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INTERNATIONAL CENTRE FOR MECHANICAL SCIENCES

LIGHT GAUGE METAL STRUCTURES RECENT ADVANCES

EDITED BY

JACQUES RONDAL
DAN DUBINA

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PREFACE

In recent years, it has been recognized that both cold-formed steel and aluminium alloy sections can be used effectively as primary framing components. In what concerns cold-formed steel sections, after their primarily application as purlins or side rails, the second major one in construction is in the building envelope. Options for steel cladding panels range from inexpensive profiled sheeting for industrial applications, through architectural flat panels used to achieve a prestigious look of the building. Light steel systems are widely used to support curtain wall panels. Cold-formed steel in the form of profiled decking has gained widespread acceptance over the past fifteen years as a basic component, along with concrete, in composite slabs. These are now prevalent in the multi-storey steel framed building market. Cold-formed steel members are efficient in terms of both their stiffness and strength. In addition, because the steel may be even less than 1 mm thick, the members are light weight. The already impressive load carrying capabilities of cold formed steel members will be enhanced by current work to develop composite systems, both for wall and floor structures.

Recent studies have shown that because the coating loss for galvanised steel members is sufficiently slow, and indeed slows down to effectively zero, a design life in excess of 60 years can be guaranteed. The production of economic coated steel coils has also given interesting solutions to architectural demands increasing the range of use of cold-formed sections. Higher yield stress steels are also becoming more common for the fabrication of cold-formed sections.

However, the use of high strength steels and thinner sections leads inevitably to complex design problems, particularly in the field of structural stability and joints. In recent years, stainless steel profiles and aluminium alloy profiles have also been used increasingly as structural members.

The aims of the "Advanced Professional Training on Light Gauge Metal Structures – Recent Advances" organized at the International Centre for Mechanical Sciences in Udine, June 3-7, 2002, were to review recent research and technical advances, including the progress in design codes, related to the engineering applications of light gauge metal sections made in carbon, high strength and stainless steel, as well as aluminium alloys.

The lectures include also a review of the new technologies for connections of light gauge metal members. Main advanced applications, for residential, non residential and industrial buildings and pallet rack systems are also covered.

This monograph is a revised version of the lecture notes. However, the lectures given by F.M. Mazzolani on the aluminium structural design have not been included in this monograph because a full CISM monograph, edited by F.M. Mazzolani (CISM Courses and Lectures n° 443, 2003) has been entirely dedicated to the use of Aluminium-Alloys in structures. The other lectures have been prepared by :

- *J.M. Davies, The University of Manchester, England;*
- *D. Dubina, Technical University of Timisoara, Romania;*
- *R. Laboube, University of Missouri-Rolla, USA;*
- *K.J.R. Rasmussen, University of Sydney, Australia;*
- *J. Rondal, University of Liege, Belgium.*

The editors wish to thank warmly these colleagues for the excellence of the work performed during the preparation of this advanced professional training, which is well reflected in this monograph.

Special thanks are also due to the CISM Rector, Prof. M.G. Velarde, the CISM Secretary General, Prof. B. Schrefler, the Executive Editor of the Series, Prof. C. Tasso, and to all the CISM staff in Udine.

Jacques Rondal
Dan Dubina

CONTENTS

Chapter 1: Introduction to Light Gauge Metal Structures

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1.1 Historical considerations

The use of cold-formed steel members in building construction began in the mid of the eighteenth century in United States and United Kingdom. However such steel members were not widely used as structural members until around 1946 and the publication of the first edition of the "Specification for the Design of Light Gage Steel Structural Members" by the American Iron and Steel Institute (AISI). Since that period, thousands of researches in the field have led to a wide use of cold-formed metal elements in all types of buildings.

If, in the past, cold-formed products were mainly used as secondary components in steel or concrete structures, there is now a wide marked for cold-formed structural elements.

These structural elements are used as single members like columns, beams or purlins but also as components of industrialized building systems. In these systems, the cold-formed elements play frequently a multifunctional role leading to economy and simplicity of the structure. Sometimes, they make the traditional steel skeleton unnecessary or, at least, they contribute largely to its load bearing capacity.

For example, the combination of cold-formed members and sheeting can be such that instability phenomena are prevented, leading to a space covering function and an improvement of the resistance.

1.2 Peculiarities of cold-formed steel members

In general, cold-formed steel structural members provide the following advantages in building construction (Yu, 1985):

- as compared with thicker hot-rolled shapes, cold-formed light members can be manufactured for relatively light loads and/or short spans;
- unusual sectional configurations can be produced economically by cold-forming operations and, consequently, favourable strength-to-weight ratios can be obtained;
- nestable sections can be produced, allowing for compact packaging and shipping;
- load-carrying panels and decks can provide useful surfaces for floor, roof, and wall construction, and in other cases, they can also provide enclosed cells for electrical and HVAC conduits;
- load-carrying panels and decks not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist force in their own planes if they are adequately interconnected to each other and to supporting members.

