Fe 360 D	250	250	250	212	250	150
Fe 430 B	90	26	63	19	45	14
Fe 430 C	250	63	150	45	123	33
Fe 430 D	250	150	250	127	250	84
Fe 510 B	40	12	29	9	21	6
Fe 510 C	106	29	73	21	52	16
Fe 510 D	250	73	177	52	150	38
Fe 510 DD(2)	250	128	250	85	250	59
prEN 10113:(3)						
Fe E 275 KG(4)	250	250	250	192	250	150
Fe E 275 KT	250	250	250	250	250	250
Fe E 355 KG(4)	250	128	250	85	250	59
Fe E 355 KT	250	250	250	250	250	150

Service conditions (5):

S1 Either:

- non-welded, or
- in compression

S2 As welded, in tension

In both cases this table assumes loading rate R1 and consequences of failure condition C2, see informative Annex C.

Notes:

(1) For rolled sections over 100 mm thick, the minimum Charpy V-notch energy specified in EN 10025 is subject to agreement. For thicknesses up to 150 mm, a minimum value of 27 J at the relevant specified test temperature is required and 23 J for thicknesses over 150 mm up to 250 mm.

(2) For steel grade Fe 510 DD to EN 10025, the specified minimum Charpy V-notch energy value is 40J at -20°C. The entries in this row assume an equivalent value of 27 J at -30°C.

(3) For steels of delivery condition N to prEN 10113-2 over 150 mm thick and for steels of delivery condition TM to prEN 10113-3 over 150 mm thick for long products and over 63 mm thick for flat products, the minimum Charpy V-notch energy specified in prEN 10113 is subject to agreement.
For thicknesses up to 150 mm, a minimum value of 27 J is required and 23 J for thicknesses over 150 mm up to 250 mm. The test temperature should be -30°C for KG quality steel and -50°C for KT quality steel.

(4) For steel of quality KG to prEN 10113, the specified minimum values of Charpy V-notch energy go down to 40 J at -20°C. The entries in this row assume an equivalent value of 27 J at -30°C.

(5) For full details of service conditions, refer to informative Annex C.

5.5.4.2 Class 1 and 2 cross-sections

5.5.4.2.1 Members not subject to lateral-torsional buckling

(1) Members shall satisfy:

$$\frac{N_{Sd}}{\chi_{min} A f_y/\gamma_{M1}} + \frac{k_y M_{y.Sd}}{W_{pl.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{pl.z} f_y/\gamma_{M1}} \leq 1 \tag{5.62}$$

in which $k_y = 1 - \dfrac{\mu_y N_{Sd}}{\chi_y A f_y}$ but $k_y \leq 1{,}5$

$$\mu_y = \bar{\lambda}_y (2\beta_{My} - 4) + \left[\frac{W_{pl.y} - W_{el.y}}{W_{el.y}}\right] \quad \text{but } \mu_y \leq 0{,}90$$

$$k_z = 1 - \frac{\mu_z N_{Sd}}{\chi_z A f_y} \quad \text{but } k_z \leq 1{,}5$$

$$\mu_z = \bar{\lambda}_z (2\beta_{Mz} - 4) + \left[\frac{W_{pl.z} - W_{el.z}}{W_{el.z}}\right] \quad \text{but } \mu_z \leq 0{,}90$$

χ_{min} is the lesser of χ_y and χ_z

where χ_y and χ_z are the reduction factors from 5.4.3 for the y-y and z-z axes respectively

and β_{My} and β_{Mz} are equivalent uniform moment factors for flexural buckling, see (7).

5.5.4.2.2 Members subject to lateral-torsional buckling

(1) Members shall satisfy the following expression in addition to 5.5.4.2.1 requirements:

$$\frac{N_{Sd}}{\chi_z A f_y/\gamma_{M1}} + \frac{k_{LT} M_{y.Sd}}{\chi_{LT} W_{pl.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{pl.z} f_y/\gamma_{M1}} \leq 1 \tag{5.63}$$

in which $k_{LT} = 1 - \dfrac{\mu_{LT} N_{Sd}}{\chi_z A f_y}$ but $k_{LT} \leq 1$

$$\mu_{LT} = 0{,}15\, \bar{\lambda}_z\, \beta_{M.LT} - 0{,}15 \quad \text{but } \mu_{LT} \leq 0{,}90$$

where $\beta_{M.LT}$ is an equivalent uniform moment factor for lateral-torsional buckling, see (7).

5.5.4.3 Class 3 cross-sections

5.5.4.3.1 Members not subject to lateral-torsional buckling

(1) Members shall satisfy:

$$\frac{N_{Sd}}{\chi_{min} A f_y/\gamma_{M1}} + \frac{k_y M_{y.Sd}}{W_{el.y} f_y/\gamma_{M1}} + \frac{k_z M_{z.Sd}}{W_{el.z} f_y/\gamma_{M1}} \leq 1 \tag{5.64}$$

where k_y, k_z and χ_{min} are as in (1)

$$\mu_y = \bar{\lambda}_y (2\, \beta_{My} - 4) \quad \text{but } \mu_y \leq 0{,}90$$

and $\mu_z = \bar{\lambda}_z (2\, \beta_{Mz} - 4)$ but $\mu_z \leq 0{,}90$

5.5.4.3.2 Members subject to lateral-torsional buckling

(1) Members shall satisfy the following expression in addition to 5.5.4.3.1 requirements:

$$\frac{N_{Sd}}{\chi_z\, A\, f_y/\gamma_{M1}} + \frac{k_{LT}\, M_{y.Sd}}{\chi_{LT}\, W_{el.y}\, f_y/\gamma_{M1}} + \frac{k_z\, M_{z.Sd}}{W_{el.z}\, f_y/\gamma_{M1}} \leq 1 \qquad (5.65)$$

5.5.4.4 Class 4 cross-sections

5.5.4.4.1 Members not subject to lateral-torsional buckling

(1) Members shall satisfy:

$$\frac{N_{Sd}}{\chi_{min}\, A_{eff}\, f_y/\gamma_{M1}} + \frac{k_y(M_{y.Sd} + N_{Sd}\, e_{Ny})}{W_{eff.y}\, f_y/\gamma_{M1}} + \frac{k_z(M_{z.Sd} + N_{Sd}\, e_{Nz})}{W_{eff.z}\, f_y/\gamma_{M1}} \leq 1 \qquad (5.66)$$

where k_y, k_z and χ_{min} are as in (1), but using A_{eff} instead of A, see 5.1.3.2(9)

μ_y and μ_z are as in (3), but adding $N_{Sd}\, e_N$ to M_{Sd} when determining β

and A_{eff}, $W_{eff.y}$, $W_{eff.z}$, e_{Ny} and e_{Nz} are as in 5.5.1.3.

5.5.4.4.2 Members subject to lateral-torsional buckling

(1) Members shall satisfy the following expression in addition to 5.5.4.4.1 requirements:

$$\frac{N_{Sd}}{\chi_z\, A_{eff}\, f_y/\gamma_{M1}} + \frac{k_{LT}(M_{y.Sd} + N_{Sd}\, e_{Ny})}{\chi_{LT}\, W_{eff.y}\, f_y/\gamma_{M1}} + \frac{k_z(M_{z.Sd} + N_{Sd}\, e_{Nz})}{W_{eff.z}\, f_y/\gamma_{M1}} \leq 1 \qquad (5.67)$$

where k_{LT} is as in (2), but using A_{eff} instead of A, see 5.1.3.2(9)

and μ_{LT} is as in (2), but adding $N_{Sd}\, e_{Ny}$ to $M_{y.Sd}$ when determining $\beta_{M.LT}$

