

# **Eurocode 8: Design provisions for earthquake resistance of structures —**

## **Part 1.3: General rules —**

### **Specific rules for various materials and elements**

ICS 91.120.20

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# Committees responsible for this Draft for Development

The preparation of this Draft for Development was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/8, Structures in seismic regions, upon which the following bodies were represented:

- Association of Consulting Engineers
- British Geological Survey
- Department of the Environment (Building Research Establishment)
- Department of the Environment (Property and Building Directorate)
- Federation of Civil Engineering Contractors
- Institution of Civil Engineers
- Steel Construction Institute

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## National foreword

This Draft for Development has been prepared by Subcommittee B/525/8 and is the English language version of ENV 1998-1-3:1995 *Eurocode 8: Design provisions for earthquake resistance of structures — Part 1-3: General rules — Specific rules for various materials and elements* published by the European Committee for Standardization (CEN).

ENV 1998-1-3:1995 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of buildings and civil engineering works. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel, composite construction, timber, masonry and aluminium alloy, and also geotechnical and seismic design.

**This publication is not to be regarded as a British Standard.**

An ENV or European Prestandard is made available for provisional application, but it does not have the status of a European Standard. The aim is to use the experience gained to modify the ENV so that it can be adopted as a European Standard (EN).

There is no existing British Standard equivalent to ENV 1998-1-3 and there is no requirement in the Building Regulations to consider seismic actions on buildings and civil engineering works in the UK.

ENV 1998-1-1 states that the provisions of Eurocode 8 may be simplified or not observed in regions of low seismicity. Within the UK, the application of Eurocode 8 should not be necessary, unless the client or user of the works assesses that the associated seismic risk is such that it needs to be addressed.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies, both with respect to use in the UK and also in other more actively seismic areas. These comments will be taken into account when preparing the UK national response to CEN on the question of whether the ENV can be converted into an EN.

Comments should be sent in writing to the Secretary of B/525/8 at BSI, 389 Chiswick High Road, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision, by 30 April 1997. Comments after this time will still be possible through corporate bodies, such as the engineering institutions.

### Summary of pages

This document comprises a front cover, an inside front cover, pages i and ii, the ENV title page, pages 2 to 90 and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

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ICS 91.120.20

Descriptors: Buildings, structures, earthquake resistant structures, earthquake resistant design, building codes, computation

English version

**Eurocode 8 —  
Design provisions for earthquake resistance of structures —  
Part 1-3: General rules —  
Specific rules for various materials and elements**

Eurocode 8 — Conception et dimensionnement  
des structures pour la résistance aux séismes —  
Partie 1-3: Règles générales — Règles  
particulières pour divers matériaux et éléments

Eurocode 8 — Auslegung von Bauwerken gegen  
Erdbeben —  
Teil 1-3: Grundlagen — Baustoffspezifische  
Regeln für Hochbauten

This European Prestandard (ENV) was approved by CEN on 1993-12-17 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

**CEN**

European Committee for Standardization  
Comité Européen de Normalisation  
Europäisches Komitee für Normung

**Central Secretariat: rue de Stassart 36, B-1050 Brussels**

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## Foreword

### Objectives of the Eurocodes

- 1) The “structural Eurocodes” comprise a group of standards for the structural and geotechnical design of buildings and civil engineering works.
- 2) They cover execution and control only to the extent that is necessary to indicate the quality of the construction products, and the standard of the workmanship needed to comply with the assumptions of the design rules.
- 3) Until the necessary set of harmonized technical specifications for products and for the methods of testing their performance are available, some of the Structural Eurocodes cover some of these aspects in informative Annexes.

### Background of the Eurocode Programme

- 4) The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonized technical rules for the design of building and civil engineering works which would initially serve as an alternative to the different rules in force in the various Member States and would ultimately replace them. These technical rules became known as the “Structural Eurocodes”.
- 5) In 1990, after consulting their respective Member States, the CEC transferred the work of further development, issue and updating of the Structural Eurocodes to CEN, and the EFTA Secretariat agreed to support the CEN work.
- 6) CEN Technical Committee CEN/TC250 is responsible for all Structural Eurocodes.