Compared with other materials such as timber and concrete, the following qualities can be realized for cold-formed steel structural members:

- lightness;
- high strength and stiffness;
- ease of prefabrication and mass production;
- fast and easy erection and installation;
- substantial elimination of delays due to weather;
- more accurate detailing;
- nonshrinking and noncreeping at ambient temperature;
- uniform quality;
- economy in transportation and handling.

The combination of the above-mentioned advantages can result in important cost saving during construction.

However, because cold-formed members are usually thin-walled, special care must be given to the design. Compared to classical hot-rolled sections, they are characterized by some peculiarities, e.g.:

- large width to thickness ratios;
- singly symmetrical or unsymmetrical shapes;
- unstiffened or partially unstiffened parts of sections;

which can lead to difficult buckling problems :

- combined torsional and flexural buckling;
- local plate buckling;
- distorsional buckling;
- interaction between local and global buckling, ...

Also connections must be designed with care because the thickness of the members can lead to local failures.

For these reasons, dedicated specifications have been published in United States firstly, and after in Europe, Australia and in other countries to cover these important questions.

1.3 Recent Advances

In recent years, it has been recognized that both cold-formed steel and aluminium alloy sections can be used effectively as primary framing components. In what concerns cold-formed steel sections, after their primarily application as purlins or side rails, the second major one in construction is in the building envelope. Options for steel cladding panels range from inexpensive profiled sheeting for industrial applications, through architectural flat panels used to achieve a prestigious look of the building. Light steel systems are widely used to support curtain wall panels. Cold-formed steel in the form of profiled decking has gained widespread acceptance over the past fifteen years as a basic component, along with concrete, in composite slabs. These are now prevalent in the multi-storey steel framed building market. Cold-formed steel members are efficient in terms of both their stiffness and strength. In addition, because the steel may be even less than 1 mm thick, the members are light weight. The already impressive load carrying capabilities of cold formed steel members will be enhanced by current work to develop composite systems, both for wall and floor structures.

Recent studies have shown that because the coating loss for galvanised steel members is sufficiently slow, and indeed slows down to effectively zero, a design life in excess of 60 years can be guaranteed. The production of economic coated steel coils has also given interesting solutions to architectural demands increasing the range of use of cold-formed sections.

As younger products, cold-formed steel sections are more open to development than classical hot-rolled profiles (Davies, 2000).

An important trend is the use of higher quality steels with an increased yield stress. The steel used actually for mass-produced products such as purlins, sheeting and decking has a yield stress in the range 280 to 600 N/mm^2. This trend is likely to continue in the future. However, the applications of high strength steels are limited by stiffness considerations in many situations.

The use of high strength steel leads inevitably to a reduction of the thickness of the profiles and to complex local stability problems. To improve the load stability of the sections, complex shapes have been developed with more folds and stiffeners.

Important progresses have also been made in the rolling and forming technology (Peköz, 1999). Modern rolling lines are computer controlled from the design office so that not only highly accurate complex shapes of precise lengths be produced to order but also holes, perforations, web opening for services can be punched in precise positions during the rolling process.

In recent years, stainless steel profiles and aluminium alloy profiles have also been used increasingly as structural members.

This Advanced Professional Training aims to review recent research and technical advances, including the progress in design codes, related to the engineering applications of light gauge metal sections made in carbon, high strength and stainless steel, as well as aluminium alloys.

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Chapter 2: Peculiar Problems in Cold-formed Steel Design Part 1

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2.1 Elements

2.2.1. Cold-formed steel sections: linear profiles, cladding and sheeting panels

In recent years, cold-formed steel sections started to be used effectively as primary framing components. In what concerns cold-formed steel sections, after their primarily applications as purlins or side rails, the second major one in construction is in the building envelope. Options for steel cladding panels range from inexpensive profiled sheeting for industrial applications, through architectural flat panels used to achieve a prestigious look of the building. Light steel systems are widely used to support curtain wall panels. Cold-formed steel in the form of profiled decking has gained widespread acceptance over the past fifteen years as a basic component, along with concrete, in composite slabs. These are now prevalent in the multi-storey steel framed building market. Cold-formed steel members are efficient in terms of both their stiffness and strength. In addition, because the steel may be even less than 1 mm thick, the members are light weight. The already impressive load carrying capabilities of cold-formed steel members will be enhanced by current work to develop composite systems, both for wall and floor structures.

The continuously increasing of cold-formed steel structures throughout the world is sustained by production of more economic steel coils particularly in coated form with zinc or aluminium / zinc coatings. These coils are subsequently formed into thin-walled sections by the cold-forming process. They are commonly called "Light gauge sections" since their thickness has been normally less than 3 mm. However, more recent developments have allowed sections up to 25 mm to be cold-formed, and open sections up to approximately 8mm thick are becoming common in building construction. The steel used for these sections may have a yield stress ranging from 250 MPa to 550 MPa (Hancock, 1997). The higher yield stress steels are also becoming more common as steel manufacturers produce high strength steel more efficiently.