Appendix 2

MEASUREMENTS OF MATERIAL AND GEOMETRIC PROPERTIES

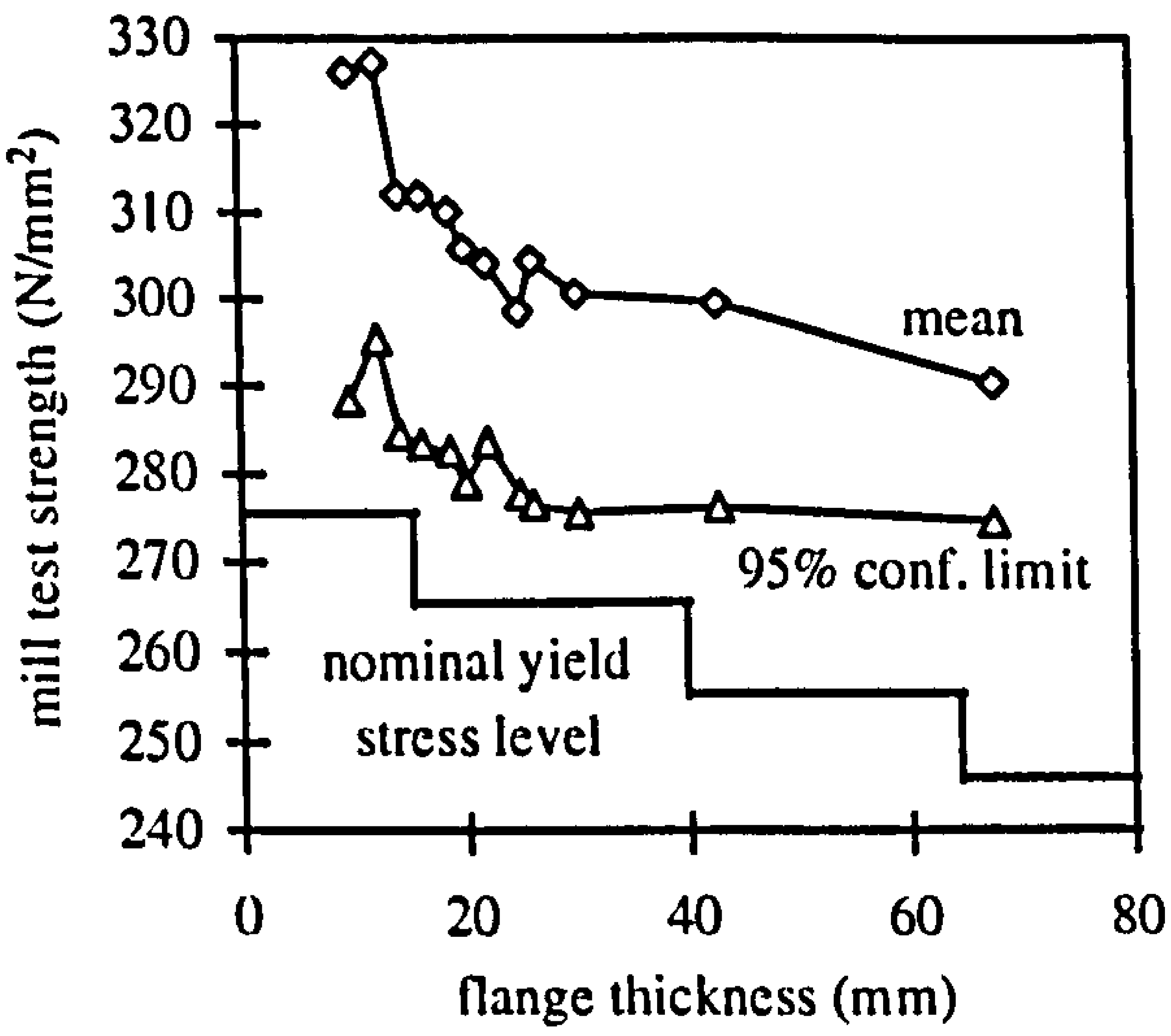

Fig. A2.1: Mill test vs. t_f
S275-A-ROS-4095

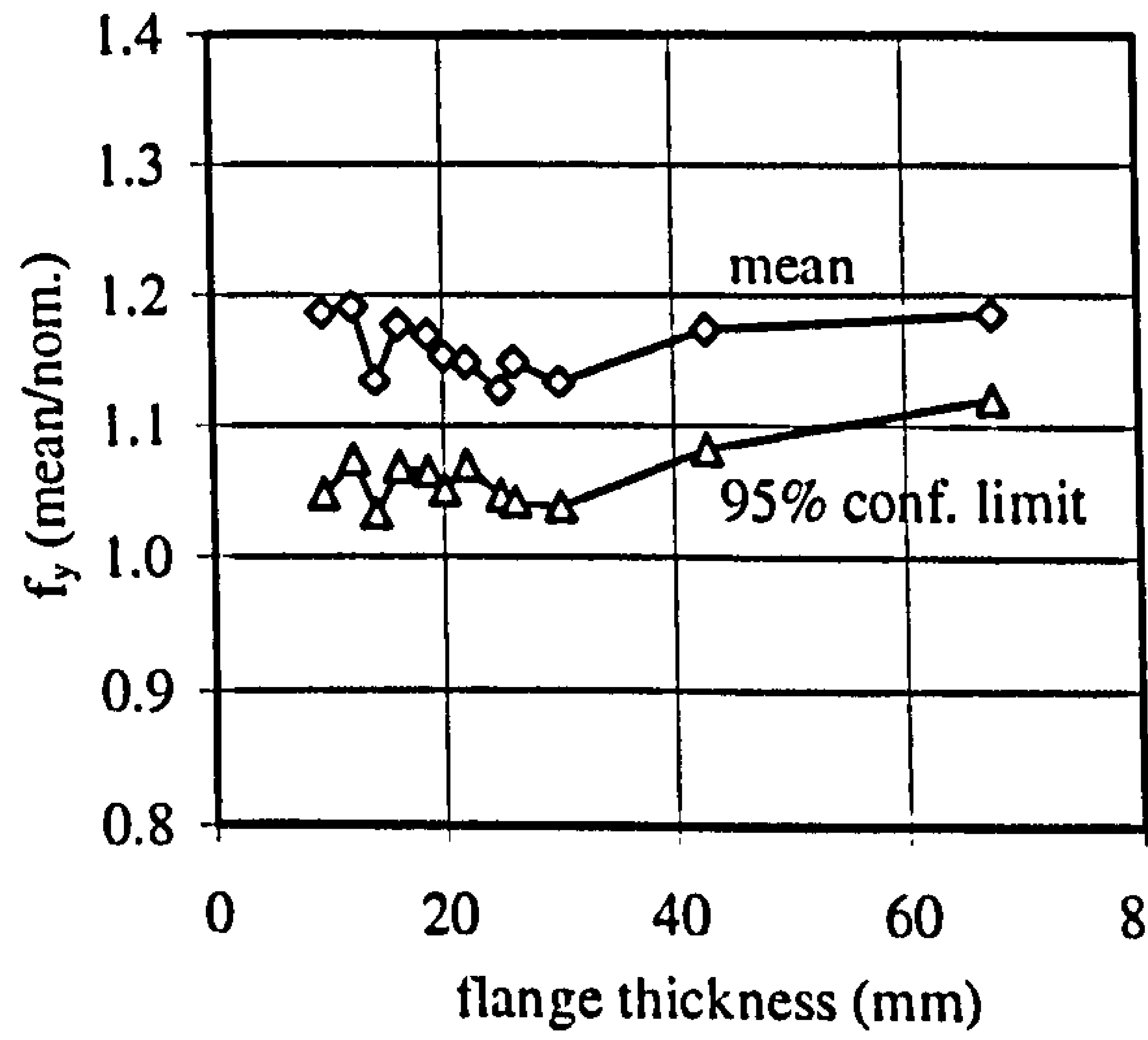

Fig. A2.2: Normalized mill test vs. t_f
S275-A-ROS-4095

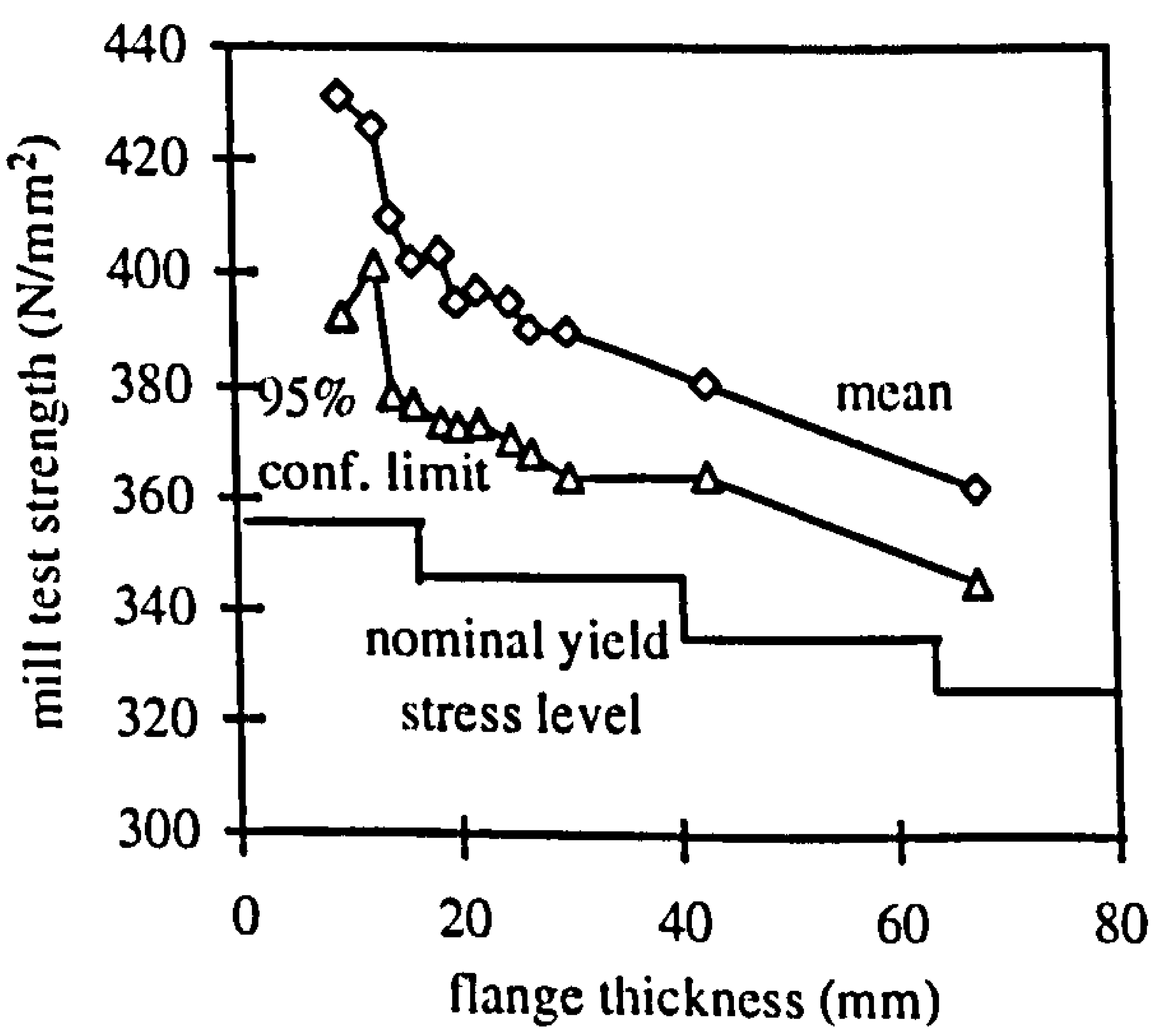

Fig. A2.3: Mill test vs. t_f
S355-A-ROS-1914

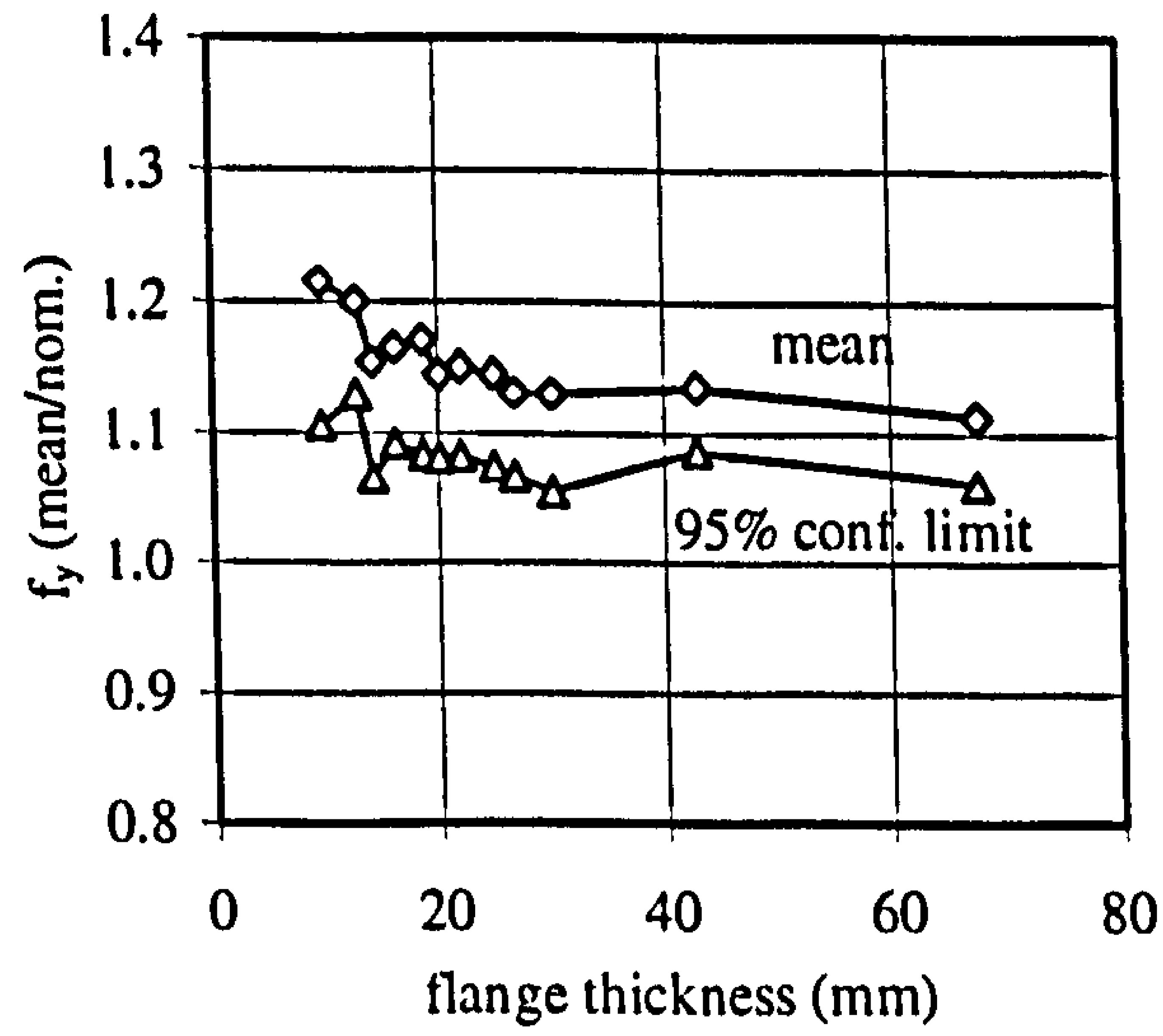

Fig. A2.4: Normalized mill test vs. t_f
S355-A-ROS-1914

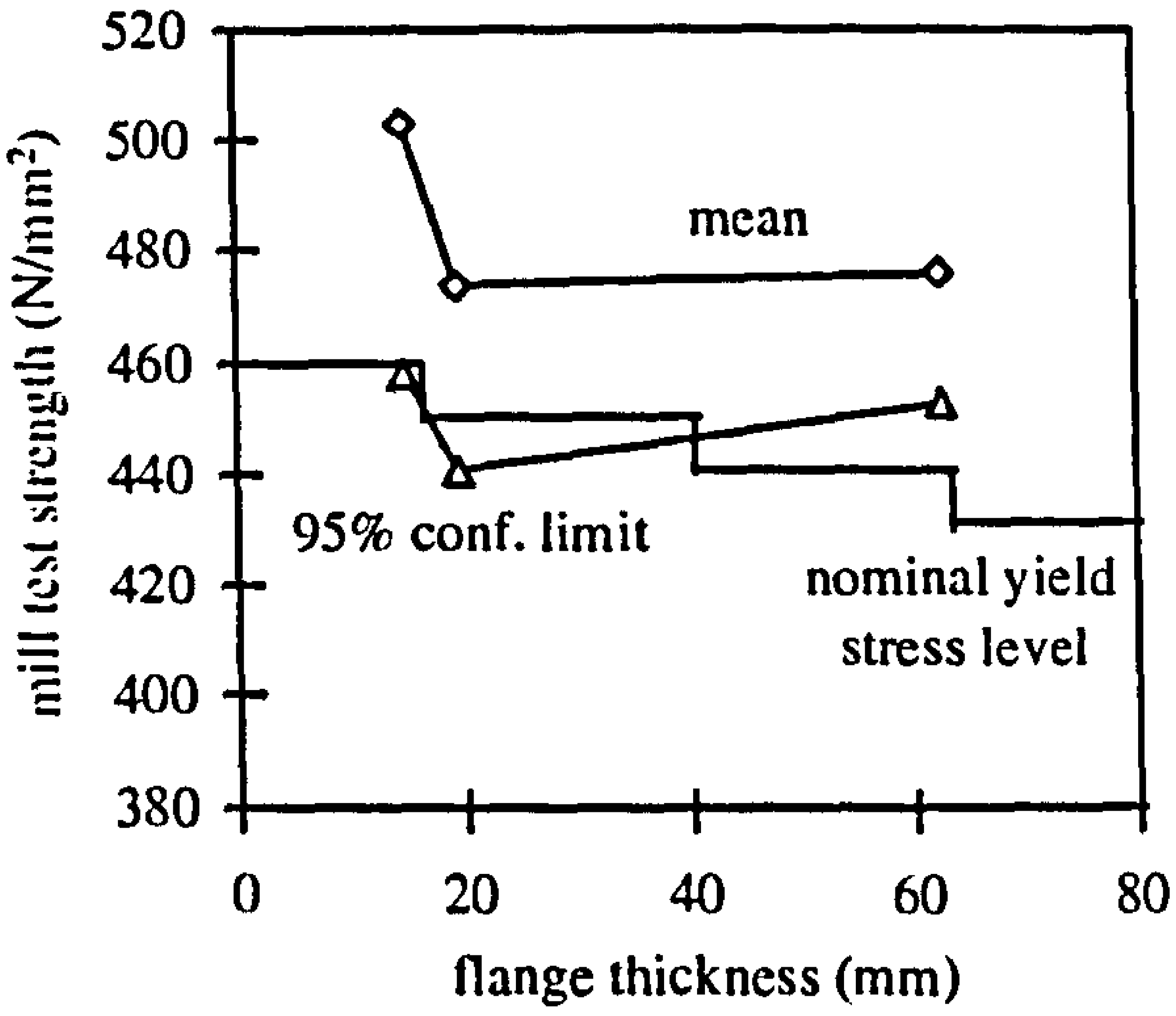

Fig. A2.5: Mill test vs. t_f
S460-A-ROS-672

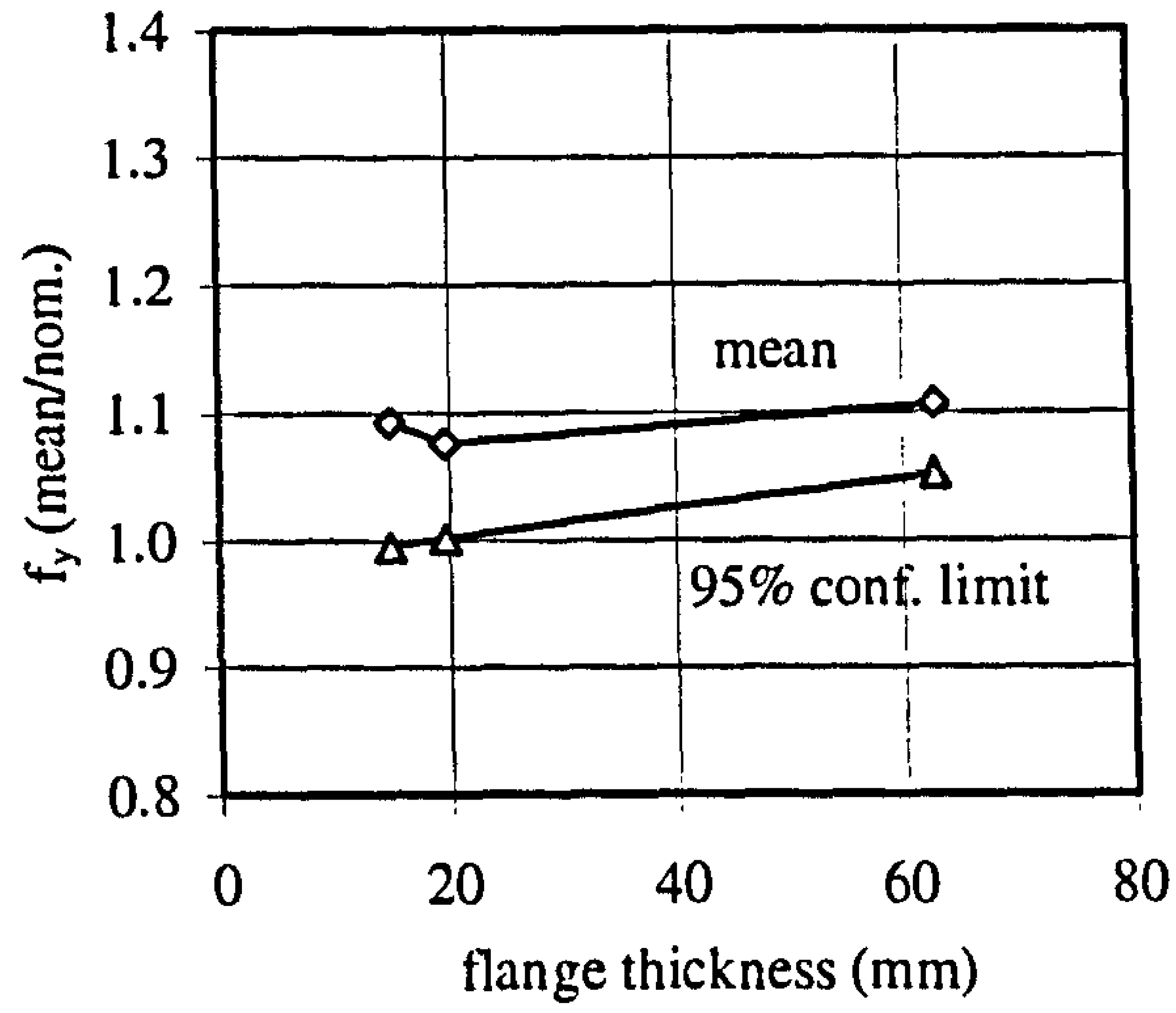

Fig. A2.6: Normalized mill test vs. t_f
S460-A-ROS-672

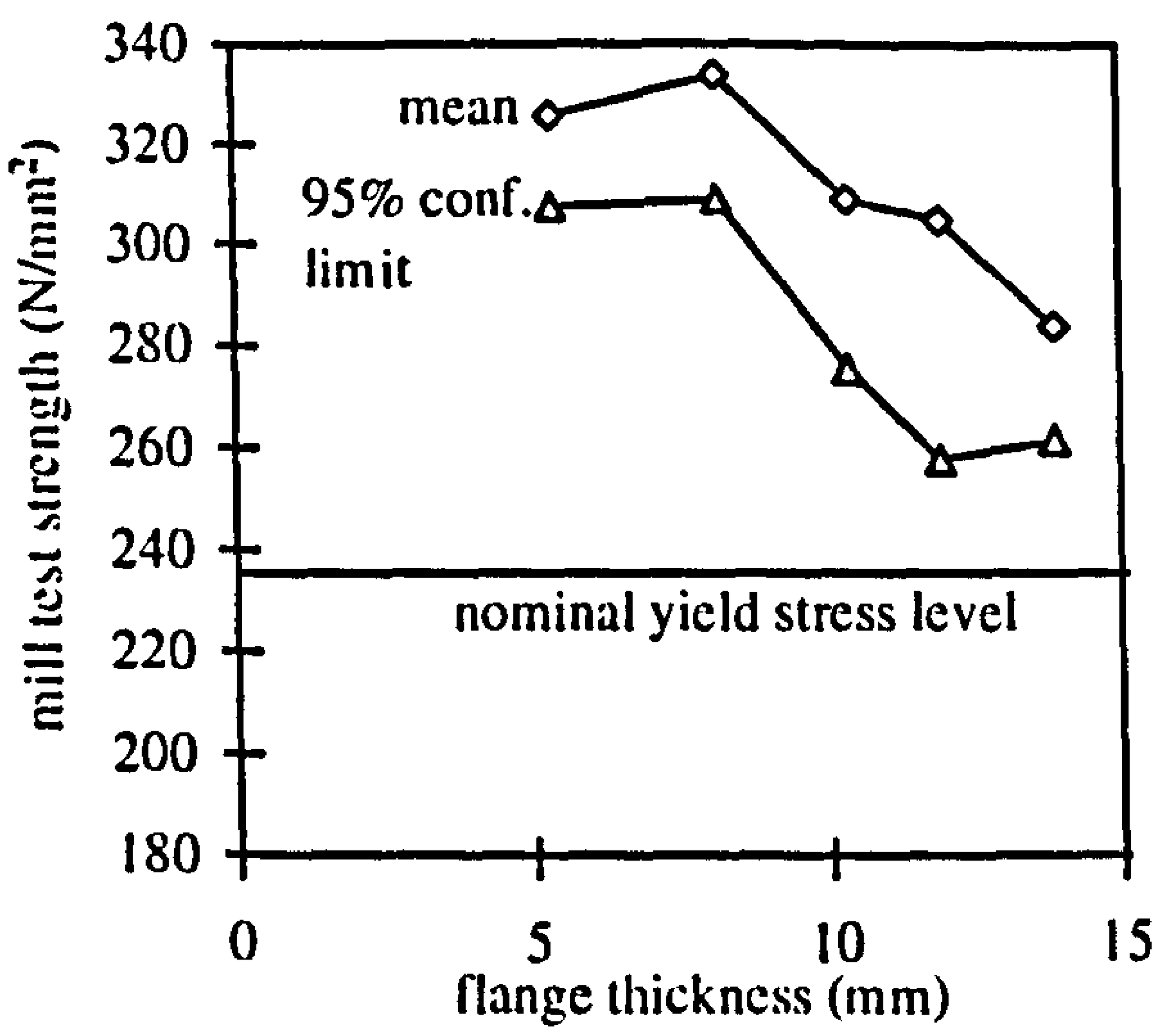

Fig. A2.7: Mill test vs. t_f
S235-B-ROS-689

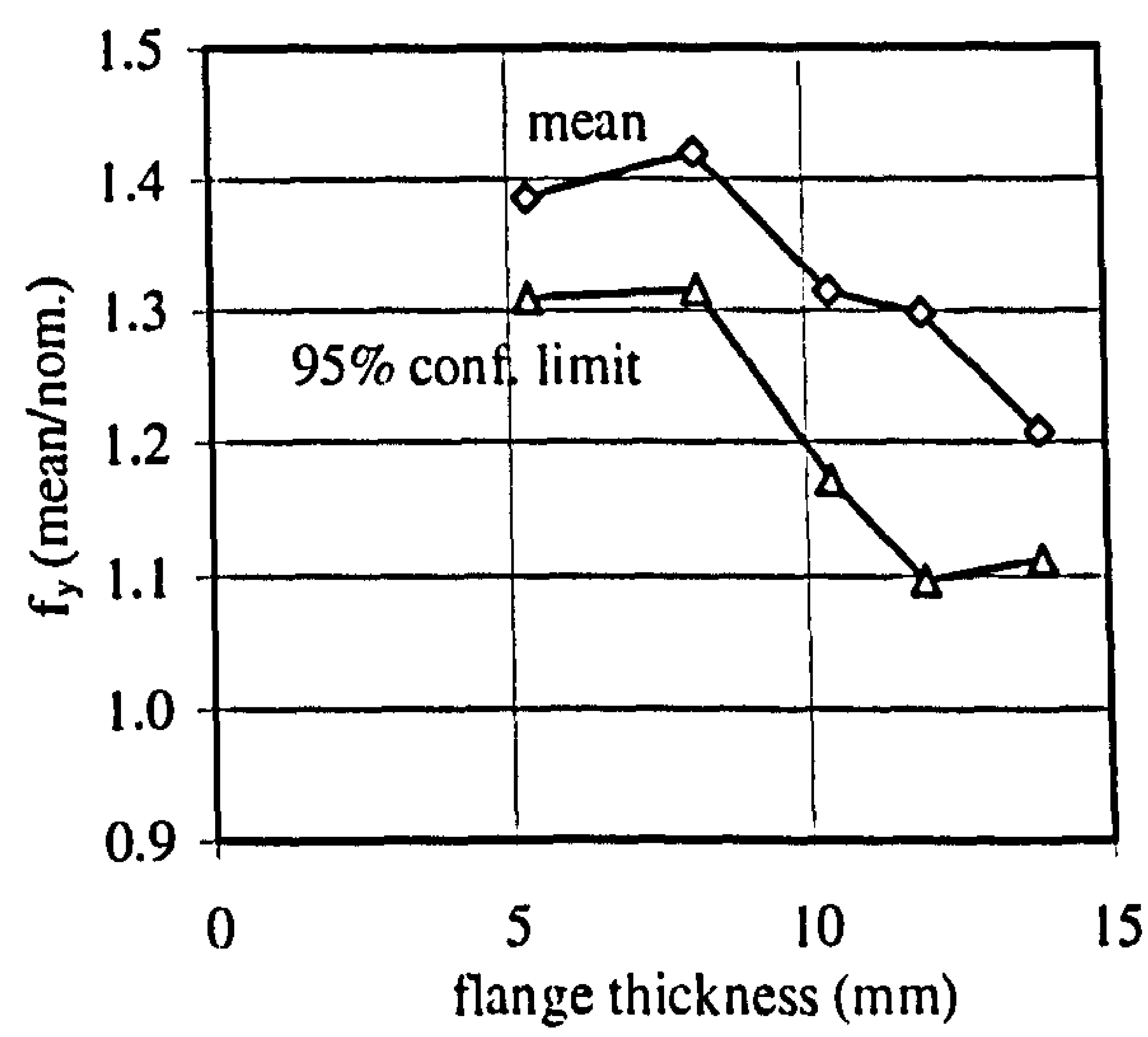

Fig. A2.8: Normalized mill test vs. t_f
S235-B-ROS-689

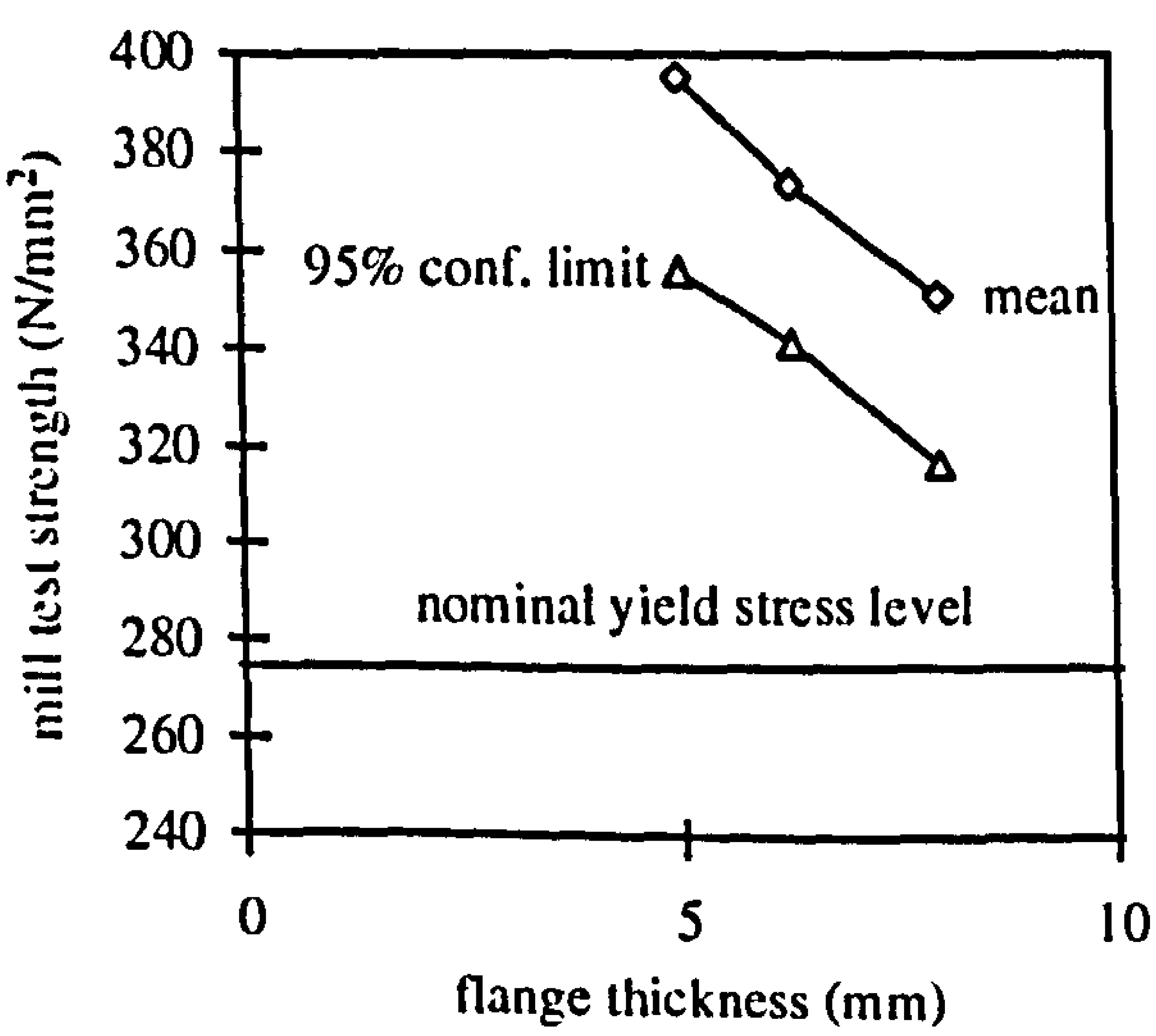

Fig. A2.9: Mill test vs. flange thickness
S275-A-RHS-290

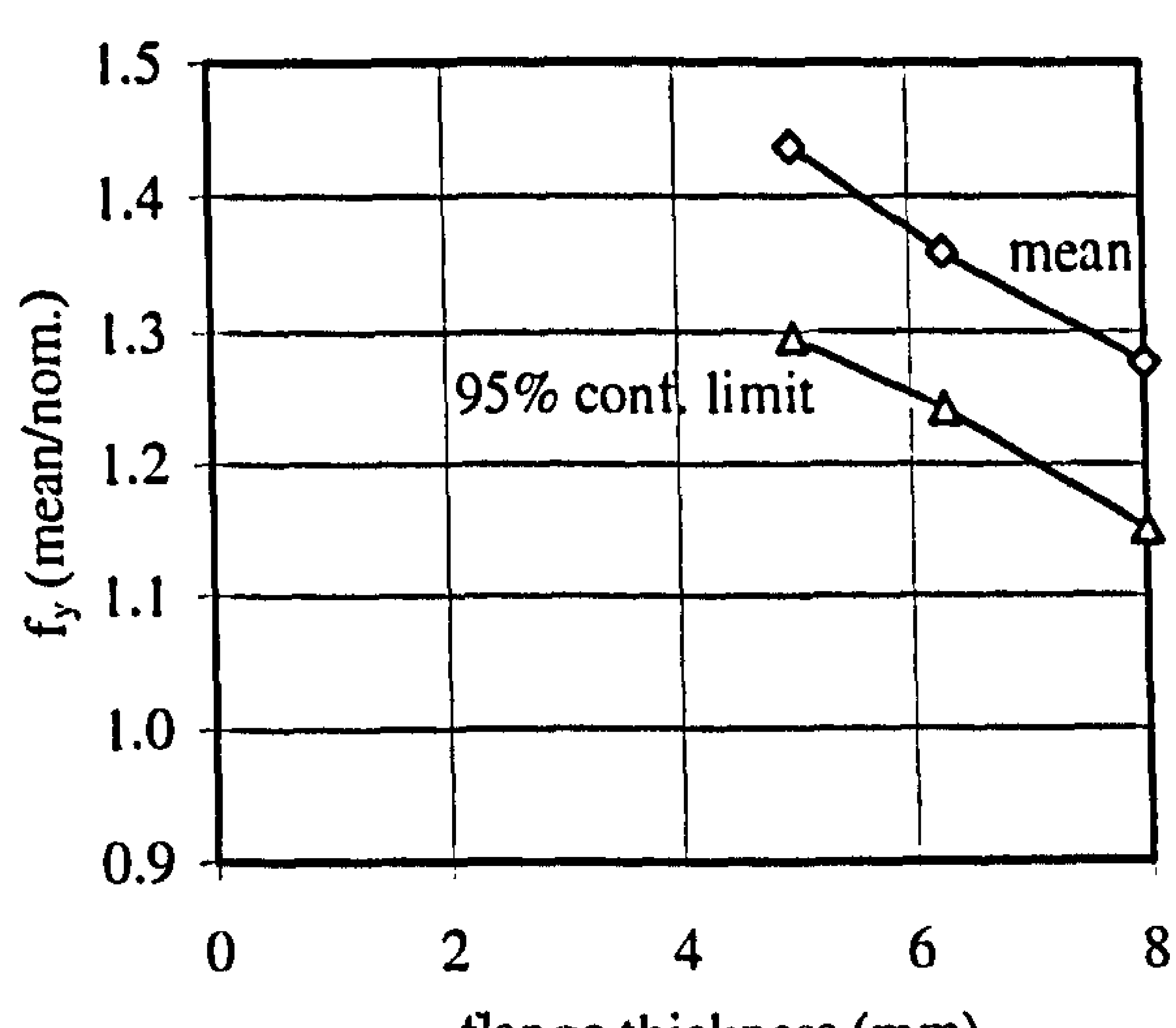

Fig. A2.10: Normalised mill test vs. t_f
S275-A-RHS-290

Producer	Grade	Sample size	t_f mean (mm)	f_y nom. (N/mm^2)	f_y mean (N/mm^2)	f_y mean/ nom.	f_y COV
A	S275	166	10.8	275	326.06	1.19	0.070
A	S275	537	13.0	275	327.20	1.19	0.059
A	S275	958	15.1	275	311.96	1.13	0.053
A	S275	454	17.3	265	311.80	1.18	0.055
A	S275	473	19.2	265	309.90	1.17	0.054
A	S275	425	21.0	265	305.74	1.15	0.053
A	S275	407	23.5	265	304.11	1.15	0.041
A	S275	127	25.2	265	298.49	1.13	0.043
A	S275	142	27.9	265	304.34	1.15	0.056
A	S275	314	32.5	265	300.42	1.13	0.050
A	S275	71	48.1	255	299.23	1.17	0.047
A	S275	21	73.4	245	290.38	1.19	0.033

Table A2.1: Summary of mill test data for S275-A-ROS-4095

Producer	Grade	Sample size	t_f mean (mm)	f_y nom. (N/mm^2)	f_y mean (N/mm^2)	f_y mean/ nom.	f_y COV
A	S355	103	10.9	355	431.0	1.21	0.054
A	S355	147	12.9	355	425.7	1.20	0.035
A	S355	435	14.9	355	409.6	1.15	0.046
A	S355	198	17.3	345	401.9	1.17	0.038
A	S355	135	19.1	345	403.5	1.17	0.045
A	S355	197	20.9	345	395.0	1.15	0.034
A	S355	82	23.4	345	396.9	1.15	0.036
A	S355	137	25.4	345	395.1	1.15	0.038
A	S355	83	27.9	345	390.1	1.13	0.034
A	S355	297	33.3	345	389.7	1.13	0.040
A	S355	77	47.2	335	380.5	1.14	0.026
A	S355	23	72.9	325	361.9	1.11	0.028

Table A2.2: Summary of mill test results for S355-A-ROS-1914

Producer	Grade	Sample size	t_f mean (mm)	f_y nom. (N/mm^2)	f_y mean (N/mm^2)	f_y mean/ nom.	f_y COV
A	S460	19	15.0	460	502.8	1.09	0.054
A	S460	647	24.8	440	473.8	1.08	0.042
A	S460	6	62.7	430	476.0	1.11	0.030

Table A2.3: Summary of mill test results for S460-A-ROS-672

Producer	Grade	Sample size	t_f mean (mm)	f_y nom. (N/mm^2)	f_y mean (N/mm^2)	f_y mean/ nom.	f_y COV
B	S235	90	5.1	235	325.7	1.39	0.033
B	S235	240	7.2	235	333.8	1.42	0.045
B	S235	209	9.9	235	309.0	1.31	0.066
B	S235	60	11.8	235	304.9	1.30	0.094
B	S235	90	13.7	235	283.8	1.21	0.048

Table A2.4: Summary of mill test results for S235-B-ROS-689

Producer	Grade	Sample size	t mean (mm)	f_y nom. (N/mm^2)	f_y mean (N/mm^2)	f_y mean/ nom.	f_y COV
A	S275	88	5.0	275	395.2	1.44	0.060
A	S275	165	6.3	275	373.7	1.36	0.053
A	S275	37	8.0	275	351.0	1.28	0.060

Table A2.5: Summary of mill test results for S275-A-RHS-290

Section	Date	n°	h mm	b_1 mm	b_2 mm	t_w mm	e_1 mm	e_2 mm	e_3 mm	e_4 mm	f_y N/mm2
IPE 80 (1)	23/10/1989	1	79.5	46.1	45.9	4.0	5.2	4.9	4.9	5.2	310.0
IPE 80 (1)	23/10/1989	2	79.5	46.1	46.0	4.0	5.1	5.0	5.0	5.3	308.0
IPE 80 (1)	23/10/1989	3	79.6	46.0	45.9	4.0	5.1	4.9	5.0	5.3	319.0
IPE 80 (1)	23/10/1989	4	79.6	46.1	46.0	4.0	5.2	4.9	5.0	5.3	310.0
IPE 80 (1)	23/10/1989	5	79.6	46.0	46.0	4.0	5.2	4.9	5.0	5.3	326.0