### Eurocode Programme

- 7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:  
EN 1991 Eurocode 1 Basis of design and actions on structures  
EN 1992 Eurocode 2 Design of concrete structures  
EN 1993 Eurocode 3 Design of steel structures  
EN 1994 Eurocode 4 Design of composite steel and concrete structures  
EN 1995 Eurocode 5 Design of timber structures  
EN 1996 Eurocode 6 Design of masonry structures  
EN 1997 Eurocode 7 Geotechnical design structures  
EN 1998 Eurocode 8 Design provisions for earthquake resistance of  
EN 1999 Eurocode 9 Design of aluminium alloy structures
- 8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.
- 9) This Part 1-3 of Eurocode 8 is being published as a European Prestandard (ENV) with an initial life of three years.
- 10) This Prestandard is intended for experimental application and for the submission of comments.
- 11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.
- 12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC8 at the following address:

IPQ c/o LNEC  
Avenida do Brasil 101  
P - 1799 LISBOA Codex  
PORTUGAL

or to your national standards organization.

### National Application Documents (NAD's)

- 13) In view of the responsibilities of authorities in member countries for safety, health and other matters covered by the essential requirements of the Construction Products Directive (CPD), certain safety elements in this ENV have been assigned indicative values which are identified by □ (“boxed values”). The authorities in each member country are expected to review the “boxed values” and may substitute alternative definitive values for these safety elements for use in national application.

14) Some of the supporting European or International standards may not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document (NAD) giving any substitute definitive values for safety elements, referencing compatible supporting standards and providing guidance on the national application of this Prestandard, will be issued by each member country or its Standards Organization.

15) It is intended that this Prestandard is used in conjunction with the NAD valid in the country where the building or civil engineering works are located.

#### **Matters specific to this Prestandard**

16) The scope of Eurocode 8 is defined in ENV 1998-1-1 clause **1.1.1**; and the scope of this Part of Eurocode 8 is defined in clause **1.1**. Additional Parts of Eurocode 8 which are planned are indicated in ENV 1998-1-1 clause **1.1.3**.

17) This Prestandard was developed from one of the Parts that was included in the Draft of Eurocode 8 dated May 1988 published by the CEC and subjected to public enquiry. Such Draft contained also Parts 1-1 and 1-2 which are now presented as separate Prestandards.

18) As mentioned in clause **1.1**, attention must be paid to the fact that for the design of structures in seismic regions the provisions of Eurocode 8 are to be applied in addition to the provisions of the other relevant Eurocodes.

19) In using this Prestandard in practice, particular regard should be paid to the underlying assumptions given in clause **1.3** of Part 1-1.

20) Although an effort has been made to harmonize the various sections of this Prestandard regarding the various materials, it must be pointed out that differences still exist with respect to the quantity of the contents and to the extent of the details presented. This is the consequence of the fact that in the more seismically active regions of Europe use has been made until now almost exclusively of specific structural materials, and not of others, so that the experience available on these matters from all points of view (including behaviour, modeling, design rules and actual performance during earthquakes) is comparatively limited.

21) As allowed in Part 1-1 of Eurocode 8 this Prestandard contains in its chapter related to masonry buildings specific provisions which simplify the design of “simple masonry buildings”. These rules are mostly presented with the use of numerical values within [] so that the authorities in the member countries may adjust them to their specific conditions.

22) This Prestandard includes four annexes of informative character related to

- flowcharts for the application of section 2,
- seismic design of precast concrete buildings,
- preliminary dimensioning of boundary elements of rc-walls,
- specific rules for steel-concrete composite buildings.

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## 1 General

### 1.1 Scope

- 1) Part 1-3 is concerned with specific rules for various materials and elements used in buildings. It is applicable in conjunction with the provisions of Parts 1-1, 1-2 and 5 of this Code and with the other Eurocodes.
- 2) The following construction materials are covered:
  - Section 2: Specific rules for concrete buildings
  - Section 3: Specific rules for steel buildings
  - Section 4: Specific rules for timber buildings
  - Section 5: Specific rules for masonry buildings
- 3) Specific rules for concrete-steel composite buildings are given in the informative Annex D.

### 1.2 Symbols and definitions

#### 1.2.1 General

- 1) For the symbols and the definition of the terms used in connection with seismic actions Part 1-1 and Part 1-2 apply.
- 2) For the material-dependent symbols as well as for symbols not specifically related to earthquakes the provisions of the relevant Eurocodes apply.
- 3) In addition, the most frequently occurring symbols are listed and defined in 1.2.2.
- 4) Definitions of terms used only in a specific material section of Part 1-3 are given in the pertinent section