Improving technology of manufacture and corrosion protection applied to cold-formed structural steelwork, provide competitiveness of resulting products and extend the area of new applications. Recent studies have shown that the coating loss for galvanized steel members is sufficiently slow, and indeed slows down to effectively zero, than a design life in excess of 60 years can be guaranteed (Owens, 2000).

Thin walled sections and high strength steels leads to design problems for structural engineers which may not normally be encountered in routine structural steel design. Structural instability of the sections is more likely to occur as a result of the reduced buckling loads (and stresses), and the use of higher strength steel which may make the buckling stress and yield stress of the thin-walled sections approximately equal (Hancock, 1997). Further, the shapes which can be cold-formed are often considerably more complex than hot-rolled steel shapes such as I-sections and unlipped channel sections. The cold-formed sections commonly have mono-symmetric or point symmetric shapes, and normally have stiffening lips on flanges and intermediate stiffeners in wide flanges and webs. Both simple and complex shapes can be formed for structural and non-structural applications as shown in Figure 1. Special design standards have been developed for these sections.

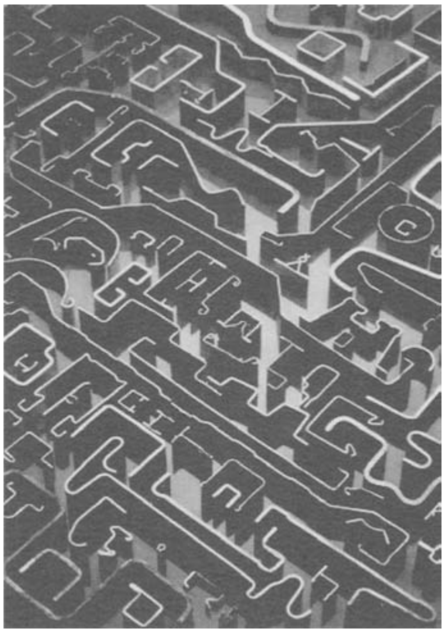

Figure 1. Collection of different cold-formed section shapes (Trebilcock, 1994).

Cold-formed members and profiles sheets are steel products made from coated or uncoated hot rolled or cold-rolled flat strip of coils. Within the permitted range of tolerances, they have constant or variable cross section.

Cold-formed structural steel members can be classified into two major types:

1. Long profile – individual structural framing
2. Cladding panels and sheeting decks

Individual structural members (bar members) obtained from so called "long products" include:

- single open sections, sown in Figure 2a;
- open built-up sections, Figure 2b;
- closed built-up sections, Figure 2c.

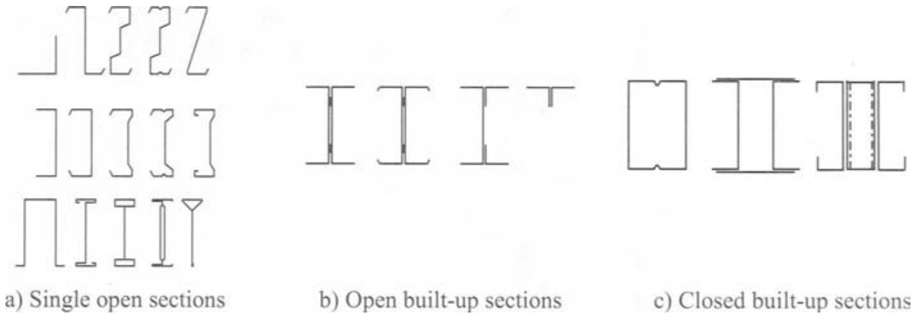

Figure 2. Typical forms of sections for cold-formed structural members.

Usual, the depth of cold-formed sections for bar members ranges from 50-70 mm to 350-400 mm, with thickness from 1 to 6 mm about.

Panel and decks are made from profiled sheets and linear trays (cassettes) shown in Figure 3. The depth of panel usually ranges from 20 to 200 mm, while thickness is from 0.4 to 1.2 (1.5) mm. They can be produced as flat or smooth curved shapes and can be used for roofing, wall cladding systems and load bearing deck panels.

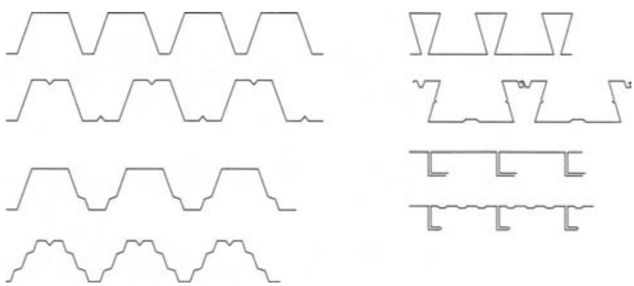

Figure 3. Profiled sheets and linear trays.

Smooth curved shape sheeting can be also produced by roll forming and bending special applications, like self-supporting arch and roof structures and also to provide a specific architectural appearance of facades.