IPE 80 (1)	23/10/1989	6	79.6	45.9	45.8	4.0	5.2	4.9	4.9	5.3	317.0
IPE 80 (1)	23/10/1989	7	79.6	45.9	45.8	4.0	5.2	5.0	5.0	5.3	302.0
IPE 80 (1)	23/10/1989	8	79.5	46.2	46.1	4.0	5.2	4.9	4.9	5.3	310.0
IPE 80 (1)	23/10/1989	9	79.5	46.0	45.8	4.0	5.2	4.9	5.0	5.3	321.0
IPE 80 (1)	23/10/1989	10	79.6	46.0	45.8	4.0	5.2	4.9	4.9	5.3	308.0
IPE 80 (1)	23/10/1989	11	79.6	46.2	45.9	4.0	5.4	5.1	5.0	5.6	313.0
IPE 80 (1)	23/10/1989	12	79.6	46.0	45.8	4.0	5.5	5.0	5.0	5.7	324.0
IPE 80 (1)	23/10/1989	13	79.5	46.2	45.6	4.0	5.5	5.0	4.9	5.6	311.0
IPE 80 (1)	23/10/1989	14	79.6	46.0	45.7	4.0	5.4	5.0	4.9	5.6	311.0
IPE 80 (1)	23/10/1989	15	79.6	46.6	45.7	4.0	5.5	5.0	5.0	5.7	310.0
IPE 80 (1)	23/10/1989	16	79.7	46.0	45.8	4.0	5.5	4.9	5.0	5.7	321.0
IPE 80 (1)	23/10/1989	17	79.6	46.4	45.6	4.0	5.4	5.0	5.0	5.6	321.0
IPE 80 (1)	23/10/1989	18	79.7	46.2	45.8	4.0	5.5	5.0	5.0	5.7	316.0
IPE 80 (1)	23/10/1989	19	79.7	46.0	46.0	4.0	5.5	5.1	5.0	5.6	318.0
IPE 80 (1)	23/10/1989	20	79.7	46.2	45.7	4.0	5.5	5.0	5.0	5.7	318.0
IPE 80 (1)	23/10/1989	21	81.2	47.0	46.7	4.0	5.5	5.3	5.1	5.5	316.0
IPE 80 (1)	23/10/1989	22	81.2	46.7	46.6	4.0	5.5	5.2	5.1	5.5	322.0
IPE 80 (1)	23/10/1989	23	81.1	47.0	47.0	4.0	5.5	5.3	5.0	5.5	308.0
IPE 80 (1)	23/10/1989	24	81.2	47.2	47.1	4.0	5.4	5.3	5.0	5.4	332.0
IPE 80 (1)	23/10/1989	25	81.1	46.8	46.7	4.0	5.4	5.2	5.0	5.5	329.0
IPE 80 (1)	23/10/1989	26	81.1	47.6	47.1	4.0	5.4	5.2	5.1	5.4	317.0
IPE 80 (1)	23/10/1989	27	81.1	46.8	46.6	4.0	5.5	5.2	5.0	5.5	321.0
IPE 80 (1)	23/10/1989	28	81.1	46.7	46.6	4.0	5.5	5.2	5.0	5.4	319.0
IPE 80 (1)	23/10/1989	29	81.1	46.7	46.8	4.0	5.4	5.2	5.0	5.4	318.0
IPE 80 (1)	23/10/1989	30	81.2	47.0	46.8	4.0	5.5	5.3	5.1	5.5	320.0
IPE 80 (2)	13/03/1990	1	78.5	46.6	46.8	3.8	5.1	4.9	5.1	5.1	322.0
IPE 80 (2)	13/03/1990	2	78.6	46.6	46.8	3.8	5.1	4.9	5.1	5.1	335.0
IPE 80 (2)	13/03/1990	3	78.6	46.5	46.8	3.9	5.1	4.9	5.1	5.1	345.0
IPE 80 (2)	13/03/1990	4	78.7	46.4	46.6	3.9	5.2	4.9	5.1	5.1	332.0
IPE 80 (2)	13/03/1990	5	78.6	46.4	46.6	3.9	5.2	4.9	5.1	5.1	328.0
IPE 80 (2)	13/03/1990	6	78.6	46.3	46.5	3.9	5.2	4.9	5.1	5.0	330.0
IPE 80 (2)	13/03/1990	7	78.6	46.4	46.5	4.0	5.2	5.0	5.1	5.1	327.0
IPE 80 (2)	13/03/1990	8	78.7	46.4	46.4	4.0	5.3	5.0	5.1	5.1	335.0
IPE 80 (2)	13/03/1990	9	78.7	46.5	46.5	4.0	5.2	4.9	5.1	5.1	340.0
IPE 80 (2)	13/03/1990	10	78.7	46.5	46.5	4.0	5.2	5.0	5.1	5.1	329.0
IPE 80 (2)	13/03/1990	11	79.0	45.9	46.4	4.3	5.5	5.0	5.2	5.2	348.0
IPE 80 (2)	13/03/1990	12	78.9	45.8	46.4	4.3	5.5	5.0	5.2	5.2	332.0
IPE 80 (2)	13/03/1990	13	78.9	45.8	46.5	4.2	5.4	5.0	5.1	5.1	326.0
IPE 80 (2)	13/03/1990	14	79.0	46.0	46.6	4.2	5.4	5.0	5.1	5.2	326.0
IPE 80 (2)	13/03/1990	15	79.0	46.0	46.6	4.2	5.4	5.1	5.1	5.2	340.0
IPE 80 (2)	13/03/1990	16	78.7	46.0	46.8	4.2	5.4	5.1	5.1	5.1	348.0
IPE 80 (2)	13/03/1990	17	78.7	46.2	46.7	4.2	5.5	5.1	5.1	5.1	354.0
IPE 80 (2)	13/03/1990	18	78.9	46.2	46.8	4.2	5.4	5.0	5.0	5.1	334.0
IPE 80 (2)	13/03/1990	19	78.8	46.2	46.8	4.2	5.4	5.0	5.0	5.1	327.0
IPE 80 (2)	13/03/1990	20	78.8	46.3	46.8	4.2	5.4	5.0	5.1	5.1	332.0
IPE 80 (2)	13/03/1990	21	78.8	47.0	47.0	4.2	5.3	4.9	5.0	5.0	325.0
IPE 80 (2)	13/03/1990	22	79.0	47.1	46.8	4.2	5.3	4.9	5.0	5.0	338.0
IPE 80 (2)	13/03/1990	23	79.0	47.0	47.0	4.2	5.3	5.0	5.0	5.1	339.0
IPE 80 (2)	13/03/1990	24	79.1	47.0	46.9	4.2	5.3	5.0	5.1	5.1	332.0
IPE 80 (2)	13/03/1990	25	79.1	46.8	46.9	4.2	5.2	4.9	5.1	5.1	324.0
IPE 80 (2)	13/03/1990	26	79.0	46.8	46.9	4.2	5.2	4.9	5.1	5.1	338.0
IPE 80 (2)	13/03/1990	27	79.1	46.8	47.0	4.1	5.3	4.9	5.1	5.1	329.0
IPE 80 (2)	13/03/1990	28	79.1	46.7	46.8	4.1	5.3	4.9	5.1	5.0	347.0