#### 1.2.2 Further symbols used in Part 1-3

##### 1.2.2.1 Latin upper case symbols

|              |                                                                                                  |
|--------------|--------------------------------------------------------------------------------------------------|
| A            | cross-sectional area;                                                                            |
| $A_C$        | gross cross-sectional area of concrete;                                                          |
| $A_O$        | area of the core of a concrete section (sectional area of concrete after spalling of the cover); |
| $A_S$        | area of reinforcement within the tension zone;                                                   |
| G            | shear modulus;                                                                                   |
| H            | building height;                                                                                 |
| $H_w$        | height of a wall;                                                                                |
| L            | length;                                                                                          |
| M            | moment;                                                                                          |
| $M_{pl, Rd}$ | design value of ultimate plastic moment resistance;                                              |
| $M_{Rd}$     | design value of moment resistance;                                                               |
| $M_{Sd}$     | design value of acting moment;                                                                   |
| N            | axial force;                                                                                     |
| $N_{pl, Rd}$ | design value of ultimate plastic axial force resistance;                                         |
| $N_{Rd}$     | design value of axial force resistance;                                                          |
| $N_{Sd}$     | design value of acting axial force;                                                              |
| $R_d$        | design value of resistance;                                                                      |
| $R_{fy}$     | yield resistance;                                                                                |
| T            | torsional moment;                                                                                |

|              |                                                                                                           |
|--------------|-----------------------------------------------------------------------------------------------------------|
| $V$          | shear force;                                                                                              |
| $V_o$        | acting shear force at beam end-sections due to vertical loads which are combined with the seismic action; |
| $V_{cd}$     | shear resistance of concrete compression zone;                                                            |
| $V_{dd}$     | dowel resistance of bars;                                                                                 |
| $V_{fd}$     | friction resistance;                                                                                      |
| $V_{id}$     | shear resistance of inclined bars;                                                                        |
| $V_{pl, Rd}$ | design value of ultimate plastic shear resistance;                                                        |
| $V_{Rd}$     | design value of shear resistance;                                                                         |
| $V_{Sd}$     | design value of acting shear force;                                                                       |
| $V_{Sd, CD}$ | acting shear force computed applying the capacity design criterion;                                       |

#### 1.2.2.2 Latin lower case symbols

|              |                                                                                  |
|--------------|----------------------------------------------------------------------------------|
| $b_c$        | width of a column parallel to the width $b_w$ of a beam framing into the column; |
| $b_{eff}$    | effective flange width of a beam;                                                |
| $b_j$        | effective width of a beam-column joint;                                          |
| $b_w$        | width of the web of a beam, thickness of the boundary element of a wall;         |
| $b_{wo}$     | thickness of the web of a wall;                                                  |
| $d_b$        | diameter of reinforcing bar;                                                     |
| $d_{bw}$     | diameter of hoop reinforcement in beams and columns;                             |
| $d_c$        | largest cross sectional dimension of a column;                                   |
| $d_d$        | diameter (respectively equivalent diameter) of dowel-type fastener;              |
| $f$          | strength of a material;                                                          |
| $f_{cd}$     | design value of concrete compressive strength;                                   |
| $f_t$        | tensile strength of reinforcement;                                               |
| $f_y$        | yield stress of steel;                                                           |
| $f_{y, act}$ | actual value of yield stress of steel;                                           |
| $f_{yd}$     | design value of yield stress of steel;                                           |
| $f_{yk}$     | characteristic value of yield stress of steel;                                   |
| $f_{y, nom}$ | nominal value of yield stress of steel;                                          |
| $h$          | height, depth;                                                                   |
| $h_c$        | width of a column in the direction of a beam framing into the column;            |
| $h_{cr}$     | critical height of a reinforced concrete wall;                                   |
| $h_s$        | clear storey height;                                                             |
| $h_w$        | depth of a beam;                                                                 |
| $k$          | coefficient, factor;                                                             |
| $l$          | length;                                                                          |
| $l_b$        | anchorage length of reinforcement;                                               |

|          |                                                     |
|----------|-----------------------------------------------------|
| $l_c$    | length of a wall area with confining reinforcement; |
| $l_{cl}$ | clear height of a column;                           |
| $l_{cr}$ | length of a critical region;                        |
| $l_w$    | length of a wall;                                   |
| $r$      | radius;                                             |
| $s$      | spacing of hoops in beams and columns;              |
| $s_h$    | spacing of horizontal web reinforcement in a wall;  |
| $s_v$    | spacing of vertical web reinforcement in a wall;    |
| $z$      | lever arm of internal forces;                       |