In general, cold-formed steel sections provide the following advantages in building constructions (Yu, 2000):

1. As compared with thicker hot-rolled shapes, cold-formed light members can be manufactured for relatively light loads and/or short spans;
2. Unusual sectional configurations can be produced economically by cold-forming operations (Figure 1), and consequently favourable strength-to-weight ratios can be obtained;
3. Nestable sections can be produced, allowing for compact packaging and shipping;

4. Load carrying panel and decks can provide useful surface for floor, roof, and wall construction, and in other cases they can also provide enclosed cells for electrical and other conduits;
5. Load-carrying panels and decks not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist force in their own panels if they are adequately interconnected to each other and to supporting members.

Compared with other materials such timber and concrete, the following qualities can be realized for cold-formed steel structural members:

1. Lightness;
2. High strength and stiffness;
3. Ability to provide long spans (up to 10 m, Rhodes, 1991);
4. Ease of prefabrication and mass production;
5. Fast and easy erection and installation;
6. Substantial elimination of delays due to weather;
7. More accurate detailing;
8. Non-shrinking and non-creeping at ambient temperatures;
9. Formwork unneeded;
10. Termite-proof and root-proof;
11. Uniform quality;
12. Economy in transportation and handling;
13. Non-combustibility;
14. Recyclable material.

Combination of above mentioned advantages can result in cost and erection time saving in construction.

2.1.2 Comparison with hot-rolled steel sections: Peculiar problems in Cold-formed steel design

The use of thin walled sections and cold-forming effects can result in special design problems, not normally encountered when tick hot-rolled sections are used. A brief summary of some special problems in cold-formed steel design is reviewed on the following.

Buckling strength of cold-formed members. Steel sections may be subjected to one of four generic types of buckling, namely local, global, distortional and shear (Davies, 2000). Local buckling is particularly prevalent in cold-formed sections and is characterised by the relatively short wavelength buckling of individual plate element. The term "global buckling" embraces Euler (flexural) and lateral-torsional buckling of columns and lateral buckling of beams. It is sometimes termed "rigid-body" buckling because any given cross-section moves as a rigid body without any distortion of the cross-section. Distortional buckling, as the term suggested, is buckling which takes place as a consequence of distortion of the cross section. In cold-formed sections, it is characterised by relative movement of fold-lines. The wavelength of distortional buckling is generally intermediate between that of local buckling and global buckling.

It is a consequence of the increasing complexity of section shapes that local buckling calculation is becoming more complicated and the torsional buckling takes on increasing importance.

Local and distortional buckling can be considered as "sectional" modes, and they can interact with each other as well as with global buckling (Dubina, 1996). Figure 4 shows single and interactive (coupled) buckling modes for a lipped channel section in compression. The results have been obtained using an elastic eigenbuckling FEM analysis. For given geometrical properties of member cross-section, the different buckling modes depend of buckling length. For shorter members, sectional buckling modes (L and D) are dominant, while for slender ones, the bar buckling modes (F and FT) prevail. Intermediate lengths are, generally, characterised by interactive sectional-bar buckling modes.

Sectional modes and their interaction with bar buckling ones do not appear in case of hot rolled sections.

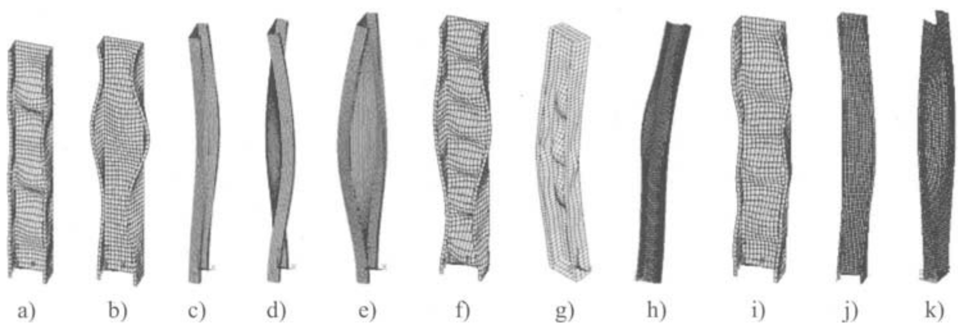

Figure 4. Buckling modes for a lipped channel in compression.
Single modes: (a) local (L); (b) distortional (D); (c) flexural (F);(d) torsional (T); (e) flexural-torsional (FT).*Coupled (interactive) modes*: (f) L + D; (g) F + L; (h) F + D; (i) FT + L; (j) FT + D; (k) F + FT

The effect of interaction between sectional and global buckling modes consists in increasing sensitivity to imperfections, leading to the erosion of theoretical buckling strength. In fact, due to the inherent presence of imperfection, buckling mode interaction always occurs in case of thin-walled members.

Figure 5 shows the difference in behaviour of a tick-walled slender bar in compression (Figure 5a), and a thin-walled one (Figure 5b). Both cases of ideal perfect bar and imperfect one are presented.