IPE 80 (2)	13/03/1990	29	79.0	46.8	46.8	4.2	5.3	4.9	5.0	5.1	344.0
IPE 80 (2)	13/03/1990	30	79.0	46.7	46.9	4.2	5.2	4.9	5.0	5.0	338.0
IPE 80 (3)	07/06/1990	1	81.4	45.8	46.5	4.0	5.2	4.9	5.2	4.9	315.0
IPE 80 (3)	07/06/1990	2	81.3	45.7	46.5	4.0	5.2	4.9	5.1	5.0	318.0
IPE 80 (3)	07/06/1990	3	81.3	45.7	46.5	3.9	5.2	4.9	5.1	5.0	325.0
IPE 80 (3)	07/06/1990	4	81.1	45.8	46.3	3.9	5.1	5.0	5.1	4.9	323.0
IPE 80 (3)	07/06/1990	5	81.2	45.9	46.4	3.9	5.1	5.0	5.1	5.1	312.0
IPE 80 (3)	07/06/1990	6	81.2	45.9	46.3	4.0	5.2	5.1	5.1	5.0	335.0
IPE 80 (3)	07/06/1990	7	81.2	46.0	46.3	4.0	5.2	5.1	5.0	5.0	332.0
IPE 80 (3)	07/06/1990	8	81.1	46.0	46.2	3.9	5.1	5.0	5.0	5.0	327.0
IPE 80 (3)	07/06/1990	9	81.1	46.0	46.2	3.9	5.1	5.0	5.0	5.0	328.0
IPE 80 (3)	07/06/1990	10	81.1	46.0	46.2	3.9	5.1	5.0	5.1	5.0	332.0
IPE 80 (3)	07/06/1990	11	81.0	46.1	45.8	3.8	5.0	4.9	4.9	5.1	344.0
IPE 80 (3)	07/06/1990	12	81.0	46.0	45.9	3.8	5.0	4.9	5.0	5.1	325.0
IPE 80 (3)	07/06/1990	13	81.0	46.2	45.9	3.8	5.0	4.9	5.0	5.0	318.0
IPE 80 (3)	07/06/1990	14	81.0	46.2	45.9	3.9	5.1	5.0	4.9	5.1	340.0
IPE 80 (3)	07/06/1990	15	81.1	46.1	45.8	3.8	5.1	5.0	4.9	5.1	331.0
IPE 80 (3)	07/06/1990	16	81.1	46.0	45.8	3.8	5.1	5.0	4.8	5.1	325.0
IPE 80 (3)	07/06/1990	17	81.1	45.9	45.9	3.8	5.0	5.0	4.9	5.1	318.0
IPE 80 (3)	07/06/1990	18	81.0	45.9	45.9	3.8	4.9	4.9	4.8	5.1	324.0
IPE 80 (3)	07/06/1990	19	81.0	46.0	45.9	3.8	4.9	5.0	4.8	5.1	331.0
IPE 80 (3)	07/06/1990	20	81.0	46.0	45.9	3.8	5.0	5.0	4.8	5.1	328.0
IPE 80 (3)	07/06/1990	21	81.0	46.0	46.0	3.8	4.8	4.9	4.9	5.0	315.0
IPE 80 (3)	07/06/1990	22	80.9	46.0	46.1	3.8	4.9	4.9	4.9	5.1	314.0
IPE 80 (3)	07/06/1990	23	81.0	45.1	46.0	3.8	4.8	4.9	4.8	5.1	317.0
IPE 80 (3)	07/06/1990	24	81.0	45.9	45.9	3.8	4.8	4.9	4.9	5.0	328.0
IPE 80 (3)	07/06/1990	25	81.0	46.0	45.9	3.9	4.9	4.8	4.9	5.0	312.0
IPE 80 (3)	07/06/1990	26	81.1	46.1	45.8	3.9	4.8	4.9	4.9	5.0	330.0
IPE 80 (3)	07/06/1990	27	81.1	46.1	45.8	3.9	4.9	4.9	5.0	4.9	327.0
IPE 80 (3)	07/06/1990	28	81.0	46.2	45.9	3.8	4.9	5.0	5.0	4.9	322.0
IPE 80 (3)	07/06/1990	29	81.0	46.1	45.9	3.9	4.8	5.0	4.9	5.0	331.0
IPE 80 (3)	07/06/1990	30	81.0	46.2	46.0	3.9	4.8	4.9	4.9	5.0	345.0
IPE 120 (1)	16/10/1989	1	120.6	63.4	63.4	4.1	6.0	6.2	6.1	6.1	317.0
IPE 120 (1)	16/10/1989	2	120.6	63.4	63.2	4.1	6.1	6.2	6.2	6.0	312.0
IPE 120 (1)	16/10/1989	3	120.7	63.3	63.4	4.1	6.0	6.1	6.1	6.1	313.0
IPE 120 (1)	16/10/1989	4	120.6	63.1	63.6	4.1	6.0	6.2	6.1	6.1	325.0
IPE 120 (1)	16/10/1989	5	120.6	63.2	63.2	4.1	6.1	6.2	6.2	6.1	316.0
IPE 120 (1)	16/10/1989	6	120.8	63.4	63.5	4.2	6.1	6.2	6.2	6.0	315.0
IPE 120 (1)	16/10/1989	7	120.8	63.3	63.0	4.1	6.2	6.3	6.3	6.1	314.0
IPE 120 (1)	16/10/1989	8	120.8	63.7	63.4	4.2	6.1	6.3	6.1	6.1	349.0
IPE 120 (1)	16/10/1989	9	120.7	63.5	64.0	4.2	6.1	6.2	6.2	6.1	341.0
IPE 120 (1)	16/10/1989	10	120.7	63.7	63.5	4.2	6.2	6.2	6.2	6.2	335.0
IPE 120 (1)	16/10/1989	11	120.9	64.5	64.3	4.5	6.1	6.3	6.3	6.1	323.0
IPE 120 (1)	16/10/1989	12	120.8	64.2	64.1	4.4	6.1	6.3	6.3	6.2	322.0
IPE 120 (1)	16/10/1989	13	120.8	64.6	64.4	4.5	6.1	6.2	6.2	6.1	308.0
IPE 120 (1)	16/10/1989	14	120.9	64.8	64.8	4.4	6.2	6.2	6.3	6.1	337.0
IPE 120 (1)	16/10/1989	15	120.8	64.3	64.0	4.5	6.2	6.3	6.2	6.1	320.0
IPE 120 (1)	16/10/1989	16	120.9	64.7	64.4	4.5	6.1	6.2	6.2	6.2	316.0
IPE 120 (1)	16/10/1989	17	120.9	64.5	64.3	4.5	6.1	6.3	6.2	6.1	304.0
IPE 120 (1)	16/10/1989	18	120.9	64.5	64.1	4.4	6.1	6.3	6.2	6.1	320.0
IPE 120 (1)	16/10/1989	19	120.8	64.3	63.6	4.5	6.2	6.3	6.1	6.1	287.0
IPE 120 (1)	16/10/1989	20	120.8	64.7	64.3	4.4	6.2	6.4	6.3	6.1	316.0
IPE 120 (1)	16/10/1989	21	121.8	67.2	66.5	4.7	6.2	6.3	6.2	6.1	330.0
IPE 120 (1)	16/10/1989	22	121.9	66.4	66.4	4.6	6.2	6.4	6.2	6.1	319.0
IPE 120 (1)	16/10/1989	23	121.7	67.4	67.0	4.7	6.2	6.3	6.3	6.1	321.0
IPE 120 (1)	16/10/1989	24	122.0	66.9	66.8	4.7	6.1	6.2	6.4	6.0	316.0
IPE 120 (1)	16/10/1989	25	121.8	67.4	67.0	4.7	6.2	6.2	6.4	6.1	323.0
IPE 120 (1)	16/10/1989	26	121.9	66.5	66.5	4.7	6.1	6.3	6.4	6.0	330.0
IPE 120 (1)	16/10/1989	27	121.9	66.6	66.4	4.6	6.1	6.4	6.2	6.1	342.0
IPE 120 (1)	16/10/1989	28	122.0	64.6	63.2	4.5	6.1	6.5	6.2	6.1	323.0
IPE 120 (1)	16/10/1989	29	121.8	65.0	65.0	4.5	6.2	6.3	6.2	6.0	329.0
IPE 120 (1)	16/10/1989	30	121.9	65.0	64.6	4.7	6.1	6.3	6.3	6.1	328.0
IPE 140 (1)	06/10/1989	1	138.7	71.8	71.5	4.5	6.5	6.4	6.8	6.4	322.0
IPE 140 (1)	06/10/1989	2	138.8	71.6	71.4	4.6	6.6	6.5	6.9	6.5	324.0
IPE 140 (1)	06/10/1989	3	138.8	73.6	73.6	4.8	6.5	6.5	6.8	6.5	336.0
IPE 140 (1)	06/10/1989	4	138.8	73.2	72.8	4.8	6.5	6.6	6.8	6.4	324.0
IPE 140 (1)	06/10/1989	5	138.9	72.4	72.0	4.6	6.5	6.5	6.9	6.5	351.0
IPE 140 (1)	06/10/1989	6	138.8	73.0	72.4	4.7	6.6	6.7	6.8	6.5	317.0
IPE 140 (1)	06/10/1989	7	138.6	72.4	71.5	4.6	6.4	6.5	6.7	6.5	308.0
IPE 140 (1)	06/10/1989	8	138.6	73.2	72.2	4.8	6.4	6.5	6.8	6.4	328.0
IPE 140 (1)	06/10/1989	9	138.7	73.4	73.2	4.9	6.5	6.4	6.8	6.4	323.0
IPE 140 (1)	06/10/1989	10	138.8	72.6	72.6	4.6	6.6	6.6	6.9	6.4	336.0
IPE 140 (1)	06/10/1989	11	138.9	75.7	75.5	5.2	6.9	6.9	6.9	6.8	314.0
IPE 140 (1)	06/10/1989	12	138.6	74.8	72.1	5.2	7.0	6.8	6.9	6.6	314.0
IPE 140 (1)	06/10/1989	13	138.7	75.4	74.9	5.1	6.9	6.9	7.0	6.7	322.0

IPE 140 (1)	06/10/1989	14	138.6	75.6	75.0	5.1	7.0	6.6	6.9	6.6	314.0
IPE 140 (1)	06/10/1989	15	138.8	75.5	75.1	5.1	7.1	6.7	7.0	6.7	320.0
IPE 140 (1)	06/10/1989	16	138.8	74.4	74.1	5.4	7.0	6.9	7.1	6.5	311.0
IPE 140 (1)	06/10/1989	17	139.0	75.0	75.0	5.3	7.0	7.0	7.1	6.7	330.0
IPE 140 (1)	06/10/1989	18	139.1	75.1	75.0	5.3	7.0	6.9	7.1	7.0	309.0
IPE 140 (1)	06/10/1989	19	139.0	75.3	75.0	5.4	7.0	6.8	7.0	6.8	318.0
IPE 140 (1)	06/10/1989	20	139.2	75.6	75.4	5.3	7.1	6.8	7.0	6.8	321.0
IPE 140 (1)	06/10/1989	21	142.6	72.5	72.4	4.6	7.0	6.8	7.0	6.4	306.0
IPE 140 (1)	06/10/1989	22	142.6	74.4	73.8	4.6	7.0	6.8	7.0	6.5	321.0
IPE 140 (1)	06/10/1989	23	142.7	74.3	73.4	4.6	6.9	6.8	7.0	6.6	320.0
IPE 140 (1)	06/10/1989	24	142.7	74.0	73.0	4.6	7.0	6.8	7.0	6.7	325.0
IPE 140 (1)	06/10/1989	25	142.7	73.9	73.5	4.6	7.0	6.5	6.9	6.8	334.0
IPE 140 (1)	06/10/1989	26	142.7	74.4	74.4	4.6	7.0	6.8	7.1	6.5	354.0
IPE 140 (1)	06/10/1989	27	142.8	75.1	74.8	4.7	7.0	6.7	7.0	6.9	360.0
IPE 140 (1)	06/10/1989	28	142.7	74.7	74.2	4.6	7.0	6.7	7.0	6.9	362.0
IPE 140 (1)	06/10/1989	29	142.8	75.3	74.6	4.7	7.0	6.9	6.9	6.6	365.0
IPE 140 (1)	06/10/1989	30	142.8	75.3	74.4	4.6	7.0	6.9	6.9	6.6	354.0
IPE 160 (1)	13/02/1990	1	160.4	81.2	82.0	5.1	7.5	6.9	7.1	7.2	348.0
IPE 160 (1)	13/02/1990	2	160.4	81.5	82.1	5.1	7.0	7.2	7.1	7.3	342.0
IPE 160 (1)	13/02/1990	3	160.3	81.5	82.1	5.1	7.1	7.2	7.2	7.3	352.0
IPE 160 (1)	13/02/1990	4	160.7	81.8	82.4	5.0	7.4	7.2	7.3	7.2	351.0
IPE 160 (1)	13/02/1990	5	160.7	81.2	82.5	5.0	7.3	7.3	7.3	7.3	335.0
IPE 160 (1)	13/02/1990	6	160.6	81.4	82.2	5.1	7.3	7.3	7.2	7.2	348.0
IPE 160 (1)	13/02/1990	7	160.4	81.5	82.5	5.1	7.4	7.2	7.2	7.1	350.0
IPE 160 (1)	13/02/1990	8	160.3	81.8	82.6	5.0	7.3	7.1	7.3	7.3	352.0
IPE 160 (1)	13/02/1990	9	160.3	82.0	82.4	5.0	7.1	7.1	7.1	7.1	358.0
IPE 160 (1)	13/02/1990	10	160.5	82.0	82.0	5.0	7.1	7.4	7.1	7.3	342.0
IPE 160 (1)	13/02/1990	11	159.7	83.0	82.8	4.9	7.0	7.3	7.1	7.2	355.0
IPE 160 (1)	13/02/1990	12	159.7	82.8	82.7	5.0	7.1	7.3	7.1	7.1	352.0
IPE 160 (1)	13/02/1990	13	160.0	82.7	82.6	4.9	7.1	7.3	7.2	7.0	345.0
IPE 160 (1)	13/02/1990	14	160.0	82.7	82.8	4.9	7.0	7.2	7.2	7.1	330.0
IPE 160 (1)	13/02/1990	15	160.1	82.8	82.7	5.0	7.0	7.3	7.3	7.1	329.0
IPE 160 (1)	13/02/1990	16	160.4	83.0	82.7	5.0	7.2	7.1	7.1	7.2	339.0
IPE 160 (1)	13/02/1990	17	159.9	83.1	82.4	5.0	7.1	7.3	7.1	7.2	342.0
IPE 160 (1)	13/02/1990	18	159.8	83.1	82.5	5.0	7.1	7.3	7.2	7.2	336.0
IPE 160 (1)	13/02/1990	19	160.0	83.1	81.9	4.9	7.1	7.3	7.2	7.1	345.0
IPE 160 (1)	13/02/1990	20	160.1	82.8	82.0	4.9	7.1	7.3	7.3	7.1	346.0
IPE 160 (1)	13/02/1990	21	160.4	82.5	83.1	5.1	7.2	7.2	7.4	7.4	337.0
IPE 160 (1)	13/02/1990	22	160.2	82.2	83.0	5.1	7.2	7.3	7.3	7.3	348.0
IPE 160 (1)	13/02/1990	23	160.3	82.3	83.1	5.1	7.3	7.2	7.4	7.1	352.0
IPE 160 (1)	13/02/1990	24	160.3	82.3	83.3	5.0	7.1	7.2	7.3	7.0	358.0
IPE 160 (1)	13/02/1990	25	160.5	82.0	83.1	5.1	7.2	7.2	7.2	7.2	347.0
IPE 160 (1)	13/02/1990	26	160.0	82.2	83.0	5.0	7.2	7.3	7.0	7.3	338.0
IPE 160 (1)	13/02/1990	27	160.4	82.2	83.2	5.1	7.0	7.2	7.2	7.2	349.0
IPE 160 (1)	13/02/1990	28	160.4	82.0	82.9	5.1	7.3	7.3	7.2	7.1	329.0
IPE 160 (1)	13/02/1990	29	160.1	82.3	82.8	5.1	7.2	7.1	7.3	7.2	340.0
IPE 160 (1)	13/02/1990	30	160.2	82.3	83.0	5.1	7.2	7.3	7.2	7.2	345.0
IPE 160 (2)	19/04/1990	1	161.4	80.4	80.1	4.6	6.9	7.0	7.0	6.9	318.0
IPE 160 (2)	19/04/1990	2	161.2	80.2	80.4	4.7	7.0	7.0	7.0	6.9	332.0
IPE 160 (2)	19/04/1990	3	161.2	80.4	80.2	4.7	7.0	7.1	7.2	7.1	315.0
IPE 160 (2)	19/04/1990	4	161.2	80.6	80.3	4.8	7.0	7.2	7.0	7.2	318.0
IPE 160 (2)	19/04/1990	5	161.4	80.8	80.5	4.7	7.0	7.2	7.1	7.2	330.0
IPE 160 (2)	19/04/1990	6	161.4	80.7	80.5	4.7	6.9	7.1	7.0	7.2	322.0
IPE 160 (2)	19/04/1990	7	161.5	80.7	80.7	4.8	7.0	7.3	7.2	7.2	340.0
IPE 160 (2)	19/04/1990	8	161.2	80.5	80.4	4.7	7.0	7.1	7.1	7.1	326.0
IPE 160 (2)	19/04/1990	9	161.3	80.7	80.6	4.7	7.0	7.4	7.1	7.2	319.0
IPE 160 (2)	19/04/1990	10	161.2	80.7	80.7	4.8	7.0	7.1	7.0	7.1	317.0
IPE 160 (2)	19/04/1990	11	160.5	81.2	81.6	4.9	7.0	7.1	7.0	7.0	328.0
IPE 160 (2)	19/04/1990	12	160.8	81.2	81.5	4.9	7.0	7.2	7.0	7.2	335.0
IPE 160 (2)	19/04/1990	13	160.7	81.0	82.0	5.0	7.1	7.3	7.1	7.1	342.0
IPE 160 (2)	19/04/1990	14	160.9	81.3	82.2	4.9	7.0	7.2	7.2	7.0	325.0
IPE 160 (2)	19/04/1990	15	160.9	81.5	83.0	5.0	7.0	7.3	7.1	7.2	328.0
IPE 160 (2)	19/04/1990	16	160.7	81.5	82.8	5.0	6.9	7.1	7.0	7.2	342.0
IPE 160 (2)	19/04/1990	17	161.0	81.5	82.2	4.9	6.9	7.1	7.1	7.0	330.0
IPE 160 (2)	19/04/1990	18	161.0	81.2	82.0	4.9	7.0	7.2	7.1	7.3	331.0
IPE 160 (2)	19/04/1990	19	161.1	81.0	82.0	4.9	7.0	7.1	7.1	7.4	325.0
IPE 160 (2)	19/04/1990	20	161.1	81.5	82.1	4.9	7.0	7.2	7.1	7.2	319.0
IPE 160 (2)	19/04/1990	21	161.3	82.1	83.2	5.0	7.0	7.5	7.2	7.0	335.0
IPE 160 (2)	19/04/1990	22	161.2	81.9	82.9	5.0	7.0	7.3	7.1	7.2	325.0
IPE 160 (2)	19/04/1990	23	161.6	82.2	83.1	5.1	7.1	7.4	7.3	7.3	322.0
IPE 160 (2)	19/04/1990	24	161.3	82.5	83.1	4.9	7.0	7.4	7.2	7.3	341.0
IPE 160 (2)	19/04/1990	25	161.5	82.4	82.8	5.0	7.0	7.5	7.1	7.4	338.0
IPE 160 (2)	19/04/1990	26	161.0	82.4	82.6	5.0	7.0	7.2	7.1	7.1	330.0
IPE 160 (2)	19/04/1990	27	161.2	82.0	82.2	5.0	6.9	7.3	7.0	7.2	325.0
IPE 160 (2)	19/04/1990	28	161.2	82.1	82.2	5.0	6.9	7.3	7.2	7.2	325.0