### 1.2.2.3 Greek symbols

|                 |                                                                                                                         |
|-----------------|-------------------------------------------------------------------------------------------------------------------------|
| $\alpha$        | angle, coefficient, factor, ratio;                                                                                      |
| $\alpha_{CD}$   | capacity design coefficient;                                                                                            |
| $\alpha_h$      | horizontal effectiveness of confinement;                                                                                |
| $\alpha_s$      | shear ratio;                                                                                                            |
| $\gamma$        | partial safety factor;                                                                                                  |
| $\gamma_{Rd}$   | design value of the overstrength ratio of steel;                                                                        |
| $\delta$        | moments reversal factor used in capacity design;                                                                        |
| $\epsilon$      | strain, coefficient, factor;                                                                                            |
| $\epsilon_c$    | compressive strain in the concrete;                                                                                     |
| $\epsilon_s$    | tensile strain in the steel;                                                                                            |
| $\zeta$         | coefficient, factor;                                                                                                    |
| $\lambda$       | factor accounting for the available shear resistance of plain concrete after cyclic degradation in a beam-column joint; |
| $\bar{\lambda}$ | non-dimensional slenderness;                                                                                            |
| $\mu_f$         | concrete-to-concrete friction coefficient;                                                                              |
| $\mu_{1/r}$     | conventional curvature ductility factor (CCDF) of a reinforced concrete section;                                        |
| $\mu_d$         | normalized design axial force;                                                                                          |
| $\xi$           | normalized neutral axis depth;                                                                                          |
| $\rho$          | tension reinforcement ratio;                                                                                            |
| $\rho'$         | compression reinforcement ratio;                                                                                        |
| $\rho_1$        | total longitudinal reinforcement ratio;                                                                                 |
| $\tau$          | shear stress;                                                                                                           |
| $\tau_{Rd}$     | basic design shear strength of members without shear reinforcement;                                                     |
| $\omega$        | mechanical volumetric reinforcement ratio;                                                                              |
| $\varphi$       | angle.                                                                                                                  |

## Section 2 Specific rules for concrete buildings

### 2.1 General

#### 2.1.1 Scope

1) P This section applies to the design of reinforced concrete buildings in seismic regions, henceforth called concrete buildings.

NOTE 1 In order to make the practical application of this section easier, flowcharts for its use are given in Annex A.

NOTE 2 Additional rules for the design of precast concrete buildings in seismic regions are given in Annex B.

NOTE 3 For buildings with a steel-concrete composite structure see Annex D.

NOTE 4 An annex covering prestressed concrete buildings will be drafted during the ENV-phase of Part 1-3.

2) P Concrete buildings with flat slab frames used as seismic resistant elements are not covered by this section<sup>1)</sup>.

3) P For the design of concrete buildings Eurocode 2 applies. The following rules are additional to those given in Eurocode 2.

#### 2.1.2 Definitions

1) The following terms are used in this section with the following meanings:

— **Critical region:** Region of a structural element, where the most adverse combination of action-effects (M, N, V, T) occurs and where plastic hinges may form (dissipative zone). The length of the critical region is defined for each structural element in the relevant clause of this section.

— **Residual resistance:** Resistance of a structural element after the cyclic deformation history induced by the most adverse seismic conditions, including degradation.

— **Beam:** Structural element (in general horizontal), subjected mainly to transverse loads and to a normalized design axial force of  $v_d = N_{Sd}/A_c f_{cd}$  not greater than 0,1.

— **Column:** Structural element (in general vertical), supporting other elements and/or subjected to a normalized design axial force  $v_d = N_{Sd}/A_c f_{cd}$  greater than 0,1.

— **Wall:** Structural element (in general vertical) without perforations in the areas where ductility is requested, supporting other elements and having an elongated cross-section with a length to thickness ratio  $l_w/b_w$  greater than 4.

— **Coupled wall:** Structural element composed of two or more single walls, connected in a regular pattern by adequately ductile beams ("coupling beams"), able to reduce by at least [25] % the sum of the base bending moments of the individual walls if working separately.

— **Wall system:** Structural system in which both vertical and lateral loads are mainly resisted by vertical structural walls, either coupled or uncoupled, whose shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structural system<sup>2)</sup>. A minimum torsional rigidity must also be provided (see 2.3.1).

— **Frame system:** Structural system in which both the vertical and lateral loads are mainly resisted by spatial frames whose shear resistance at the building base exceeds 65 % of the total shear resistance of the whole structural system<sup>2)</sup>. A minimum torsional rigidity must also be provided (see 2.3.1).