Looking to the behaviour of actual tick-walled bar it can be seen that it begins to depart from the elastic curve at point B when the first fibre reaches the yield stress and it reaches its maximum (ultimate) load capacity, N_u, at point C; after which it declines and the curve approaches the theoretical rigid-plastic curve asymptotically. The elastic theory is able to define the deflections and stresses up to the point of first yield and to define the load at which first yield occurs. The position of rigid-plastic curve determinates the absolute limit of load carrying capacity, for above it is a region in which the structures cannot carry a load and remain in a state of equilibrium.

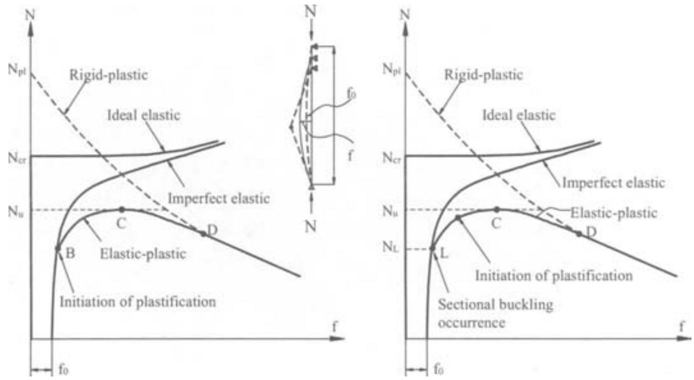

Figure 5. Behaviour of (a) slender tick-walled (hot-rolled section) and (b) thin-walled (cold-formed section) compression bar.

In case of thin-walled bar the sectional buckling, e.g. local or distortional buckling, occurs prior to the initiation of plastification. Sectional buckling is characterised by the stable post-critical path and bar does not fail when it occurs, but significantly lose from its stiffness. The yielding starts at the corners of cross-section, a few time before the failure of the bar, when sectional buckling changes into local plastic mechanism quasi-simultaneously with global buckling occurrence (Dubina, 2000).

In Figure 6 are shown the comparison between the buckling curves of a lipped channel member in compression, calculated according to ENV 1993-1-3, considering the full effective cross-section (e.g. no local buckling effect, which is generally the case of hot-rolled sections), and the reduced (effective) cross-section (e.g. when the local buckling occurs and interacts with global buckling).

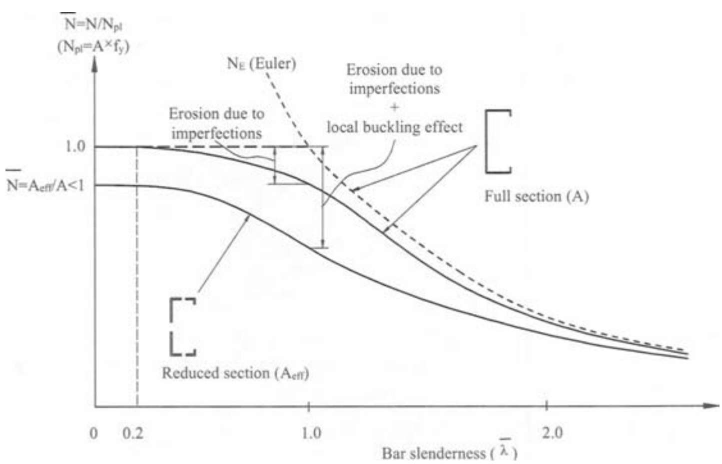

Figure 6. Effect of local buckling on the member capacity.

Web Crippling. Web crippling at points of concentrated load and supports can be a critical problem in cold-formed steel structural members and sheeting for several reasons. These are:

1. In cold-formed steel design, it is often not practical to provide load bearing and end bearing stiffeners. This is always the case in continuous sheeting and decking spanning several support points.
2. The depth-to-thickness ratios of the webs of cold-formed members are usually larger than hot-rolled structural members.
3. In many cases, the webs are inclined rather than vertical.
4. The intermediate element between the flange, on which the load is applied, and the web of a cold-formed member usually consists of a bend of finite radius. Hence the load is applied eccentrically from the web.

Web crippling is really a very peculiar feature of the behaviour of thin-walled cold-formed sections and special design provisions are included in design codes in order to manage this phenomenon, which doesn't occur in case of hot-rolled section.

Connections. Because of the wall thinness of cold-formed sections, conventional method for connection used in steel construction, such as bolting and arc-welding are, of course, available, but are generally less appropriate and emphasis is on special techniques, more suited to thin materials (Davies, 2000). Long-standing methods for connecting two elements thin material are blind rivets and self-drilling, self-tapping screws. Fired pins are often used to connect thin materials to a ticker-supporting member. More recently, press-joining or clinching technology (Predeschi, 1997), which is very productive, requires no additional components and causes no damage to the galvanizing or other metallic coating. This technology has been taken from the automotive industry, but actually it is successfully used in building construction. "Rosette" system is another innovative connecting technology (Makelainen and Kesti, 1999), proper to cold-formed steel structures.

Therefore, connection technology of cold-formed steel structures is representing one of their particular advantages, both in manufacturing and erection process.