IPE 160 (2)	19/04/1990	29	161.0	82.2	82.6	5.1	7.0	7.5	7.0	7.3	329.0
IPE 160 (2)	19/04/1990	30	161.0	82.1	82.6	4.9	7.0	7.3	7.1	7.2	328.0
IPE 160 (3)	29/05/1990	1	162.2	83.4	82.6	4.8	7.1	6.9	6.9	7.1	312.0
IPE 160 (3)	29/05/1990	2	161.9	83.2	82.5	4.8	7.0	7.0	6.9	7.1	318.0
IPE 160 (3)	29/05/1990	3	162.0	83.2	82.0	4.9	7.2	7.1	6.9	7.1	328.0
IPE 160 (3)	29/05/1990	4	162.0	83.0	82.0	4.9	7.2	7.2	7.0	7.1	325.0
IPE 160 (3)	29/05/1990	5	162.0	83.2	82.2	4.9	7.1	7.1	7.0	7.0	330.0
IPE 160 (3)	29/05/1990	6	161.9	82.8	82.4	4.8	7.1	7.2	7.1	7.1	320.0
IPE 160 (3)	29/05/1990	7	161.9	82.8	82.3	4.8	7.1	7.2	7.0	7.1	315.0
IPE 160 (3)	29/05/1990	8	162.1	82.8	82.4	4.9	7.2	7.3	7.0	7.1	315.0
IPE 160 (3)	29/05/1990	9	162.1	82.4	82.0	4.9	7.2	7.2	7.1	7.1	310.0
IPE 160 (3)	29/05/1990	10	162.0	82.4	82.1	4.9	7.2	7.2	7.2	7.2	320.0
IPE 160 (3)	29/05/1990	11	162.0	82.0	82.1	5.0	7.4	7.0	7.4	7.5	318.0
IPE 160 (3)	29/05/1990	12	161.7	82.3	82.4	5.0	7.4	7.1	7.3	7.3	314.0
IPE 160 (3)	29/05/1990	13	161.7	81.8	82.7	5.0	7.4	7.2	7.3	7.1	322.0
IPE 160 (3)	29/05/1990	14	161.9	81.7	82.5	5.0	7.5	7.1	7.2	7.3	330.0
IPE 160 (3)	29/05/1990	15	161.8	82.3	82.6	5.0	7.4	7.1	7.3	7.2	315.0
IPE 160 (3)	29/05/1990	16	161.8	82.5	82.6	5.1	7.4	7.0	7.3	7.2	322.0
IPE 160 (3)	29/05/1990	17	161.7	81.8	82.6	5.0	7.4	7.0	7.3	7.2	321.0
IPE 160 (3)	29/05/1990	18	161.8	81.9	82.3	5.0	7.3	7.2	7.2	7.1	332.0
IPE 160 (3)	29/05/1990	19	161.6	82.3	82.4	5.0	7.3	7.1	7.1	7.2	315.0
IPE 160 (3)	29/05/1990	20	161.7	82.0	82.3	5.0	7.4	7.1	7.1	7.3	318.0
IPE 160 (3)	29/05/1990	21	162.0	81.9	82.0	5.0	7.3	7.0	7.2	7.3	320.0
IPE 160 (3)	29/05/1990	22	161.8	82.0	82.0	5.1	7.4	7.0	7.1	7.2	332.0
IPE 160 (3)	29/05/1990	23	161.8	82.3	82.0	5.1	7.4	7.1	7.3	7.1	325.0
IPE 160 (3)	29/05/1990	24	161.7	81.7	82.1	5.0	7.3	7.0	7.2	7.1	322.0
IPE 160 (3)	29/05/1990	25	162.0	81.5	82.4	5.1	7.5	7.1	7.3	7.3	323.0
IPE 160 (3)	29/05/1990	26	162.1	81.5	82.7	5.1	7.4	7.1	7.3	7.2	312.0
IPE 160 (3)	29/05/1990	27	162.2	81.3	82.7	4.9	7.5	7.2	7.2	7.2	317.0
IPE 160 (3)	29/05/1990	28	162.0	81.9	82.9	5.0	7.3	7.2	7.1	7.2	321.0
IPE 160 (3)	29/05/1990	29	161.8	81.8	82.7	5.1	7.2	7.1	7.0	7.3	334.0
IPE 160 (3)	29/05/1990	30	162.0	81.7	82.8	5.1	7.3	7.2	7.2	7.1	333.0
IPE 180 (1)	11/10/1989	1	179.9	90.0	89.6	5.4	7.8	7.7	7.3	7.8	344.0
IPE 180 (1)	11/10/1989	2	180.0	90.3	89.6	5.4	7.8	7.8	7.6	7.9	346.0
IPE 180 (1)	11/10/1989	3	179.8	90.8	90.6	5.5	7.9	7.7	7.7	7.9	343.0
IPE 180 (1)	11/10/1989	4	179.8	90.2	89.7	5.5	7.7	7.8	7.3	7.8	344.0
IPE 180 (1)	11/10/1989	5	179.8	90.3	80.6	5.5	8.0	7.8	7.6	8.0	367.0
IPE 180 (1)	11/10/1989	6	180.0	90.4	90.0	5.5	8.1	7.9	7.4	8.1	385.0
IPE 180 (1)	11/10/1989	7	180.0	90.0	89.8	5.5	7.8	7.6	7.4	7.9	352.0
IPE 180 (1)	11/10/1989	8	179.9	90.6	89.5	5.4	7.8	7.6	7.3	7.9	352.0
IPE 180 (1)	11/10/1989	9	180.0	91.2	90.8	5.4	7.8	7.8	7.4	7.9	343.0
IPE 180 (1)	11/10/1989	10	180.0	90.5	90.0	5.4	7.9	7.6	7.5	7.9	354.0
IPE 180 (1)	11/10/1989	11	179.1	91.4	91.0	5.1	7.9	7.1	7.1	8.2	324.0
IPE 180 (1)	11/10/1989	12	179.0	91.8	91.5	5.2	7.7	7.0	7.3	7.9	331.0
IPE 180 (1)	11/10/1989	13	179.3	91.0	90.6	5.2	8.0	7.0	7.2	8.2	331.0
IPE 180 (1)	11/10/1989	14	179.0	91.4	91.2	5.3	7.9	7.0	7.2	7.9	316.0
IPE 180 (1)	11/10/1989	15	179.3	91.5	91.2	5.1	8.0	6.9	7.2	8.0	333.0
IPE 180 (1)	11/10/1989	16	179.5	91.0	90.7	5.3	7.9	7.4	7.2	8.0	333.0
IPE 180 (1)	11/10/1989	17	179.4	91.9	91.0	5.3	7.9	7.1	7.2	7.9	338.0
IPE 180 (1)	11/10/1989	18	179.6	91.8	91.6	5.3	7.9	7.0	7.3	7.9	346.0
IPE 180 (1)	11/10/1989	19	179.6	91.2	90.8	5.3	8.0	7.0	7.2	8.1	336.0
IPE 180 (1)	11/10/1989	20	179.4	92.0	91.4	5.2	8.0	7.0	7.2	8.1	349.0
IPE 180 (1)	11/10/1989	21	179.4	91.7	91.5	5.4	8.2	8.1	7.7	8.1	368.0
IPE 180 (1)	11/10/1989	22	179.5	91.0	90.7	5.5	8.2	8.0	7.8	8.2	356.0
IPE 180 (1)	11/10/1989	23	179.6	91.3	91.2	5.5	8.2	8.2	7.4	8.0	380.0
IPE 180 (1)	11/10/1989	24	179.6	91.5	91.3	5.5	8.2	7.9	7.4	8.3	358.0
IPE 180 (1)	11/10/1989	25	179.8	91.0	91.0	5.4	8.3	7.8	8.0	8.2	348.0
IPE 180 (1)	11/10/1989	26	179.7	91.1	90.9	5.4	8.1	7.9	7.6	8.1	374.0
IPE 180 (1)	11/10/1989	27	179.5	91.9	91.4	5.5	8.1	7.9	7.5	8.3	359.0
IPE 180 (1)	11/10/1989	28	179.7	91.4	91.4	5.5	8.2	7.8	7.6	8.3	361.0
IPE 180 (1)	11/10/1989	29	179.7	91.4	91.3	5.5	8.2	7.9	7.7	8.1	378.0
IPE 180 (1)	11/10/1989	30	179.6	91.4	91.0	5.4	8.1	8.0	7.8	8.0	338.0
IPE 180 (2)	31/03/1990	1	183.0	90.7	90.6	5.6	7.5	7.8	7.8	7.6	330.0
IPE 180 (2)	31/03/1990	2	182.9	90.8	90.5	5.5	7.6	7.9	7.7	7.7	327.0
IPE 180 (2)	31/03/1990	3	182.9	90.6	90.6	5.5	7.7	7.8	7.6	7.8	348.0
IPE 180 (2)	31/03/1990	4	182.8	90.7	90.7	5.6	7.6	7.9	7.7	7.9	339.0
IPE 180 (2)	31/03/1990	5	182.9	90.8	90.7	5.6	7.6	7.7	7.7	7.7	335.0
IPE 180 (2)	31/03/1990	6	182.8	90.8	90.8	5.5	7.5	7.8	7.7	7.8	327.0
IPE 180 (2)	31/03/1990	7	183.0	91.0	90.7	5.6	7.6	7.9	7.8	7.7	349.0
IPE 180 (2)	31/03/1990	8	182.9	91.2	90.8	5.6	7.7	7.8	7.7	7.8	352.0
IPE 180 (2)	31/03/1990	9	182.7	91.0	90.8	5.5	7.6	7.8	7.7	7.7	345.0
IPE 180 (2)	31/03/1990	10	182.7	90.8	90.9	5.5	7.6	7.9	7.9	7.7	338.0
IPE 180 (2)	31/03/1990	11	182.6	90.6	91.4	5.6	7.6	8.0	8.1	7.6	342.0
IPE 180 (2)	31/03/1990	12	182.7	90.7	91.4	5.6	7.6	8.0	8.0	7.7	335.0
IPE 180 (2)	31/03/1990	13	182.7	90.8	91.3	5.5	7.7	8.1	8.0	7.7	327.0

IPE 180 (2)	31/03/1990	14	182.7	90.7	91.0	5.4	7.6	8.0	7.9	7.7	326.0
IPE 180 (2)	31/03/1990	15	182.7	90.7	91.2	5.4	7.6	8.0	7.9	7.7	345.0
IPE 180 (2)	31/03/1990	16	182.6	90.6	91.2	5.6	7.5	7.9	7.9	7.6	351.0
IPE 180 (2)	31/03/1990	17	182.6	90.6	91.2	5.6	7.6	7.9	7.8	7.6	354.0
IPE 180 (2)	31/03/1990	18	182.7	90.6	91.1	5.5	7.7	7.9	7.9	7.7	350.0
IPE 180 (2)	31/03/1990	19	182.8	90.4	91.2	5.4	7.7	8.0	7.9	7.7	345.0
IPE 180 (2)	31/03/1990	20	182.6	90.4	90.9	5.4	7.6	8.0	7.8	7.6	348.0
IPE 180 (2)	31/03/1990	21	182.5	90.5	90.8	5.4	7.6	8.1	8.0	7.5	355.0
IPE 180 (2)	31/03/1990	22	182.6	90.7	90.7	5.3	7.7	8.0	8.0	7.6	328.0
IPE 180 (2)	31/03/1990	23	182.6	90.7	90.8	5.3	7.7	8.0	7.9	7.7	342.0
IPE 180 (2)	31/03/1990	24	182.7	91.0	90.8	5.4	7.6	8.0	7.9	7.6	337.0
IPE 180 (2)	31/03/1990	25	182.5	90.8	91.0	5.5	7.7	8.0	8.0	7.6	340.0
IPE 180 (2)	31/03/1990	26	182.6	91.1	90.8	5.4	7.6	7.9	8.0	7.6	351.0
IPE 180 (2)	31/03/1990	27	182.6	91.0	90.7	5.4	7.7	7.9	7.8	7.8	332.0
IPE 180 (2)	31/03/1990	28	182.7	90.8	90.7	5.4	7.7	8.0	7.7	7.8	350.0
IPE 180 (2)	31/03/1990	29	182.8	90.8	90.7	5.3	7.6	8.0	7.9	7.8	338.0
IPE 180 (2)	31/03/1990	30	182.7	90.8	90.7	5.4	7.6	8.0	7.9	7.6	342.0
IPE 180 (3)	01/06/1990	1	182.3	90.3	89.3	5.6	7.4	7.9	7.6	7.8	321.0
IPE 180 (3)	01/06/1990	2	182.5	90.4	89.5	5.6	7.4	7.9	7.6	7.7	327.0
IPE 180 (3)	01/06/1990	3	182.4	90.4	89.7	5.6	7.4	7.9	7.7	7.9	320.0
IPE 180 (3)	01/06/1990	4	182.4	90.5	89.7	5.5	7.5	7.7	7.6	7.7	332.0
IPE 180 (3)	01/06/1990	5	182.5	90.5	90.2	5.5	7.5	7.7	7.6	7.8	345.0
IPE 180 (3)	01/06/1990	6	182.5	90.5	90.1	5.5	7.5	7.8	7.6	7.8	335.0
IPE 180 (3)	01/06/1990	7	182.4	90.7	90.2	5.5	7.5	7.8	7.5	7.9	334.0
IPE 180 (3)	01/06/1990	8	182.3	90.7	90.4	5.6	7.5	7.9	7.7	7.9	329.0
IPE 180 (3)	01/06/1990	9	182.3	90.7	90.3	5.6	7.6	7.9	7.6	7.8	325.0
IPE 180 (3)	01/06/1990	10	182.5	90.7	90.3	5.6	7.6	7.9	7.7	7.7	340.0
IPE 180 (3)	01/06/1990	11	183.0	91.5	91.0	5.6	7.4	7.8	7.6	7.7	332.0
IPE 180 (3)	01/06/1990	12	182.7	91.2	90.9	5.5	7.5	7.6	7.7	7.8	348.0
IPE 180 (3)	01/06/1990	13	182.7	91.2	90.9	5.5	7.6	7.7	7.7	7.8	340.0
IPE 180 (3)	01/06/1990	14	182.8	91.2	91.0	5.5	7.8	7.7	7.6	7.9	328.0
IPE 180 (3)	01/06/1990	15	182.6	91.4	91.0	5.5	7.6	7.9	7.7	7.7	332.0
IPE 180 (3)	01/06/1990	16	182.7	91.4	91.1	5.4	7.5	7.9	7.8	7.8	350.0
IPE 180 (3)	01/06/1990	17	182.8	91.0	90.9	5.5	7.7	7.8	7.8	7.8	337.0
IPE 180 (3)	01/06/1990	18	182.5	91.0	90.9	5.4	7.7	7.7	7.7	7.7	328.0
IPE 180 (3)	01/06/1990	19	182.6	91.1	90.9	5.4	7.8	7.8	7.6	7.7	342.0
IPE 180 (3)	01/06/1990	20	182.6	91.1	90.8	5.4	7.9	7.8	7.6	7.8	344.0
IPE 180 (3)	01/06/1990	21	182.3	90.9	90.8	5.3	8.6	7.8	7.7	8.6	337.0
IPE 180 (3)	01/06/1990	22	182.4	90.8	90.7	5.4	8.5	7.6	7.7	8.4	328.0
IPE 180 (3)	01/06/1990	23	182.3	90.7	90.7	5.3	8.3	7.8	7.9	8.4	345.0
IPE 180 (3)	01/06/1990	24	182.5	90.7	90.7	5.4	8.5	7.9	7.7	8.3	341.0
IPE 180 (3)	01/06/1990	25	182.5	90.8	90.7	5.4	8.5	7.9	7.8	8.3	336.0
IPE 180 (3)	01/06/1990	26	182.3	90.8	90.9	5.4	8.0	7.9	7.8	8.2	340.0
IPE 180 (3)	01/06/1990	27	182.4	90.7	90.9	5.5	8.1	8.0	7.9	8.2	335.0
IPE 180 (3)	01/06/1990	28	182.3	90.8	90.8	5.4	8.1	7.8	7.9	8.1	339.0
IPE 180 (3)	01/06/1990	29	182.3	90.7	90.8	5.4	8.0	7.8	7.8	8.1	339.0
IPE 180 (3)	01/06/1990	30	182.5	90.7	90.8	5.4	8.0	7.8	7.7	8.2	345.0
IPE 240 (1)	21/09/1989	1	238.7	120.1	119.7	6.2	9.9	9.0	9.5	9.5	318.0
IPE 240 (1)	21/09/1989	2	238.8	120.1	119.1	6.1	9.9	9.1	9.4	9.4	319.0
IPE 240 (1)	21/09/1989	3	238.7	120.2	119.0	6.1	10.0	9.1	9.7	9.5	320.0
IPE 240 (1)	21/09/1989	4	238.9	120.0	118.2	6.2	10.2	9.4	9.9	9.4	317.0
IPE 240 (1)	21/09/1989	5	238.6	119.6	119.6	6.2	10.1	9.2	9.9	9.3	322.0
IPE 240 (1)	21/09/1989	6	238.7	120.0	119.2	6.1	10.0	9.2	9.6	9.7	322.0
IPE 240 (1)	21/09/1989	7	239.5	120.1	119.9	6.2	10.0	9.6	10.0	9.7	315.0
IPE 240 (1)	21/09/1989	8	239.0	119.9	119.0	6.2	10.1	9.4	10.1	9.5	323.0
IPE 240 (1)	21/09/1989	9	238.6	119.5	119.3	6.1	9.7	9.6	10.1	9.3	318.0
IPE 240 (1)	21/09/1989	10	238.6	120.1	119.1	6.1	9.7	9.6	9.9	9.3	318.0
IPE 240 (1)	21/09/1989	11	238.6	120.2	118.3	6.1	10.0	9.8	10.0	9.5	335.0
IPE 240 (1)	21/09/1989	12	238.5	120.1	118.9	6.2	10.0	9.5	9.9	9.4	358.0
IPE 240 (1)	21/09/1989	13	238.1	120.2	119.2	6.2	10.0	9.3	9.8	9.5	329.0
IPE 240 (1)	21/09/1989	14	238.5	120.2	119.0	6.2	10.0	9.4	10.0	9.4	357.0
IPE 240 (1)	21/09/1989	15	238.6	120.1	119.2	6.1	10.0	9.5	9.9	9.7	326.0
IPE 240 (1)	21/09/1989	16	238.5	120.0	118.5	6.2	9.9	9.6	9.9	9.4	330.0
IPE 240 (1)	21/09/1989	17	239.0	120.0	118.3	6.1	9.9	9.9	9.9	9.6	313.0
IPE 240 (1)	21/09/1989	18	238.7	120.2	118.0	6.2	9.9	9.6	9.9	9.7	354.0
IPE 240 (1)	21/09/1989	19	238.5	119.9	119.0	6.1	9.9	9.4	9.9	9.7	345.0
IPE 240 (1)	21/09/1989	20	238.6	120.2	118.2	6.1	9.8	9.5	9.8	9.7	329.0
IPE 240 (1)	21/09/1989	21	238.2	120.2	118.7	6.2	9.9	9.8	9.8	9.7	292.0
IPE 240 (1)	21/09/1989	22	238.0	120.0	119.6	6.4	10.0	9.4	10.3	9.5	293.0
IPE 240 (1)	21/09/1989	23	238.0	120.0	118.1	6.4	10.3	9.3	10.1	9.5	288.0
IPE 240 (1)	21/09/1989	24	238.1	120.5	118.5	6.4	10.4	9.2	10.1	9.6	294.0
IPE 240 (1)	21/09/1989	25	238.4	120.9	117.9	6.3	10.1	9.5	10.2	9.7	296.0
IPE 240 (1)	21/09/1989	26	238.3	120.6	117.7	6.3	10.1	9.6	9.9	9.8	305.0
IPE 240 (1)	21/09/1989	27	238.4	120.0	119.0	6.4	10.1	9.6	9.6	9.8	299.0
IPE 240 (1)	21/09/1989	28	238.5	120.6	118.0	6.4	9.9	9.6	10.1	9.7	299.0