— **Dual system:** Structural system in which support for the vertical loads is mainly provided by a spatial frame and resistance to lateral loads is contributed in part by the frame system and in part by structural walls, single or coupled. A minimum torsional rigidity must also be provided (see 2.3.1).

— **Frame-equivalent dual system:** Dual system in which the shear resistance of the frame system at the building base is higher than 50 % of the total shear resistance of the whole structural system<sup>2)</sup>.

— **Wall-equivalent dual system:** Dual system in which the shear resistance of the walls at the building base is higher than 50 % of the total seismic resistance of the whole structural system<sup>2)</sup>.

— **Core system:** Dual or wall system not having a minimum torsional rigidity (see 2.3.1), e.g. a structural system composed of flexible frames combined with walls concentrated near the centre of the building in plan.

<sup>1)</sup> Because of the essentially non-dissipative behaviour of flat slab frames, additional measures are needed (e.g. the possible combination with other seismic resistant structural systems) and/or additional conditions should be prescribed (such as consideration of the low local ductility available or limitations related to the form and height of the building).

<sup>2)</sup> Normally the percentages of shear resistance mentioned in 2.1.2 may be substituted by percentages of shear action effects in the seismic design situation.

NOTE This definition does not cover systems containing several heavily perforated wall arrangements around vertical services and facilities. For such systems the engineer should give the most appropriate definition of the respective overall structural configuration on a case by case basis.

— **Inverted pendulum system:** System in which 50 % or more of its mass is located in the upper third of the height of the structure, or in which the dissipation of energy takes place mainly at the base of one isolated building element.

NOTE One storey frames with column-tops connected along both main directions of the building do not belong to this category.

### 2.1.3 Design concepts

1) P The design of earthquake resistant concrete buildings shall provide an adequate energy dissipation capacity to the structure without substantial reduction of its overall resistance against horizontal and vertical loading. To this end, the requirements and criteria of clause 2 of Part 1-1 apply. Adequate resistance of all structural elements shall be provided under the seismic combination of actions, whereas non-linear deformations in critical regions should allow for the overall ductility assumed in calculations.

2) An overall ductile behaviour is ensured if the ductility demand is spread over a large number of elements and locations per element<sup>3)</sup>. To this end, ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability.

3) P With regard to the required hysteretic dissipation capacity three ductility classes DC“L” (low ductility), DC“M” (medium ductility) and DC“H” (high ductility) are distinguished for concrete structures:

— DC“L”

Ductility Class “L” corresponds to structures designed and dimensioned according to Eurocode 2, supplemented by rules enhancing available ductility.

— DC“M”

Ductility Class “M” corresponds to structures designed, dimensioned and detailed according to specific earthquake resistant provisions, enabling the structure to enter well within the inelastic range under repeated reversed loading, without suffering brittle failures.

— DC“H”

Ductility Class “H” corresponds to structures for which the design, dimensioning and detailing provisions are such as to ensure, in response to the seismic excitation, the development of chosen stable mechanisms associated with large dissipation of hysteretic energy.

4) P In order to provide in the three ductility classes the appropriate amount of ductility, specific provisions for all structural elements shall be satisfied in each class (see 2.6-2.11).

5) In correspondence with the different available ductility in the three ductility classes, different values of the behaviour factor  $q$  are used for each class (see 2.3.2).

6) P In low seismicity zones (see clause 4.1 of Part 1-1) concrete buildings may be designed under the seismic load combination following only the rules of Eurocode 2 and neglecting the specific provisions given in this Section, provided a specific assessment of the behaviour factor  $q$  is made based on the principles of Eurocode 8.

## 2.2 Materials

### 2.2.1 Concrete

1) P The use of concrete class lower than C 16/20 for DC“L” or C 20/25 for DC“M” and DC“H” is not allowed.

### 2.2.2 Reinforcing steel

1) P Except for closed stirrups or cross-ties, only ribbed bars are allowed as reinforcing steel in critical regions.

2) P In critical regions reinforcing steel shall satisfy the additional requirements given in Table 2.1.

3) P Welded meshes are allowed if they observe the conditions specified in paragraphs (1) and (2) above.

<sup>3)</sup> Obviously, a single structural wall concentrates its plastic region near its base only; however well designed wall systems and dual systems may still generate energy dissipation at several locations.