Design assisted by testing. Cold-forming technology makes available production of unusual sectional configurations (see Figure 1). However, from the point of view of structural design, the analysis and design of such unusual members may be very complex. Structural systems formed by different cold-formed sections connected one to each other (like purlins and sheeting, for instance) can also lead to complex design situations, not entirely covered by design code procedures. Of course, numerical FEM analysis is always available, but even for some simply practical situations, modelling could be very complicate. For complex design problems, modern design codes permit to use testing procedures to evaluate structural performances. Testing can be used either to replace design by calculation or combined with calculation. Also one using numerical simulations, experimentally calibrated based numerical models are recommended. Only officially accredited laboratories, by competent authorities, are allowed to perform such tests and to delivery relevant certificates.

2.1.3 Ductility and plastic design

Usually, hot-rolled sections are of class 1 and 2, while cold-formed ones cold-formed sections are of class 4 or class 3, at the most. For this reason and also due to the effect of cold-forming by stress hardening, the cold-formed steel sections possess a low ductility and are not generally allowed for plastic design. Therefore, the previous discussion related to Figure 9b revealed the low inelastic capacity reserve for these sections, after the yielding was initiated. However, for members in bending, design codes allow to use the inelastic capacity reserve of their cross-section part which is working in tension. Moreover, because of their reduced ductility, cold-formed sections cannot dissipate energy in seismic resistant structures. Cold-formed sections can be used in seismic resistant structures because there are structural benefits coming from their reduced weight, but only elastic design is allowed and no reduction of shear seismic force is possible. Hence, in seismic design, a reduction factor $q=1$ has to be taken, as stated in EUROCODE 8 (ENV 1998, 1994).

2.1.4 Corrosion

The main factors governing the corrosion resistance of cold-formed steel sections is the type and thickness of the protective treatment applied to the steel and not the base metal thickness. Cold-formed steel has the advantage that the protective coatings can be applied to the strip during manufacture and before roll forming. Consequently, galvanized strip can be passed through the rolls and requires no further treatment.

Usually, steel profiles are hot dip galvanized with 275 gram of zinc per square meter (Zn 275), corresponding to a zinc thickness of 20 μm on each side. Hot dip galvanized is sufficient to protect the steel profiles against corrosion during the entire life of a building, if it was constructed in the correct manner. The most severe effects of corrosion on the steel occur during transport and storage outdoors. When making holes in hot dip galvanized steel framing members, normally no treatment is needed afterwards since the zinc layer a healing effect, i.e. transfers to unprotected surfaces.

Hot dip galvanizing is sufficient to protect the steel profiles against corrosion during the life of a building. The service life of hot dip galvanized steel studs was studied by British Steel and others (Burling P.M, 1990). The loss in zinc weight will be around 0.1g per m^2 per year indoors. A similar study was also carried out for steel floors above crawl spaces with plastic sheeting on the ground. Results showed that a zinc weight of 275g/ m^2 is sufficient to provide a durability of around 100 years.

Special attentions should be given to the cases in which different materials are intended to act compositely, if these materials are such that electrochemical phenomena might produce conditions leading to corrosion. Such phenomena are possible to appear when sheeting and fastener materials are different.

2.1.5 Fire resistance

Due to the small values of section factor (e.g. the ratio of the heated parameter to the cross-section area of the member) the fire resistance of unprotected cold-formed steel sections is reduced. In ENV 1993-1-2 (1995) simple calculation models are given by which the critical steel temperature and load bearing capacity easily can be found for different structures as beam and columns. However, these simple models are restricted to steel sections where the first order theory in a global plastic analysis may be used (ECCS TC3, 2001). Class 4 cross-sections do not fulfil the requirement and the subject has not been investigated much. The alternative is to use more general and more complicated calculation models and have to be verified by relevant test results. Very few attempts have been made to develop and verify calculation models for load bearing class 4 cross-sections in fire. In Klippstein (1979) and Gerlich (1995) calculation models are given for class 4 cross-sections, studs, in walls which have been verified by a number of fire tests. In Building Design: Fire protection (1993) the subject is described, principles and prediction methods are presented. In Ranby (1997), results show that a calculation according to ENV 1993-1-2 (1995) with reduced yield strength and plastic modulus is accurate enough also at elevated temperatures provided that the yield strength is taken as the 0.2 percent proof stress.

Table 1 shows comparatively the fire strength expressed in minutes for three different sections of which the area is approximately of comparable magnitude. One of these sections is built-up by back-to-back cold-formed lipped channels. The utilization factor is about 0.5. Eurocode 3-1.2 provisions and advanced FEM code SAFIR (Franssen, 2000) were used for calculation. The buckling was assumed to be prevented.

Sprayed cementations or gypsum based coatings can be employed for beams concealed behind a suspended ceiling. In load bearing applications, fire resistance periods of 30 minutes can usually be achieved by one layer of "special" fire resistant plasterboard, and 60 minutes by two layers of this plasterboard, which possesses low shrinkage and high integrity properties in fire. Planar protection to floors and walls provides adequate fire resistance to the enclosed sections, which retain a significant proportion of their strength, even at temperatures of 500°C.