IPE 240 (1)	21/09/1989	29	238.3	120.6	117.6	6.4	9.8	9.6	10.2	9.5	297.0
IPE 240 (1)	21/09/1989	30	238.5	120.6	117.7	6.4	10.2	9.7	10.1	9.5	290.0
IPE 270 (1)	09/11/1989	1	270.4	133.0	133.2	6.6	9.9	10.3	10.6	10.5	321.0
IPE 270 (1)	09/11/1989	2	270.7	133.6	133.8	6.6	9.9	10.2	10.5	10.5	311.0
IPE 270 (1)	09/11/1989	3	270.1	135.0	134.4	6.5	10.2	9.6	10.5	10.5	317.0
IPE 270 (1)	09/11/1989	4	270.0	133.3	133.4	6.5	10.1	10.2	10.5	10.5	319.0
IPE 270 (1)	09/11/1989	5	269.8	132.4	133.2	6.6	10.3	10.7	9.3	10.3	288.0
IPE 270 (1)	09/11/1989	6	270.2	133.9	133.8	6.6	10.0	9.9	10.5	10.3	320.0
IPE 270 (1)	09/11/1989	7	269.6	134.0	133.2	6.7	10.5	10.5	10.1	9.6	312.0
IPE 270 (1)	09/11/1989	8	270.1	133.0	132.6	6.7	10.3	9.6	10.5	10.4	312.0
IPE 270 (1)	09/11/1989	9	269.5	134.3	134.8	6.7	10.0	10.2	10.6	10.6	340.0
IPE 270 (1)	09/11/1989	10	269.2	134.2	134.3	6.9	10.3	10.1	10.7	10.3	280.0
IPE 270 (1)	09/11/1989	11	270.0	133.9	134.0	6.7	10.7	10.1	10.1	9.8	266.0
IPE 270 (1)	09/11/1989	12	269.6	134.7	134.1	6.8	10.6	10.4	10.0	9.9	288.0
IPE 270 (1)	09/11/1989	13	269.9	133.9	133.6	6.8	10.3	9.9	10.7	10.1	300.0
IPE 270 (1)	09/11/1989	14	270.0	134.0	134.4	6.9	10.8	10.2	10.0	10.0	281.0
IPE 270 (1)	09/11/1989	15	269.9	134.4	134.0	6.8	10.1	10.9	10.0	10.0	276.0
IPE 270 (1)	09/11/1989	16	269.3	134.3	134.3	6.9	10.2	10.1	10.7	10.3	271.0
IPE 270 (1)	09/11/1989	17	269.5	134.8	134.3	6.9	10.1	10.9	10.0	10.1	289.0
IPE 270 (1)	09/11/1989	18	269.5	133.3	134.0	6.9	10.0	10.1	10.2	10.3	282.0
IPE 270 (1)	09/11/1989	19	269.7	133.2	133.7	6.9	10.4	10.6	10.1	10.0	290.0
IPE 270 (1)	09/11/1989	20	270.3	134.5	135.0	7.6	9.8	10.0	10.8	9.8	280.0
IPE 270 (1)	09/11/1989	21	269.1	134.5	134.3	7.6	9.8	10.0	10.7	9.9	282.0
IPE 270 (1)	09/11/1989	22	268.5	134.5	134.3	7.6	11.1	9.7	9.6	9.8	269.0
IPE 270 (1)	09/11/1989	23	269.3	134.5	134.0	7.8	9.9	10.0	10.6	10.0	281.0
IPE 270 (1)	09/11/1989	24	270.2	134.5	134.8	7.8	10.0	9.6	9.9	10.6	275.0
IPE 270 (1)	09/11/1989	25	270.5	134.0	134.4	7.7	9.8	10.8	9.7	9.5	290.0
IPE 270 (1)	09/11/1989	26	269.3	135.0	135.3	8.1	10.3	10.8	10.2	9.8	284.0
IPE 270 (1)	09/11/1989	27	270.0	135.8	136.2	8.2	10.3	10.8	10.3	9.6	313.0
IPE 270 (1)	09/11/1989	28	269.3	135.2	134.8	8.1	10.0	10.3	10.7	10.2	288.0
IPE 270 (1)	09/11/1989	29	269.5	135.5	135.3	8.2	10.4	9.8	10.2	10.7	277.0
IPE 270 (2)	03/04/1990	1	271.0	138.0	137.0	6.2	9.8	9.4	9.7	10.3	275.0
IPE 270 (2)	03/04/1990	2	270.8	137.2	137.1	6.3	9.8	9.5	9.6	10.2	269.0
IPE 270 (2)	03/04/1990	3	271.0	137.5	137.0	6.3	9.9	9.6	9.7	10.1	288.0
IPE 270 (2)	03/04/1990	4	270.9	137.5	136.9	6.2	9.8	9.6	9.7	10.2	298.0
IPE 270 (2)	03/04/1990	5	270.9	137.8	136.8	6.3	9.8	9.6	9.7	10.2	304.0
IPE 270 (2)	03/04/1990	6	271.0	137.5	136.9	6.3	9.9	9.6	9.8	10.2	312.0
IPE 270 (2)	03/04/1990	7	270.9	137.6	136.2	6.3	9.8	9.5	9.8	10.2	292.0
IPE 270 (2)	03/04/1990	8	270.9	137.4	136.4	6.4	9.8	9.5	9.8	10.3	277.0
IPE 270 (2)	03/04/1990	9	271.0	137.0	135.9	6.3	9.9	9.6	9.7	10.1	295.0
IPE 270 (2)	03/04/1990	10	271.0	137.0	136.3	6.4	9.9	9.5	9.7	10.1	302.0
IPE 270 (2)	03/04/1990	11	271.5	136.0	134.2	6.4	10.0	10.1	10.2	10.0	318.0
IPE 270 (2)	03/04/1990	12	271.2	136.2	135.5	6.4	10.0	10.0	10.1	10.1	300.0
IPE 270 (2)	03/04/1990	13	271.3	136.4	134.9	6.4	9.9	10.0	10.1	10.0	295.0
IPE 270 (2)	03/04/1990	14	271.2	135.9	135.0	6.3	9.9	10.0	10.0	10.0	279.0
IPE 270 (2)	03/04/1990	15	271.2	136.0	135.2	6.4	10.0	10.0	10.1	10.1	285.0
IPE 270 (2)	03/04/1990	16	271.4	136.2	135.2	6.5	10.1	10.1	10.1	10.2	282.0
IPE 270 (2)	03/04/1990	17	271.3	136.4	135.0	6.4	10.1	10.0	10.0	10.2	297.0
IPE 270 (2)	03/04/1990	18	271.3	135.8	135.1	6.4	10.1	10.0	10.0	10.2	301.0
IPE 270 (2)	03/04/1990	19	271.3	135.7	134.8	6.4	10.3	10.1	10.0	10.1	299.0
IPE 270 (2)	03/04/1990	20	271.2	135.5	134.7	6.4	10.1	10.1	9.9	10.1	287.0
IPE 270 (2)	03/04/1990	21	271.5	135.2	134.2	6.4	10.4	9.5	9.2	10.2	300.0
IPE 270 (2)	03/04/1990	22	271.4	135.5	134.5	6.5	10.4	9.7	9.4	10.1	292.0
IPE 270 (2)	03/04/1990	23	271.3	136.0	134.7	6.5	10.3	9.6	9.3	10.1	274.0
IPE 270 (2)	03/04/1990	24	271.4	135.8	135.0	6.4	10.3	9.7	9.4	10.2	304.0
IPE 270 (2)	03/04/1990	25	271.4	135.7	135.2	6.5	10.4	9.7	9.5	10.2	308.0
IPE 270 (2)	03/04/1990	26	271.4	135.8	134.8	6.4	10.3	9.6	9.6	10.0	290.0
IPE 270 (2)	03/04/1990	27	271.5	136.2	134.7	6.4	10.3	9.6	9.5	10.0	275.0
IPE 270 (2)	03/04/1990	28	271.3	136.4	134.6	6.5	10.3	9.7	9.5	10.0	277.0
IPE 270 (2)	03/04/1990	29	271.4	135.9	134.6	6.4	10.2	9.7	9.6	10.1	280.0
IPE 270 (2)	03/04/1990	30	271.4	136.0	134.6	6.4	10.3	9.8	9.6	10.1	281.0
IPE 270 (3)	06/07/1990	1	269.5	136.0	137.7	6.3	9.6	10.5	10.6	10.9	270.0
IPE 270 (3)	06/07/1990	2	269.5	135.9	137.5	6.3	9.7	10.4	10.5	10.8	275.0
IPE 270 (3)	06/07/1990	3	269.5	136.0	137.2	6.4	9.7	10.4	10.5	10.7	275.0
IPE 270 (3)	06/07/1990	4	269.6	136.2	137.3	6.4	9.7	10.5	10.5	10.7	282.0
IPE 270 (3)	06/07/1990	5	269.5	136.0	137.3	6.3	9.6	10.5	10.6	10.7	290.0
IPE 270 (3)	06/07/1990	6	269.5	136.4	136.5	6.4	9.6	10.5	10.6	10.8	272.0
IPE 270 (3)	06/07/1990	7	269.6	136.3	136.5	6.9	9.6	10.4	10.5	10.7	272.0
IPE 270 (3)	06/07/1990	8	269.6	136.5	137.2	6.4	9.5	10.5	10.5	10.6	280.0
IPE 270 (3)	06/07/1990	9	269.5	136.5	137.0	6.4	9.6	10.5	10.4	10.6	278.0
IPE 270 (3)	06/07/1990	10	269.5	135.9	136.8	6.4	9.6	10.5	10.5	10.6	278.0
IPE 270 (3)	06/07/1990	11	269.2	134.0	134.9	6.5	9.4	10.4	9.9	10.4	285.0
IPE 270 (3)	06/07/1990	12	269.2	134.3	134.5	6.5	9.5	10.3	10.0	10.4	292.0
IPE 270 (3)	06/07/1990	13	269.2	134.0	134.8	6.6	9.5	10.3	10.0	10.4	294.0
IPE 270 (3)	06/07/1990	14	269.1	134.2	134.2	6.5	9.6	10.2	9.9	10.3	292.0

IPE 270 (3)	06/07/1990	15	269.2	134.4	134.8	6.5	9.6	10.4	9.9	10.3	287.0
IPE 270 (3)	06/07/1990	16	269.2	134.7	134.9	6.4	9.6	10.4	9.9	10.4	305.0
IPE 270 (3)	06/07/1990	17	269.2	134.7	134.9	6.5	9.5	10.4	10.0	10.4	302.0
IPE 270 (3)	06/07/1990	18	269.2	135.0	135.2	6.5	9.5	10.3	10.0	10.3	278.0
IPE 270 (3)	06/07/1990	19	269.3	135.0	135.5	6.5	9.6	10.3	10.0	10.3	290.0
IPE 270 (3)	06/07/1990	20	269.2	135.0	134.9	6.5	9.6	10.3	10.0	10.4	290.0
IPE 270 (3)	06/07/1990	21	269.5	135.9	135.5	6.6	10.7	10.1	10.0	10.2	305.0
IPE 270 (3)	06/07/1990	22	269.6	135.8	135.7	6.6	10.6	10.0	10.2	10.2	300.0
IPE 270 (3)	06/07/1990	23	269.5	135.9	135.6	6.6	10.5	10.1	10.1	10.1	310.0
IPE 270 (3)	06/07/1990	24	269.6	135.8	135.6	6.5	10.6	10.0	10.2	10.2	301.0
IPE 270 (3)	06/07/1990	25	269.5	135.8	135.2	6.6	10.6	10.2	10.2	10.2	295.0
IPE 270 (3)	06/07/1990	26	269.5	136.0	135.8	6.5	10.6	10.1	10.1	10.1	298.0
IPE 270 (3)	06/07/1990	27	269.6	136.1	135.9	6.5	10.7	10.1	10.0	10.1	295.0
IPE 270 (3)	06/07/1990	28	269.6	136.1	135.8	6.5	10.6	10.0	10.1	10.1	300.0
IPE 270 (3)	06/07/1990	29	269.8	136.1	135.6	6.5	10.5	10.0	10.1	10.2	290.0
IPE 270 (3)	06/07/1990	30	269.7	136.1	135.4	6.6	10.6	10.0	10.1	10.2	288.0
HEB 100B (1)	08/09/1989	1	101.6	99.0	98.2	6.0	9.8	9.9	9.7	9.9	335.0
HEB 100B (1)	08/09/1989	2	101.7	99.0	99.0	5.9	9.7	10.1	9.7	9.8	331.0
HEB 100B (1)	08/09/1989	3	101.9	98.8	98.2	6.0	9.7	10.0	9.8	9.8	327.0
HEB 100B (1)	08/09/1989	4	101.6	98.3	97.4	6.0	9.7	9.9	9.9	9.8	320.0
HEB 100B (1)	08/09/1989	5	102.0	99.1	98.8	6.0	9.7	9.9	9.9	9.7	318.0
HEB 100B (1)	08/09/1989	6	101.8	97.7	97.2	6.0	9.7	9.9	9.8	9.9	329.0
HEB 100B (1)	08/09/1989	7	101.8	98.8	98.6	6.0	9.8	9.8	9.6	9.9	335.0
HEB 100B (1)	08/09/1989	8	101.6	98.7	98.5	5.9	9.8	10.0	9.5	9.8	329.0
HEB 100B (1)	08/09/1989	9	101.8	98.0	97.2	6.0	9.5	10.0	9.7	9.8	320.0
HEB 100B (1)	08/09/1989	10	101.7	98.4	97.9	6.0	9.6	9.9	9.7	9.9	326.0
HEB 100B (1)	08/09/1989	11	101.1	100.0	99.2	6.0	9.7	9.8	9.5	9.6	322.0
HEB 100B (1)	08/09/1989	12	101.1	100.9	100.5	6.1	9.6	9.9	9.5	9.6	328.0
HEB 100B (1)	08/09/1989	13	100.8	100.4	99.8	6.0	9.8	9.7	9.5	9.6	325.0
HEB 100B (1)	08/09/1989	14	101.2	100.0	99.8	6.1	9.7	9.7	9.7	9.7	331.0
HEB 100B (1)	08/09/1989	15	101.1	101.0	100.4	6.1	9.7	10.0	9.6	9.7	328.0
HEB 100B (1)	08/09/1989	16	101.2	100.4	99.8	6.1	9.7	9.9	9.6	9.8	338.0
HEB 100B (1)	08/09/1989	17	101.3	101.8	101.5	6.2	9.7	10.1	9.7	9.7	314.0
HEB 100B (1)	08/09/1989	18	101.6	102.9	102.6	6.1	9.9	10.0	9.7	9.9	327.0
HEB 100B (1)	08/09/1989	19	101.7	101.3	101.0	6.1	9.7	9.8	9.9	9.9	325.0
HEB 100B (1)	08/09/1989	20	101.4	101.2	101.2	6.0	9.9	10.0	9.6	9.8	319.0
HEB 100B (1)	08/09/1989	21	100.8	100.9	100.6	5.9	9.7	9.8	9.5	9.6	313.0
HEB 100B (1)	08/09/1989	22	101.7	99.2	98.7	5.8	9.7	9.7	9.5	9.8	314.0
HEB 100B (1)	08/09/1989	23	100.8	99.6	99.0	5.9	9.7	9.9	9.5	9.7	317.0
HEB 100B (1)	08/09/1989	24	101.7	99.9	99.6	5.9	9.6	9.8	9.6	9.6	315.0
HEB 100B (1)	08/09/1989	25	101.0	100.0	99.8	5.9	9.8	9.8	9.5	9.7	317.0
HEB 100B (1)	08/09/1989	26	101.0	99.4	99.2	5.9	9.8	9.8	9.5	9.7	315.0
HEB 100B (1)	08/09/1989	27	100.8	100.0	99.7	6.0	9.8	9.8	9.5	9.6	313.0
HEB 100B (1)	08/09/1989	28	100.8	99.3	98.2	5.9	9.7	9.8	9.5	9.6	319.0
HEB 100B (1)	08/09/1989	29	100.8	100.2	99.7	5.9	9.6	9.8	9.4	9.7	316.0
HEB 100B (1)	08/09/1989	30	101.8	100.0	100.0	5.9	9.7	9.8	9.4	9.7	315.0
HEB 100B (2)	07/11/1989	1	102.2	101.6	100.0	5.7	9.7	10.0	10.0	9.8	335.0
HEB 100B (2)	07/11/1989	2	102.1	101.2	100.0	5.8	9.8	9.9	10.0	9.8	332.0
HEB 100B (2)	07/11/1989	3	102.1	101.0	100.9	5.8	9.8	10.0	9.9	9.9	333.0
HEB 100B (2)	07/11/1989	4	102.0	101.1	100.2	5.7	9.7	10.0	10.0	9.8	328.0
HEB 100B (2)	07/11/1989	5	102.1	100.7	100.5	5.7	9.8	9.9	9.9	9.9	319.0
HEB 100B (2)	07/11/1989	6	102.0	101.0	100.3	5.8	9.7	10.0	10.0	9.8	340.0
HEB 100B (2)	07/11/1989	7	102.2	101.2	100.3	5.8	9.8	10.0	10.1	9.8	329.0
HEB 100B (2)	07/11/1989	8	102.0	101.0	100.5	5.8	9.7	10.1	9.8	9.8	330.0
HEB 100B (2)	07/11/1989	9	102.1	101.2	100.4	5.7	9.8	9.9	10.0	9.8	334.0
HEB 100B (2)	07/11/1989	10	102.2	100.8	100.8	5.8	9.7	9.9	10.1	9.9	328.0
HEB 100B (2)	07/11/1989	11	101.6	100.7	99.8	6.1	9.7	9.9	10.0	9.4	337.0
HEB 100B (2)	07/11/1989	12	101.4	100.6	100.2	6.0	9.6	10.0	10.0	9.5	304.0
HEB 100B (2)	07/11/1989	13	101.6	100.5	100.2	6.1	9.6	10.0	10.0	9.5	310.0
HEB 100B (2)	07/11/1989	14	101.6	100.8	99.8	6.0	9.5	10.1	10.1	9.6	323.0
HEB 100B (2)	07/11/1989	15	101.6	100.6	100.1	6.1	9.6	10.0	10.0	9.5	308.0
HEB 100B (2)	07/11/1989	16	101.5	100.7	100.0	6.1	9.5	10.0	10.1	9.4	303.0
HEB 100B (2)	07/11/1989	17	101.4	100.4	100.0	6.0	9.6	10.1	10.0	9.5	321.0
HEB 100B (2)	07/11/1989	18	101.4	100.7	100.1	6.0	9.5	10.2	10.0	9.5	309.0
HEB 100B (2)	07/11/1989	19	101.5	100.4	100.0	6.0	9.5	10.0	10.1	9.4	322.0
HEB 100B (2)	07/11/1989	20	101.5	100.7	100.1	6.0	9.6	10.0	10.1	9.6	314.0
HEB 100B (2)	07/11/1989	21	101.6	100.8	99.2	6.2	9.8	10.2	10.0	10.1	317.0
HEB 100B (2)	07/11/1989	22	101.6	100.7	99.7	6.2	9.8	10.1	10.0	10.1	312.0
HEB 100B (2)	07/11/1989	23	101.6	100.8	99.2	6.2	9.7	10.1	10.1	10.0	322.0
HEB 100B (2)	07/11/1989	24	101.5	100.2	100.0	6.2	9.7	10.1	10.1	10.1	327.0
HEB 100B (2)	07/11/1989	25	101.5	99.7	99.4	6.1	9.7	10.0	10.1	10.1	315.0
HEB 100B (2)	07/11/1989	26	101.5	100.1	99.6	6.1	9.8	10.0	10.1	10.1	323.0
HEB 100B (2)	07/11/1989	27	101.4	100.5	98.8	6.1	9.7	10.0	10.1	10.0	313.0
HEB 100B (2)	07/11/1989	28	101.3	100.7	99.4	6.2	9.6	10.0	10.0	10.0	318.0
HEB 100B (2)	07/11/1989	29	101.4	100.2	99.3	6.1	9.7	10.1	10.0	10.1	321.0