**Table 2.1 — Additional requirements for reinforcing steel in critical regions<sup>4)</sup>**

| PROPERTIES                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                    |                                                                    |                                                 | DC“L”                          | DC“M”       | DC“H”       |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------|-------------------------------------------------|--------------------------------|-------------|-------------|
| i                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                             | Uniform elongation at maximum load (charact. value)                | $\epsilon_{su,k}$                               | high ductility steel acc. EC 2 | $\geq 6 \%$ | $\geq 9 \%$ |
| ii                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            | Tensile strength to yield strength ratio (mean value of the ratio) | —                                               |                                | $\geq 1,15$ | $\geq 1,20$ |
| iii                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           |                                                                    | $f_t/f_y$                                       |                                | $\leq 1,35$ | $\leq 1,35$ |
| iv                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                            | Actual to nominal yield strength ratio                             | $\frac{f_{y, \text{ act}}}{f_{y, \text{ nom}}}$ |                                | $\leq 1,25$ | $\leq 1,20$ |
| <b>Comments</b><br><b>i) and ii):</b> These rules aim at ensuring an adequate plastic hinge length and high local ductilities (high rotational capacities). In addition, higher $\epsilon_{su, k}$ and $f_t/f_y$ values lead to higher resistances after cover spalling.<br><b>iii) and iv):</b> These rules are necessary to ensure an economical and reliable control of the desired inelastic mechanisms through the capacity design procedures. If they are not respected, then appropriately higher $\gamma_{Rd}$ – factors than those given in <b>2.7 – 2.11</b> should be used, e.g. by multiplying them with $(f_t/f_y)_{act}/1,35$ . |                                                                    |                                                 |                                |             |             |

## 2.3 Structural types and behaviour factors

### 2.3.1 Structural types

1) P Concrete buildings shall be considered to belong to one of the following structural types (see 2.1.2) according to their behaviour under horizontal seismic actions:

- **frame system**,
- **dual system** (frame or wall equivalent),
- **wall system** (coupled or uncoupled),
- **core system**,
- **inverted pendulum system**.

2) P Frame, dual and wall systems shall possess a minimum torsional rigidity.

3) Such requirement of a minimum torsional rigidity is considered satisfied, if the following condition is fulfilled:

$$r/l_s \geq 0,8 \quad (2.1)$$

where

$r$  minimum torsional radius for all relevant horizontal directions (see Annex A of Part 1-2),

$l_s$  radius of gyration of the structure in plan (see Annex A of Part 1-2).

4) For frame systems in which all the vertical elements are well distributed in plan the condition set forth in paragraph 2) above may be considered as satisfied without analytical verification.

5) Frame, dual or wall systems without a minimum torsional rigidity according to exp. (2.1) should be classified as core systems.

<sup>4)</sup> Developments are expected in the technology of reinforcing steel production and standardization in Europe, which may result in changes of these provisions.

## 2.3.2 Behaviour factors

### 2.3.2.1 Horizontal seismic actions

1) P The behaviour factor  $q$ , introduced in clause 2.2.2 of Part 1-1 to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_0 k_D k_R k_W \geq 1,5 \quad (2.2)$$

where

$q_0$  basic value of the behaviour factor, dependent on the structural type [see paragraph 2)],

$k_D$  factor reflecting the ductility class [see paragraph 6)],

$k_R$  factor reflecting the structural regularity in elevation [see paragraph 8)],

$k_W$  factor reflecting the prevailing failure mode in structural systems with walls [see paragraph 9)].

2) The basic values  $q_0$  for the various structural types are given in Table 2.2.

**Table 2.2 — Basic value  $q_0$  of behaviour factor**

| STRUCTURAL TYPE          |                                       | $q_0$ |
|--------------------------|---------------------------------------|-------|
| Frame system             |                                       | 5,0   |
| Dual system              | frame equivalent                      | 5,0   |
|                          | wall equivalent, with coupled walls   | 5,0   |
|                          | wall equivalent, with uncoupled walls | 4,5   |
| Wall system              | with coupled walls                    | 5,0   |
|                          | with uncoupled walls                  | 4,0   |
| Core system              |                                       | 3,5   |
| Inverted pendulum system |                                       | 2,0   |

3) P When both coupled and uncoupled walls are included in the same structural system, the selection of the appropriate value  $q_0$  shall be made on the basis of the type of walls resisting the larger part of shear at the base of the building.

4) The value of  $q_0$  given for inverted pendulum systems may be increased provided that the designer proves that a correspondingly higher energy dissipation is ensured in the critical region of the structure.