In Light Gauge Steel Framing, the board covering of walls and floors can protect the steel against fire for up to 120 minutes, depending on the board material and the number of boards. The choice of insulation material, mineral wool or rock wool is also crucial to fire strength.

Table 1. Fire strength.

Member		Section	Am/V [m^{-1}]	FEM (SAFIR)	EC3-1.2	Difference
Unprotected	Columns	IPE220	254.7	11	11.5	-5%
		HEA160	234.7	11.5	10.8	6%
		][C300/3	480.4	9	9.3	-3%
	Beams	IPE220	254.7	11.9	11	8%
		HEA160	234.7	11.1	10.2	8%
		][C300/3	480.4	9.6	7.6	21%
Protected (Chartek 4 spray)	Columns	IPE220	254.7	34	29.8	12%
		HEA160	234.7	35.5	27.9	21%
		][C300/3	480.4	28	23.6	16%
	Beams	IPE220	254.7	35.9	28.5	21%
		HEA160	234.7	34.9	26.3	25%
		][C300/3	480.4	29.4	22.3	24%

Box protection to individual CFS sections used as beams and columns is provided in the same way as with hot rolled sections.

Non-load bearing members require less fire protection, as they only have to satisfy the "insulation" criterion in fire conditions. Ordinary plasterboard may be used in such cases.

However, using active fire protection and natural fire based design procedures seems to be the best solution in case of cold-formed steel sections.

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Chapter 2: Peculiar Problems in Cold-formed Steel Design Part 2

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2.2 Mechanical Properties and Imperfections

2.2.1 Introduction

Buckling and post-buckling of cold-formed members are rather difficult to predict due to material and geometrical nonlinearities. However, numerical techniques have reached a level of maturity such that many are now successfully undertaking ultimate strength analysis of cold-formed steel members (Schafer and Peköz, 1998).

The first condition to success of numerical simulations is not the theoretical formulation nor the solution technique but the knowledge of the initial state of a cold-formed steel member. Precise characterization of geometrical imperfections and residual stresses is largely unavailable. Also the distribution of the yield stress along the perimeter of the section is not uniform due to the cold-rolling process. A good knowledge of these fundamental quantities is necessary for reliable completion of advanced analysis or parametric studies of cold-formed steel members.

The aim of this chapter is to give information on the existing data related to these imperfections.

2.2.2 Mechanical properties of cold-formed steel members

Thin-walled steel sections are fabricated by means of cold-rolling of coils or press-braking of plates made in carbon steel. However, for these members which are now very frequently used in modern steel construction, the initial σ-ε relation of the steel is considerably changed by the cold-straining due to the manufacturing processes. Figure 7 shows the modification of the σ-ε diagram when a carbon steel specimen is first strained beyond the yield plateau and then unloaded.

If strain aging is now very rare, or at least limited, with modern steels, the cold-straining however modified the apparent σ-ε diagram which is pertinent for cold-formed steel members.

However, the strain-hardening can vary considerably along the cross-section due to the forming process. The apparent increase of the yield stress is more pronounced in the corners than in the flat faces as shown in Table 2.

Table 2. Increase of the yield stress

Forming process		Cold-rolling	Press-braking
Yield-strength (f_y)	Corners	High	High
	Flat faces	Moderate	—
Ultimate stress (f_u)	Corners	High	High
	Flat faces	Moderate	—