HEB 100B (2)	07/11/1989	30	101.4	100.2	99.0	6.1	9.6	10.0	10.0	10.1	326.0
HEB 100B (3)	03/07/1990	1	102.4	99.8	99.0	5.8	10.0	9.8	10.0	9.8	325.0
HEB 100B (3)	03/07/1990	2	102.5	100.0	99.2	5.8	10.1	9.9	9.9	9.9	328.0
HEB 100B (3)	03/07/1990	3	102.4	100.0	99.2	5.7	10.0	9.8	9.8	9.8	315.0
HEB 100B (3)	03/07/1990	4	102.4	99.9	99.4	5.7	10.0	9.8	9.9	9.8	320.0
HEB 100B (3)	03/07/1990	5	102.4	100.1	99.8	5.7	9.9	9.9	9.9	9.9	321.0
HEB 100B (3)	03/07/1990	6	102.5	100.2	99.6	5.6	9.9	9.8	10.0	10.0	338.0
HEB 100B (3)	03/07/1990	7	102.5	100.3	99.4	5.6	9.8	9.7	10.0	10.0	324.0
HEB 100B (3)	03/07/1990	8	102.7	100.3	99.6	5.7	9.9	9.7	10.0	10.1	320.0
HEB 100B (3)	03/07/1990	9	102.5	100.3	99.7	5.7	9.9	9.7	10.0	9.9	322.0
HEB 100B (3)	03/07/1990	10	102.2	100.3	99.7	5.7	9.9	9.7	9.9	9.9	312.0
HEB 100B (3)	03/07/1990	11	101.7	100.6	100.0	5.5	9.9	9.4	9.2	9.7	318.0
HEB 100B (3)	03/07/1990	12	101.8	100.5	100.2	5.5	9.2	9.5	9.2	9.6	335.0
HEB 100B (3)	03/07/1990	13	101.6	100.7	99.8	5.6	9.1	9.5	9.2	9.7	338.0
HEB 100B (3)	03/07/1990	14	101.8	100.8	99.9	5.6	9.3	9.5	9.3	9.7	330.0
HEB 100B (3)	03/07/1990	15	101.8	100.8	99.7	5.6	9.4	9.5	9.5	9.6	322.0
HEB 100B (3)	03/07/1990	16	101.8	101.0	99.6	5.5	9.4	9.5	9.6	9.5	317.0
HEB 100B (3)	03/07/1990	17	102.0	101.0	99.7	5.5	9.4	9.6	9.6	9.7	340.0
HEB 100B (3)	03/07/1990	18	102.0	100.8	99.9	5.6	9.5	9.5	9.7	9.7	328.0
HEB 100B (3)	03/07/1990	19	102.0	100.6	99.7	5.6	9.5	9.5	9.7	9.7	333.0
HEB 100B (3)	03/07/1990	20	102.0	100.7	99.6	5.6	9.5	9.5	9.7	9.7	326.0
HEB 100B (3)	03/07/1990	21	102.2	100.3	99.5	5.6	9.5	9.8	9.8	10.0	330.0
HEB 100B (3)	03/07/1990	22	102.2	100.5	99.2	5.6	9.6	9.7	9.7	10.1	332.0
HEB 100B (3)	03/07/1990	23	102.0	100.4	99.6	5.7	9.6	9.7	9.8	10.0	318.0
HEB 100B (3)	03/07/1990	24	102.2	100.4	99.7	5.7	9.6	9.8	9.7	10.0	319.0
HEB 100B (3)	03/07/1990	25	102.2	100.5	99.8	5.6	9.7	9.8	9.7	10.0	325.0
HEB 100B (3)	03/07/1990	26	102.0	100.5	100.2	5.6	9.6	9.8	9.6	9.8	342.0
HEB 100B (3)	03/07/1990	27	102.1	100.2	100.2	5.7	9.7	9.9	9.7	9.8	338.0
HEB 100B (3)	03/07/1990	28	102.1	100.3	99.8	5.6	9.6	9.9	9.7	9.8	337.0
HEB 100B (3)	03/07/1990	29	102.0	100.2	99.9	5.6	9.6	9.8	9.6	9.8	337.0
HEB 100B (3)	03/07/1990	30	102.0	100.2	99.8	5.5	9.6	9.8	9.6	9.9	330.0
HEB 140B (1)	08/09/1989	1	143.0	137.7	139.6	7.0	11.4	11.7	12.0	11.9	298.0
HEB 140B (1)	08/09/1989	2	143.0	137.9	138.0	6.9	11.4	11.7	12.0	11.8	227.0
HEB 140B (1)	08/09/1989	3	143.0	138.1	138.0	7.2	11.5	11.8	12.0	11.8	224.0
HEB 140B (1)	08/09/1989	4	143.0	139.4	139.0	7.0	11.3	11.9	11.9	12.0	320.0
HEB 140B (1)	08/09/1989	5	142.9	140.4	139.4	7.0	11.4	11.8	12.0	12.1	328.0
HEB 140B (1)	08/09/1989	6	143.0	140.0	139.0	6.9	11.3	11.9	11.8	12.0	341.0
HEB 140B (1)	08/09/1989	7	143.2	138.0	138.4	7.0	11.6	11.9	12.0	11.9	337.0
HEB 140B (1)	08/09/1989	8	143.6	137.8	138.0	7.0	11.3	11.7	12.0	11.9	332.0
HEB 140B (1)	08/09/1989	9	143.0	138.1	138.0	6.9	11.5	11.8	12.0	11.8	332.0
HEB 140B (1)	08/09/1989	10	143.0	138.0	138.0	6.9	11.4	11.8	12.0	11.9	328.0
HEB 140B (1)	08/09/1989	11	142.9	139.0	139.1	7.0	11.4	11.9	12.0	12.0	353.0
HEB 140B (1)	08/09/1989	12	142.9	139.2	139.2	6.9	11.4	11.8	12.0	12.0	336.0
HEB 140B (1)	08/09/1989	13	143.0	140.1	140.9	7.0	11.9	12.0	12.0	11.6	350.0
HEB 140B (1)	08/09/1989	14	142.8	140.0	141.0	6.9	11.9	12.0	11.7	11.4	352.0
HEB 140B (1)	08/09/1989	15	143.7	140.1	141.0	6.9	11.9	12.0	11.8	11.6	347.0
HEB 140B (1)	08/09/1989	16	143.2	139.6	140.2	6.8	11.8	12.0	11.8	11.5	340.0
HEB 140B (1)	08/09/1989	17	143.2	140.5	139.7	7.0	11.8	12.0	11.6	11.4	343.0
HEB 140B (1)	08/09/1989	18	143.5	139.0	139.0	6.9	11.9	12.0	11.7	11.5	341.0
HEB 140B (1)	08/09/1989	19	143.2	139.0	139.0	6.8	11.9	12.0	11.8	11.6	331.0
HEB 140B (1)	08/09/1989	20	142.9	139.6	139.8	6.8	11.9	11.9	11.9	11.5	346.0
HEB 140B (1)	08/09/1989	21	142.9	140.8	140.0	7.1	11.6	12.0	11.7	11.6	276.0
HEB 140B (1)	08/09/1989	22	142.9	139.9	140.0	7.1	11.6	11.7	12.1	11.6	280.0
HEB 140B (1)	08/09/1989	23	143.1	138.5	139.0	6.9	11.6	11.6	12.0	11.8	258.0
HEB 140B (1)	08/09/1989	24	142.9	140.5	140.0	7.0	11.6	11.6	12.1	11.6	262.0
HEB 140B (1)	08/09/1989	25	142.9	139.8	139.6	6.9	11.6	11.7	11.9	11.7	285.0
HEB 140B (1)	08/09/1989	26	143.4	139.0	140.1	7.0	11.8	11.8	12.0	11.6	275.0
HEB 140B (1)	08/09/1989	27	142.8	139.4	140.1	6.9	11.5	11.7	12.0	11.4	289.0
HEB 140B (1)	08/09/1989	28	142.9	138.6	138.9	6.9	11.7	11.9	11.7	11.6	278.0
HEB 140B (1)	08/09/1989	29	142.9	139.2	138.9	7.0	11.6	11.7	11.9	11.9	269.0
HEB 140B (1)	08/09/1989	30	142.8	139.6	139.7	7.0	11.8	11.9	11.7	11.7	282.0
HEB 140B (2)	24/10/1989	1	141.7	139.0	137.3	7.0	11.7	11.3	11.4	11.8	308.0
HEB 140B (2)	24/10/1989	2	141.8	139.0	137.8	6.9	11.7	11.3	11.2	11.7	281.0
HEB 140B (2)	24/10/1989	3	141.7	141.5	140.5	6.9	11.8	11.4	11.4	11.8	278.0
HEB 140B (2)	24/10/1989	4	141.7	139.5	137.5	6.9	11.6	11.4	11.5	11.7	275.0
HEB 140B (2)	24/10/1989	5	141.7	140.4	140.0	7.0	11.6	11.3	11.3	11.7	279.0
HEB 140B (2)	24/10/1989	6	141.8	139.5	139.2	6.9	11.6	11.4	11.3	11.8	281.0
HEB 140B (2)	24/10/1989	7	141.7	140.0	139.0	6.9	11.8	11.3	11.4	11.7	289.0
HEB 140B (2)	24/10/1989	8	141.8	140.8	139.6	7.0	11.7	11.4	11.5	11.7	282.0
HEB 140B (2)	24/10/1989	9	141.7	139.1	137.6	6.8	11.8	11.4	11.5	11.8	288.0
HEB 140B (2)	24/10/1989	10	141.8	141.2	141.0	6.9	11.7	11.4	11.4	11.7	295.0
HEB 140B (2)	24/10/1989	11	141.8	141.5	141.3	7.0	11.9	11.6	11.9	11.9	316.0
HEB 140B (2)	24/10/1989	12	141.9	139.8	139.7	7.0	12.0	11.6	11.8	11.8	313.0
HEB 140B (2)	24/10/1989	13	142.0	139.1	137.6	6.9	11.9	11.7	11.8	11.9	306.0
HEB 140B (2)	24/10/1989	14	142.0	141.6	140.3	7.0	11.9	11.6	11.7	11.9	308.0

HEB 140B (2)	24/10/1989	15	141.9	138.8	138.0	6.9	11.9	11.6	11.8	11.8	303.0
HEB 140B (2)	24/10/1989	16	142.0	138.9	137.0	6.9	11.8	11.6	11.7	11.8	317.0
HEB 140B (2)	24/10/1989	17	141.9	140.0	139.5	7.0	11.9	11.6	11.8	11.8	309.0
HEB 140B (2)	24/10/1989	18	142.0	140.5	140.5	7.0	11.8	11.7	11.8	11.9	309.0
HEB 140B (2)	24/10/1989	19	141.9	140.0	139.5	7.0	11.8	11.6	11.7	11.9	307.0
HEB 140B (2)	24/10/1989	20	142.0	138.4	137.6	6.9	11.8	11.7	11.7	11.8	307.0
HEB 140B (2)	24/10/1989	21	142.3	140.0	139.6	6.9	12.1	11.9	11.7	12.1	302.0
HEB 140B (2)	24/10/1989	22	142.3	140.1	140.0	6.9	12.0	11.8	11.8	12.0	296.0
HEB 140B (2)	24/10/1989	23	142.4	139.0	138.0	7.0	11.9	11.8	11.8	12.0	308.0
HEB 140B (2)	24/10/1989	24	142.3	140.6	140.6	6.9	12.1	11.9	11.7	12.0	310.0
HEB 140B (2)	24/10/1989	25	142.2	139.0	137.6	7.0	12.0	11.9	11.8	12.1	306.0
HEB 140B (2)	24/10/1989	26	142.2	139.5	139.2	6.9	12.0	11.8	11.8	12.1	322.0
HEB 140B (2)	24/10/1989	27	142.2	140.0	140.0	6.9	11.9	11.8	11.8	12.1	313.0
HEB 140B (2)	24/10/1989	28	142.2	140.7	140.0	6.9	12.0	11.9	11.7	12.0	317.0
HEB 140B (2)	24/10/1989	29	142.3	140.1	140.0	6.9	12.0	11.8	11.8	12.0	305.0
HEB 140B (2)	24/10/1989	30	142.2	140.8	140.5	7.0	11.9	11.7	11.7	12.0	304.0
HEB 180B (1)	11/09/1989	1	181.3	178.4	179.2	8.5	14.3	14.0	14.0	13.3	271.0
HEB 180B (1)	11/09/1989	2	181.0	178.0	179.2	8.5	14.0	14.0	13.8	13.4	274.0
HEB 180B (1)	11/09/1989	3	181.2	178.2	179.2	8.6	14.2	14.0	14.0	13.2	277.0
HEB 180B (1)	11/09/1989	4	181.0	178.4	179.7	8.6	14.0	14.0	13.9	13.2	283.0
HEB 180B (1)	11/09/1989	5	181.2	178.4	179.0	8.5	14.0	13.8	13.6	13.4	277.0
HEB 180B (1)	11/09/1989	6	181.3	178.3	179.3	8.6	14.1	14.0	13.6	13.6	270.0
HEB 180B (1)	11/09/1989	7	182.1	178.4	179.6	8.5	14.1	13.9	13.5	13.1	268.0
HEB 180B (1)	11/09/1989	8	181.7	177.5	179.5	8.6	14.0	13.9	13.7	13.2	274.0
HEB 180B (1)	11/09/1989	9	181.1	179.3	180.2	8.6	14.0	13.9	13.8	13.4	273.0
HEB 180B (1)	11/09/1989	10	181.0	179.4	180.4	8.7	14.0	13.9	13.6	13.5	279.0
HEB 180B (1)	11/09/1989	11	180.7	180.0	181.0	8.3	13.9	13.8	13.5	12.9	278.0
HEB 180B (1)	11/09/1989	12	181.0	180.0	180.6	8.6	14.0	14.0	13.5	13.5	274.0
HEB 180B (1)	11/09/1989	13	181.0	180.0	181.0	8.5	13.9	14.0	13.5	13.3	272.0
HEB 180B (1)	11/09/1989	14	180.9	179.9	180.9	8.5	13.9	14.0	13.4	13.3	268.0
HEB 180B (1)	11/09/1989	15	181.0	180.0	181.0	8.4	13.8	14.1	13.4	13.4	270.0
HEB 180B (1)	11/09/1989	16	181.2	180.0	180.4	8.4	14.0	14.0	13.6	13.3	273.0
HEB 180B (1)	11/09/1989	17	180.8	179.8	180.6	8.2	13.9	13.8	13.6	13.1	269.0
HEB 180B (1)	11/09/1989	18	181.0	179.9	180.5	8.5	13.9	13.9	13.5	13.3	261.0
HEB 180B (1)	11/09/1989	19	181.2	180.4	180.4	8.5	14.0	13.9	13.7	13.3	284.0
HEB 180B (1)	11/09/1989	20	180.2	180.2	181.0	8.5	14.0	13.9	13.8	13.4	270.0
HEB 180B (1)	11/09/1989	21	181.0	179.7	180.4	9.0	14.1	14.0	13.8	13.5	290.0
HEB 180B (1)	11/09/1989	22	181.0	179.5	180.5	8.6	13.9	13.9	13.5	13.2	267.0
HEB 180B (1)	11/09/1989	23	180.7	179.7	180.2	8.7	14.1	14.0	13.8	13.2	282.0
HEB 180B (1)	11/09/1989	24	181.2	179.8	180.2	8.8	14.1	13.9	13.8	13.4	283.0
HEB 180B (1)	11/09/1989	25	180.8	179.8	180.0	9.0	14.1	13.8	13.7	13.3	288.0
HEB 180B (1)	11/09/1989	26	180.6	179.9	180.6	8.8	13.9	13.9	13.5	13.3	275.0
HEB 180B (1)	11/09/1989	27	180.2	179.8	180.2	8.7	14.0	13.9	13.4	13.3	267.0
HEB 180B (1)	11/09/1989	28	181.2	180.0	180.7	8.6	14.0	13.9	13.7	13.4	272.0
HEB 180B (1)	11/09/1989	29	181.3	179.8	180.6	8.7	13.9	14.0	13.8	13.4	265.0
HEB 180B (1)	11/09/1989	30	181.3	180.4	181.2	8.7	14.1	13.8	13.7	13.3	272.0
HEB 180B (2)	23/10/1989	1	181.0	179.7	179.7	8.5	13.7	13.2	13.5	14.0	273.0
HEB 180B (2)	23/10/1989	2	181.0	179.4	178.8	8.4	13.8	13.2	13.6	14.0	285.0
HEB 180B (2)	23/10/1989	3	181.1	179.7	179.8	8.4	13.6	13.3	13.7	14.0	278.0
HEB 180B (2)	23/10/1989	4	180.9	180.3	179.7	8.5	13.8	13.4	13.6	14.1	282.0
HEB 180B (2)	23/10/1989	5	180.8	180.7	179.0	8.6	13.6	13.3	13.6	14.2	276.0
HEB 180B (2)	23/10/1989	6	181.0	181.1	179.0	8.5	13.8	13.4	13.6	14.1	270.0
HEB 180B (2)	23/10/1989	7	181.0	179.7	179.7	8.4	13.6	13.2	13.7	14.0	276.0
HEB 180B (2)	23/10/1989	8	181.1	180.1	179.6	8.5	13.6	13.3	13.5	14.0	283.0
HEB 180B (2)	23/10/1989	9	181.0	179.8	178.8	8.6	13.6	13.2	13.6	14.0	268.0
HEB 180B (2)	23/10/1989	10	180.9	179.6	179.4	8.4	13.7	13.1	13.5	14.0	283.0
HEB 180B (2)	23/10/1989	11	181.1	179.1	178.6	8.6	13.5	13.3	13.7	13.9	271.0
HEB 180B (2)	23/10/1989	12	181.0	179.5	179.0	8.6	13.7	13.4	13.7	13.9	272.0
HEB 180B (2)	23/10/1989	13	180.9	179.6	179.4	8.5	13.5	13.4	13.8	13.9	270.0
HEB 180B (2)	23/10/1989	14	181.0	180.0	179.1	8.4	13.7	13.3	13.7	14.0	282.0
HEB 180B (2)	23/10/1989	15	181.2	180.4	179.7	8.5	13.6	13.4	13.8	14.1	276.0
HEB 180B (2)	23/10/1989	16	181.2	180.6	179.6	8.5	13.6	13.4	13.8	14.0	275.0
HEB 180B (2)	23/10/1989	17	181.1	179.6	179.0	8.5	13.5	13.3	13.7	13.9	287.0
HEB 180B (2)	23/10/1989	18	181.0	179.8	179.4	8.6	13.6	13.4	13.7	13.9	281.0
HEB 180B (2)	23/10/1989	19	180.9	179.7	179.5	8.4	13.5	13.2	13.8	14.0	288.0
HEB 180B (2)	23/10/1989	20	181.0	180.0	179.4	8.4	13.7	13.4	13.7	13.9	274.0
HEB 180B (2)	23/10/1989	21	180.6	179.4	179.0	8.6	13.8	13.7	13.3	14.1	294.0
HEB 180B (2)	23/10/1989	22	180.7	180.0	179.5	8.5	13.8	13.6	13.2	13.9	278.0
HEB 180B (2)	23/10/1989	23	180.7	179.7	179.4	8.6	13.8	13.7	13.1	14.0	297.0
HEB 180B (2)	23/10/1989	24	180.6	179.5	179.4	8.6	13.7	13.6	13.2	13.9	283.0
HEB 180B (2)	23/10/1989	25	180.6	179.3	179.0	8.6	13.8	13.6	13.2	13.8	278.0
HEB 180B (2)	23/10/1989	26	180.7	179.4	179.0	8.5	13.8	13.7	13.3	13.7	278.0
HEB 180B (2)	23/10/1989	27	180.6	179.3	179.1	8.5	13.7	13.6	13.3	13.7	268.0
HEB 180B (2)	23/10/1989	28	180.7	179.7	179.4	8.6	13.7	13.6	13.1	13.9	273.0
HEB 180B (2)	23/10/1989	29	180.7	179.5	179.5	8.5	13.8	13.7	13.3	13.8	275.0