5) P In cases where a special and formal Quality System Plan is applied in addition to normal quality control schemes, increased values of  $q_0$  may be allowed after a specific decision of the National Authorities. However, the increased values shall not exceed the values given in Table 2.2 by more than 20 %.

6) P The factor  $k_D$  reflecting the ductility class shall be taken as follows:

$$k_D = \begin{cases} 1,00 & \text{for DC"H"} \\ 0,75 & \text{for DC"M"} \\ 0,50 & \text{for DC"L"} \end{cases} \quad (2.3)$$

7) P The same ductility class shall be adopted for each design direction.

8) P The factor  $k_R$  reflecting the regularity in elevation according to clause 2.2.3 of Part 1-2 shall be taken as follows:

$$k_R = \begin{cases} 1,00 & \text{for regular structures} \\ 0,80 & \text{for non-regular structures} \end{cases} \quad (2.4)$$

9) P The factor  $k_W$  reflecting the prevailing failure mode in structural systems with walls shall be taken as follows:

$$k_W = \begin{cases} 1,00 & \text{for frame and frame equivalent} \\ & \text{dual systems} \\ 1/(2,5-0,5 \cdot \alpha_0) & \text{for wall, wall equivalent} \\ \leq 1 & \text{and core systems} \end{cases} \quad (2.5)$$

where

$\alpha_0$  prevailing aspect ratio of the walls of the structural system ( $\alpha_0 \equiv \text{prev. } (H_w/l_w)$ ).

10) If the aspect ratios  $H_{wi}/l_{wi}$  of all walls  $i$  of a structural system do not significantly differ, the prevailing aspect ratio  $\alpha_0$  may be determined as follows:

$$\alpha_0 = \Sigma H_{wi} / \Sigma l_{wi} \quad (2.6)$$

where

$H_{wi}$  height of wall  $i$ ,

$l_{wi}$  length of the section of wall  $i$ .

### 2.3.2.2 Vertical seismic actions

1) For the vertical component of the seismic action a behaviour factor  $q$  equal to 1,0 should in general be adopted for all structural systems.

2) P The adoption of  $q$ -values greater than 1,0 shall be justified through an appropriate analysis.

## 2.4 Design criteria

### 2.4.1 General

1) The design concepts described in 2.1.3 and in Part 1-1 are implemented into the earthquake resistant structural elements of concrete buildings as specified in 2.4.2 – 2.4.7.

2) The design criteria given in 2.4.2 – 2.4.7 are deemed to be satisfied, if the provisions given in 2.6 - 2.12 are observed.

### 2.4.2 Local resistance criterion

1) P All critical regions of the structure shall have a resistance adequately higher than the corresponding action effects occurring in these regions under the seismic design situation.

2) P Second order effects shall be taken into account as prescribed in Part 1-2.

### 2.4.3 Capacity design criterion

1) P Brittle or other undesirable failure mechanisms (e.g. shear failure of structural elements, failure of beam-column joints, yielding of foundations or of any element intended to remain elastic) shall be prevented by deriving the design action effects of selected regions from equilibrium conditions when plastic hinges with their possible overstrengths have been formed in their adjacent areas.

2) P Plastic hinges shall be distributed throughout the structure without concentration in any single storey (“soft storey mechanism”) and shall — except at the base of the building — develop, with adequate reliability, only in beams and not in columns.

3) Implementation of the provision given in paragraph 1) above is made easier if

— the actual yield strength of the reinforcing steel is not excessively higher than the yield strength assumed in the design,

— the ratio of the tensile strength to the yield strength of the reinforcing steel bars is appropriately limited,

as indicated in Table 2.1.

#### 2.4.4 Local ductility criterion

- 1) P For the overall ductility of the structure the potential regions for plastic hinge formation — to be defined later for each building element — shall possess high plastic rotational capacities.
- 2) Paragraph 1) is deemed to be satisfied if the following conditions are met:
  - a) A sufficient curvature ductility is provided in all critical regions, including all column ends (regardless of the intended avoidance of plastic hinge formation on columns).
  - b) Local buckling of compressed steel within potential plastic hinge regions is appropriately prevented. Relevant application rules are given in 2.7 and 2.8.
  - c) Appropriate concrete and steel qualities are adopted in order to ensure local ductility. More specifically,
    - the steel used in the critical regions has an adequate uniform plastic elongation,
    - the tensile strength to yield strength ratio of the steel used in critical regions is adequately higher than unity,

as indicated in Table 2.1.