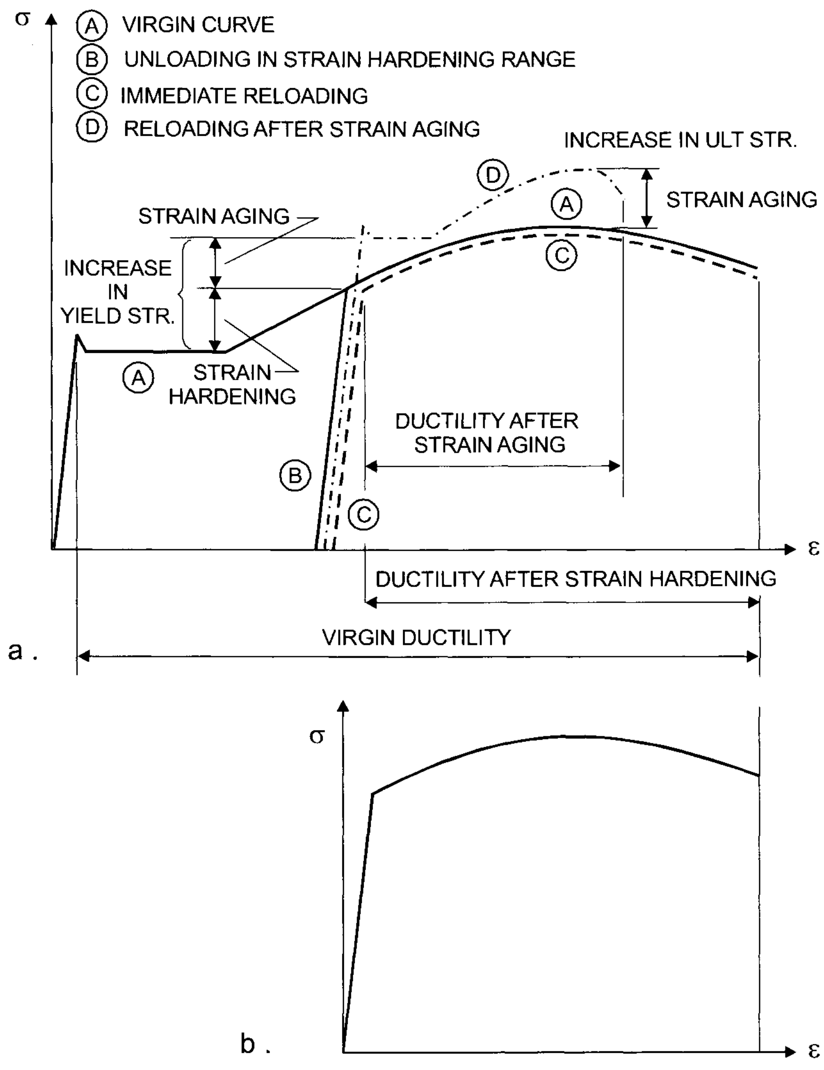

Figure 7. Effects of cold straining and strain aging on σ-ε characteristics of carbon steel (a – global σ-ε diagram; b – apparent σ-ε diagram for a cold-formed member)

Many authors have investigated the influence of cold work on the distribution of the yield strength along the cross-section.

Karren and Winter have proposed the following equation for the corner yield strength (Karren, 1967; Karren and Winter, 1967):

$$f_{yc} = \frac{kg}{(r/t)^h} \tag{1}$$

with

$$g = 0.945 - 1.315\,q \tag{2}$$

$$h = 0.803\,q \tag{3}$$

where t is the thickness of the sheet, r is the inside bend radius and k and q are the parameters of the hardening law which are given by :

$$k = 2.80 f_u - 1.55 f_{yb} \tag{4}$$

$$q = 0.225\frac{f_u}{f_{yb}} - 0.120 \tag{5}$$

where f_u is the virgin ultimate strength and f_{yb} the virgin yield strength of the sheet.

With regard to the full-section properties, the average tensile yield strength may be approximated by using a weighted average as follows (Karren and Winter, 1967):

$$f_{ya} = A_c f_{yc} + (1 - A_c) f_{yb} \tag{6}$$

where A_c is the ratio of corner area to total cross-sectional area.

Eurocode 3, Part 1.3 gives the following formula to evaluate the average design strength f_{ya} of the full section. This formula is, in fact, a modification of formula (6) where a zone closed to the corner is considered as fully plastified:

$$f_{ya} = f_{yb} + (Cnt^2 / A_g).(f_u - f_{yb}) \tag{7}$$

where A_g is the gross cross sectional area and n is the number of 90° bends in the section, with an internal radius $r < 5t$ (Eurocode 3, 1996). In this relation, $C = 7$ for cold-rolling and $C = 5$ for other methods of forming.

The increase of the yield strength is however limited to a certain extent. Two formulae have been proposed for this limit:

$$f_{ya} \leq 0.5(f_{yb} + f_u) \tag{8}$$

or

$$f_{ya} \leq 1.25 f_{yb} \tag{9}$$

2.2.3 Geometrical imperfections

Geometrical imperfections refer to deviations from a so-called "perfect" geometry. Member imperfections included bowing, warping and twisting as well as local deviations characterized by dents and regular undulations in the plates.

Many researchers have measured geometrical imperfections of cold-formed steel structures.

Schafer and Peköz have analysed the frequency and amplitude of two types of local/sectional geometrical imperfections shown in Figure 8, where d_1 can be regarded as representative of local buckling, while d_2 characterises distorsional buckling (Schafer and Pekoz, 1998).

For simple analysis, these authors proposed to use the following values of the imperfections, for width to thickness ratio (w/t) less than 200 for type 1 imperfection and w/t less than 100 for the type 2, with a thickness less than 3 mm:

- type 1 :

$$d_1 \approx 0.006\, w \tag{10}$$

or

$$d_1 \approx 6t.e^{-2t} \tag{11}$$

The last formula is based on an exponential curve fit to the thickness of the profile.

- type 2 :

$$d_2 \approx t \tag{12}$$

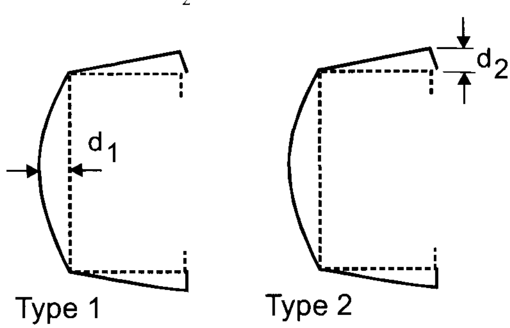

Figure 8. Definition of geometric imperfections