HEB 180B (2)	23/10/1989	30	180.6	179.7	179.6	8.6	13.8	13.6	13.2	13.7	258.0
HEB 180B (3)	11/06/1990	1	180.7	179.4	180.8	8.3	13.7	13.6	13.8	13.6	295.0
HEB 180B (3)	11/06/1990	2	180.8	179.7	180.5	8.3	13.8	13.6	13.9	13.6	298.0
HEB 180B (3)	11/06/1990	3	180.8	179.6	180.5	8.3	13.7	13.7	13.9	13.7	310.0
HEB 180B (3)	11/06/1990	4	180.8	180.0	180.8	8.3	13.7	13.7	13.9	13.8	307.0
HEB 180B (3)	11/06/1990	5	180.7	179.6	180.7	8.4	13.9	13.6	13.9	13.7	310.0
HEB 180B (3)	11/06/1990	6	180.8	180.0	181.0	8.4	13.9	13.6	13.9	13.7	305.0
HEB 180B (3)	11/06/1990	7	180.8	179.9	181.2	8.3	14.0	13.7	13.8	13.7	298.0
HEB 180B (3)	11/06/1990	8	180.7	179.8	180.7	8.3	13.9	13.8	13.8	13.6	300.0
HEB 180B (3)	11/06/1990	9	180.7	179.7	180.9	8.3	13.9	13.8	13.8	13.5	312.0
HEB 180B (3)	11/06/1990	10	180.6	180.0	181.2	8.3	13.9	13.8	13.8	13.6	297.0
HEB 180B (3)	11/06/1990	11	180.9	182.0	183.0	8.3	13.9	13.9	13.5	13.4	290.0
HEB 180B (3)	11/06/1990	12	180.7	181.8	182.8	8.3	13.9	13.8	13.0	13.4	288.0
HEB 180B (3)	11/06/1990	13	180.8	181.9	182.8	8.3	13.9	13.8	13.5	13.4	292.0
HEB 180B (3)	11/06/1990	14	180.9	182.0	183.0	8.3	13.8	13.9	13.5	13.6	300.0
HEB 180B (3)	11/06/1990	15	180.8	181.7	181.9	8.3	13.9	14.0	13.6	13.6	307.0
HEB 180B (3)	11/06/1990	16	180.7	181.6	182.3	8.3	13.9	13.9	13.6	13.5	304.0
HEB 180B (3)	11/06/1990	17	180.8	181.6	183.0	8.2	14.0	14.0	13.5	13.4	302.0
HEB 180B (3)	11/06/1990	18	180.7	181.3	182.0	8.2	13.9	14.0	13.5	13.4	299.0
HEB 180B (3)	11/06/1990	19	180.6	181.2	182.5	8.3	13.9	13.9	13.6	13.5	300.0
HEB 180B (3)	11/06/1990	20	180.6	181.2	182.3	8.2	13.9	13.8	13.6	13.5	300.0
HEB 180B (3)	11/06/1990	21	181.1	180.4	181.7	8.1	13.8	13.8	13.8	13.5	298.0
HEB 180B (3)	11/06/1990	22	181.1	180.8	181.6	8.1	13.8	13.7	13.8	13.6	282.0
HEB 180B (3)	11/06/1990	23	181.2	180.7	181.3	8.2	13.9	13.9	13.9	13.7	288.0
HEB 180B (3)	11/06/1990	24	181.2	180.7	180.9	8.2	13.8	13.9	13.9	13.7	306.0
HEB 180B (3)	11/06/1990	25	181.0	180.6	180.9	8.2	13.9	13.9	13.8	13.6	307.0
HEB 180B (3)	11/06/1990	26	181.0	180.4	181.3	8.2	13.9	13.9	13.7	13.6	315.0
HEB 180B (3)	11/06/1990	27	181.0	180.5	181.6	8.2	13.9	13.8	13.8	13.7	300.0
HEB 180B (3)	11/06/1990	28	181.1	180.8	181.2	8.1	13.8	13.8	13.8	13.6	298.0
HEB 180B (3)	11/06/1990	29	181.0	181.0	181.3	8.2	13.9	13.9	13.8	13.7	290.0
HEB 180B (3)	11/06/1990	30	181.0	181.2	181.4	8.2	13.8	13.9	13.8	13.7	290.0

Table A2.6: Measurements of material properties and section dimensions for S235-B-ROS-689

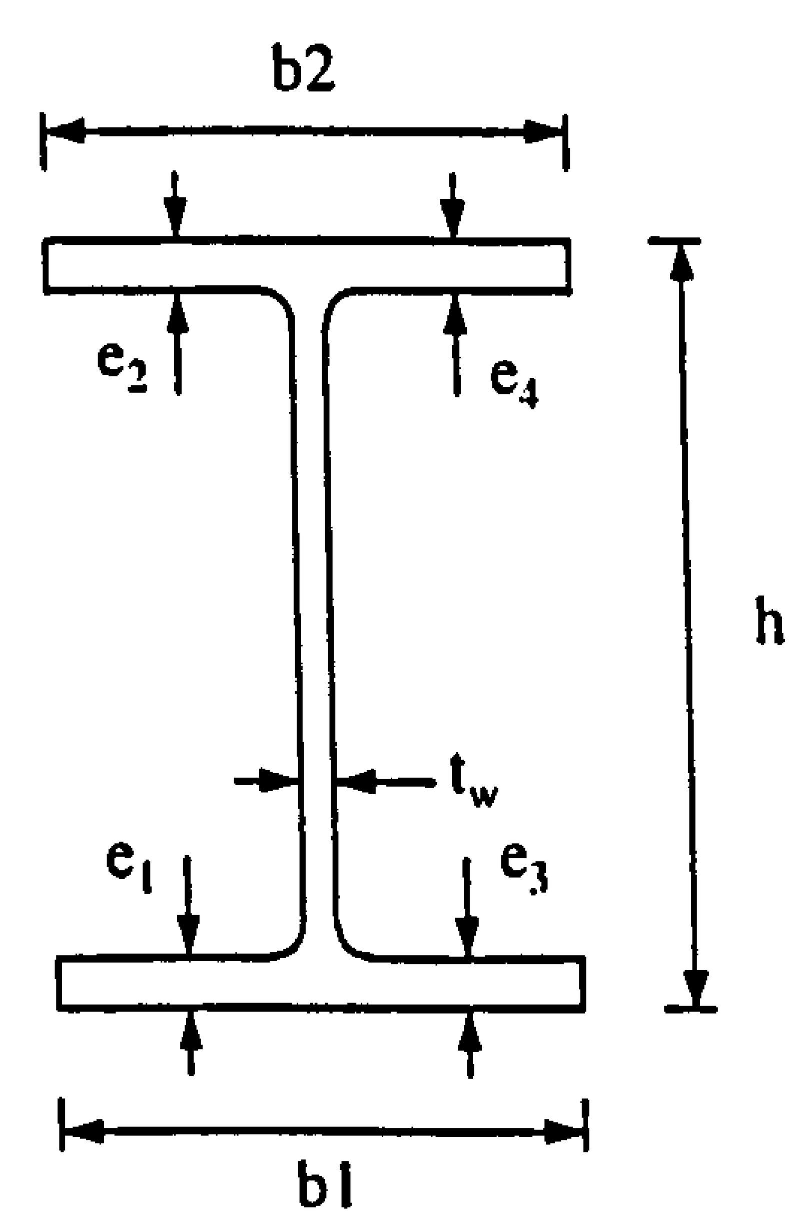

Fig. A2.11: The notation used in Table A2.6

Appendix 3

PLATE GIRDER TEST RESULTS

Test	a mm	d mm	b mm	t_w mm	t_f mm	f_{yw} N/mm^2	f_{yf} N/mm^2	la	d /t_w	Ref.
A1	172	133	25.4	1.42	6.35	258	287	172	94	Longbottom and Heyman, 1956
A4	254	121	34.9	1.42	6.35	258	287	254	85	Longbottom and Heyman, 1957
C4	254	356	41.3	1.47	6.35	258	287	508	242	Longbottom and Heyman, 1958
G6-T1	1905	1270	308	4.9	19.8	253	261	1905	259	Basler et al, 1960
G6-T2	953	1270	308	4.9	19.8	253	261	953	259	Basler et al, 1961
G6-T3	635	1270	308	4.9	19.8	253	261	635	259	Basler et al, 1962
G7-T1	1270	1270	310	4.98	19.5	253	259	1270	255	Basler et al, 1963
G7-T2	1270	1270	310	4.98	19.5	253	259	1270	255	Basler et al, 1964
G8-T1	3810	1270	305	5	19.1	263	284	3810	254	Basler et al, 1965
G9-T1	3810	1270	305	3.33	19.1	307	288	3810	381	Basler et al, 1966
G9-T2	1905	1270	305	3.33	19.1	307	288	1905	381	Basler et al, 1967
H1T2	1905	1270	459	9.98	24.8	745	703	1905	127	Cooper et al, 1964
H2T1	1270	1270	459	9.91	51.2	760	750	3810	128	Cooper et al, 1965
H2T2	635	1270	459	9.91	51.2	760	750	3810	128	Cooper et al, 1966
B	1200	1200	240	4.5	12	490	491	1200	267	Konshi, 1965
G2	1150	440	200	8	30	431	412	1150	55	Sakai et al, 1967
G5	1500	560	250	8	30	431	412	1500	70	Sakai et al, 1968
G6	687	560	250	8	30	431	412	687	70	Sakai et al, 1969
G7	1500	560	250	8	30	431	412	1500	70	Sakai et al, 1970
S3	577	477	101	3.2	10.5	317	272	577	149	Sakai et al, 1968
US2/5	788	359	96.6	3.17	12	230	422	1220	113	Kamtekar et al, 1972
US3/5	788	359	96.1	2.7	12	257	422	1620	133	Kamtekar et al, 1973
TG14	305	305	76.2	0.97	3.12	219	305	610	314	Rokey and Skaloud 1972
TG15	305	305	76.2	0.97	5	219	286	610	314	Rokey and Skaloud 1973
TG16	305	305	76.2	0.97	6.45	219	337	610	314	Rokey and Skaloud 1974
TG17	305	305	76.2	0.97	9.32	219	308	610	314	Rokey and Skaloud 1975
TG18	305	305	76.2	0.97	13	219	304	610	314	Rokey and Skaloud 1976
TG19	305	305	76.2	0.97	15.5	219	268	610	314	Rokey and Skaloud 1977
TG22	305	305	76.2	2.03	6.48	229	337	610	150	Rokey and Skaloud 1978
TG23	305	305	76.2	2.03	9.22	229	308	610	150	Rokey and Skaloud 1979
TG24	305	305	76.2	2.03	13	229	307	610	150	Rokey and Skaloud 1980
TG25	305	305	76.2	2.03	15.5	229	268	610	150	Rokey and Skaloud 1981
TS1/4	700	813	212	4.06	12	265	429	1397	200	Kamtekar et al, 1974
MSO	947	608	102	2.01	10.1	261	269	947	302	Evans et al, 1977
SD1	594	594	250	2	12	276	212	2670	297	Evans et al, 1979
SD3	594	594	250	2	12	276	212	1070	297	Evans et al, 1980
TGV1-1	1200	600	200	2.07	10	211	247	1200	290	Rockey et al, 1981
TGV1-2	600	600	200	2.07	10	211	247	1200	290	Rockey et al, 1982
TGV2-2	600	600	200	2.08	10	211	247	1200	288	Rockey et al, 1983
TGV3-2	600	600	200	2.01	10	211	247	1200	299	Rockey et al, 1984
TGV4	597	598	201	1.97	10.1	224	255	1193	304	Rockey et al, 1985
TGV5	595	598	201	1.98	9.95	232	252	1189	302	Rockey et al, 1986
TGV7-2	596	599	201	1.98	10.1	221	250	1191	303	Rockey et al, 1987
TGV10-1	595	599	200	1.91	10	219	284	1189	314	Rockey et al, 1988
TGV10-2	595	599	200	1.91	10	219	284	1191	314	Rockey et al, 1989
TGV11-2	597	599	200	1.91	10	220	211	1194	314	Rockey et al, 1990
S3/1	300	300	35	1.03	3.2	169	295	300	291	Adorisio, 1982
S4/1	345	351	39.5	1.07	3.17	169	295	345	328	Adorisio, 1983
S5/1	400	399	39	1.09	3.15	169	295	400	366	Adorisio, 1984
S2/1.5	375	249	39.5	1.05	3.16	169	295	375	237	Adorisio, 1985
S3/1.5	450	301	39	1.03	3.16	169	295	450	292	Adorisio, 1986
S4/1.5	522	352	39.1	1.1	3.27	169	295	522	320	Adorisio, 1987
LS1-PA	942	608	100	2.1	10	183	269	942	290	Evans and Tang, 1983
LS3-PA	947	608	100	2.46	10.1	201	283	947	247	Evans and Tang, 1984
MCS1-PB3	732	1000	300	4.4	15.1	170	227	1464	227	Evans, 1984

PA1	600	800	249	1	12	216	206	3000	800	Tang and Evans 1984
PA2	600	800	249	1	12	216	206	2400	800	Tang and Evans 1985
PA3	600	800	249	1	12	216	206	1800	800	Tang and Evans 1986
PB1	500	800	249	1	12	216	206	3000	800	Tang and Evans 1987
PB2	500	800	249	1	12	216	206	2500	800	Tang and Evans 1988
PC1	1000	800	250	1	10	216	262	2750	800	Tang and Evans 1989
PC2	1000	800	250	1	10	216	262	1750	800	Tang and Evans 1990
PD1	750	800	250	1	10	216	262	2750	800	Tang and Evans 1991
PD2	750	800	250	1	10	216	262	2000	800	Tang and Evans 1992
PD3	750	800	250	1	10	216	262	1250	800	Tang and Evans 1993
PC3	750	800	250	1	10	216	262	750	800	Tang and Evans 1994
PB3	732	1000	300	4.4	15.1	169.7	226.6	1464	227	Evans, 1986

Table A3.1: The material and geometric properties of the plate girders used for calibration purposes

Test	M_{exp}	$M_{f.Rd}$	M_{exp}/M_{fRD}	V_{exp}	$V_{ba.Rd}$	$V_{bb.Rd}$	$V_{exp}/V_{ba.Rd}$	$V_{exp}/V_{bb.Rd}$	Notes
A1	5	6	0.8	29	26	29	1.13	0.99	V. small models, relatively thick webs & flanges
A4	7	8	0.8	26	23	26	1.12	0.99	V. small models, relatively thick webs & flanges
C4	21	27	0.8	41	39	#N/A	1.04	#N/A	V. small models, relatively thick webs & flanges
G6-T1	983	2053	0.5	516	304	431	1.70	1.20	Large models, virtually full scale
G6-T2	631	2053	0.3	662	418	#N/A	1.58	#N/A	Large models, virtually full scale
G6-T3	500	2053	0.2	787	573	#N/A	1.37	#N/A	Large models, virtually full scale
G7-T1	791	2019	0.4	623	359	567	1.73	1.10	Large models, virtually full scale
G7-T2	819	2019	0.4	645	359	560	1.80	1.15	Large models, virtually full scale
G8-T1	1429	2133	0.7	375	291	288	1.29	1.30	Large models, virtually full scale
G9-T1	812	2163	0.4	213	139	184	1.53	1.16	Large models, virtually full scale
G9-T2	636	2163	0.3	334	154	313	2.16	1.07	Large models, virtually full scale
H1T2	6572	10361	0.6	3450	2161	2827	1.60	1.22	High strength material, double flange
H2T1	15541	23287	0.7	4079	2465	4724	1.65	0.86	High strength material, plate
H2T2	19065	23287	0.8	5004	4062	#N/A	1.23	#N/A	High strength material
B	894	1714	0.5	745	408	709	1.83	1.05	
G2	948	1162	0.8	824	866	881	0.95	0.93	La assumed equal to panel width in each case
G5	1574	1823	0.9	1049	945	1090	1.11	0.96	La assumed equal to panel width in each case
G6	808	1823	0.4	1176	1048	1148	1.12	1.02	La assumed equal to panel width in each case
G7	1574	1823	0.9	1049	945	1090	1.11	0.96	La assumed equal to panel width in each case
S3	114	141	0.8	198	154	191	1.28	1.04	
US2/5	165	181	0.9	135	113	136	1.20	0.99	Relatively thick webs and strong flanges
US3/5	146	181	0.8	90	86	107	1.04	0.84	Relatively thick webs and strong flanges
TG14	15	22	0.7	25.4	13	21	2.00	1.23	
TG15	18	34	0.5	29.4	13	24	2.32	1.23	
TG16	19	52	0.4	31.8	13	28	2.51	1.15	
TG17	24	69	0.3	39	13	32	3.08	1.20	
TG18	31	96	0.3	50.5	13	39	3.98	1.29	Failure of web
TG19	33	101	0.3	54.5	13	41	4.30	1.34	Failure of web
TG22	48	52	0.9	78.5	57	74	1.38	1.07	
TG23	49	68	0.7	81	57	79	1.43	1.03	
TG24	59	97	0.6	96	57	86	1.69	1.11	
TG25	63	101	0.6	104	57	89	1.83	1.16	
TS1/4	541	900	0.6	387	268	#N/A	1.45	#N/A	
MSO	89	171	0.5	93.5	51	78	1.82	1.20	
SD1	344	385	0.9	129	61	128	2.13	1.01	
SD3	167	385	0.4	156	61	128	2.58	1.22	
TGV1-1	100	301	0.3	83	47	66	1.78	1.27	
TGV1-2	133	301	0.4	111	57	102	1.96	1.08	
TGV2-2	138	301	0.5	115	57	103	2.01	1.12	
TGV3-2	136	301	0.4	113	53	99	2.11	1.14	
TGV4	122	315	0.4	102	53	#N/A	1.93	#N/A	
TGV5	125	306	0.4	105	55	#N/A	1.93	#N/A	
TGV7-2	126	309	0.4	106	53	#N/A	1.99	#N/A	
TGV10-1	121	346	0.4	102	49	#N/A	2.07	#N/A	
TGV10-2	126	346	0.4	106	49	#N/A	2.15	#N/A	
TGV11-2	122	257	0.5	102	49	#N/A	2.07	#N/A	
S3/1	6	10	0.6	19	13	17	1.51	1.11	Very small models
S4/1	7	13	0.6	21	14	#N/A	1.53	#N/A	Very small models
S5/1	9	15	0.6	23	14	20	1.64	1.12	Very small models
S2/1.5	6	9	0.6	15.5	11	14	1.36	1.15	Very small models
S3/1.5	7	11	0.7	16	11	13	1.46	1.19	Very small models
S4/1.5	7	13	0.5	13	13	16	1.04	0.84	Very small models
LS1-PA	71	166	0.4	75.5	47	66	1.60	1.15	

LS3-PA	98	177	0.6	103	68	87	1.52	1.18	
MCS1-PB3	568	1044	0.5	388	281	#N/A	1.38	#N/A	Virtually full scale
PA1	243	500	0.5	81	16	#N/A	5.04	#N/A	V. thin web
PA2	200	500	0.4	83.5	16	#N/A	5.19	#N/A	V. thin web
PA3	153	500	0.3	85	16	#N/A	5.28	#N/A	V. thin web
PB1	270	500	0.5	90	18	#N/A	4.89	#N/A	V. thin web
PB2	228	500	0.5	91	18	#N/A	4.94	#N/A	V. thin web
PC1	147	531	0.3	53.5	12	55	4.35	0.97	V. thin web
PC2	94	531	0.2	53.5	12	55	4.35	0.97	V. thin web
PD1	179	531	0.3	65	14	#N/A	4.68	#N/A	V. thin web
PD2	130	531	0.2	65	14	#N/A	4.68	#N/A	V. thin web
PD3	94	531	0.2	75	14	#N/A	5.40	#N/A	V. thin web
PC3	59	531	0.1	78.7	14	#N/A	5.66	#N/A	V. thin web
PB3	568	1042	0.5	388	281	#N/A	1.38	#N/A	

Table A3.2: The experimental and predicted load capacities of the plate girders used for calibration purposes