- 3) In case more precise data are not available, paragraph 2) a) is deemed to be satisfied if the conventional curvature ductility factor (CCDF)  $\mu_{1/r}$  of these regions (defined as the ratio of the curvature at the post-failure 85 %-moment resistance level, to the curvature at yield, provided the limiting strains  $\epsilon_{cu}$  and  $\epsilon_{su, k}$  are not exceeded) is higher than the specific values given in 2.8 and 2.11 for each ductility class <sup>5)</sup>.

#### 2.4.5 Structural redundancy

- 1) P A high degree of redundancy accompanied by redistribution capacity shall be sought enabling a more widely spread energy-dissipation and an increased total dissipated energy. Consequently structural systems with lower degrees of static indeterminacy are assigned lower behaviour factors (see Table 2.2). The necessary redistribution capacity is achieved through the local ductility rules given in 2.4.4.

#### 2.4.6 Secondary resistances

- 1) Resistances or stabilizing effects not explicitly taken into account in calculations, may enhance both strength and energy dissipation (e.g. membrane reactions of slabs mobilised by upwards deflections of structural walls).
- 2) Non-structural elements may also contribute to energy dissipation provided that they are uniformly distributed throughout the structure. However, appropriate measures should be taken against possible local adverse effects due to the interaction between structural and nonstructural systems.
- 3) For the most frequent case of non-structural elements (masonry-infilled frames) special rules are given in 2.9.

<sup>5)</sup> As a simplification the conventional curvature ductility factor (CCDF) may be calculated by the following expression:

$$\mu_{1/r} = \min \left[ \begin{array}{ll} \frac{\epsilon_{cu}}{\epsilon_{sy, k}} \cdot \frac{1 - \xi_{sy}}{\xi_{cu}} & \text{(concrete failure)} \\ \frac{\epsilon_{su, k}}{\epsilon_{sy, k}} \cdot \frac{1 - \xi_{sy}}{1 - \xi_{su}} & \text{(steel failure)} \end{array} \right]$$

$\epsilon_{cu}$  concrete strain at the post-peak 0,85  $f_{ck}$  — level of the  $\sigma_c - \epsilon_c$  — diagram; in estimating the  $\epsilon_{cu}$  — value actual confinement may be taken into account,

$\epsilon_{sy, k}$  characteristic value of tensile strain of steel at yield,

$\epsilon_{su, k}$  characteristic value of uniform elongation of steel at maximum load,

$\xi_{cu}$  normalized neutral axis depth at the post-peak 85 % strength level, when concrete is critical; spalling of concrete cover is taken into account,

$\xi_{sy}$  normalized neutral axis depth at yielding of steel,

$\xi_{su}$  normalized neutral axis depth at steel failure, when the tension plastic strain of steel is critical.

### 2.4.7 Specific additional measures

1) P Due to the random nature of the seismic action and to the uncertainties of the post-elastic cyclic behaviour of concrete structures, the overall uncertainty is substantially higher than under normal actions. Therefore appropriate measures shall be taken to reduce

- uncertainties related to the structural configuration,
- uncertainties related to the analysis,
- resistance uncertainties,
- ductility uncertainties.

2) P Important resistance uncertainties may be produced by geometric errors. To minimize this type of uncertainties, the following rules shall be applied:

- a) Certain minimum dimensions of the structural elements shall be respected (see 2.7 and 2.8) to decrease the sensitivity to geometrical errors.
- b) A limitation of the minimum to maximum dimensions of linear elements shall be observed, to minimize the risk of lateral instability of these elements (see 2.7 and 2.11).
- c) Appropriate limitations of column drifts shall be respected to limit the effects of geometric non-linearity.
- d) A substantial percentage of the top reinforcement of beams at their end cross-sections shall continue along the entire length of the beam (see 2.7) to account for the uncertainty in the location of the inflection points in beams.
- e) Account shall be taken of reversals of moment not predicted by the analysis by providing minimum reinforcement at the relevant face of beams (see 2.7).

3) P In order to minimize ductility uncertainties, the following rules shall be observed:

- a) An appropriate minimum local ductility shall be provided in every seismic resistant part of the structure, independently of the ductility class adopted in design (see 2.7 and 2.8).
- b) A minimum amount of tension reinforcement shall be provided to avoid brittle failure upon cracking (see 2.7).
- c) An appropriate limitation of the normalized design axial force value shall be respected (see 2.8) to reduce the consequences of cover spalling and to avoid the larger uncertainties in the available ductility at high levels of axial force.

## 2.5 Safety verifications

1) P The combination of actions shall be taken from clause 4.4 of Part 1-1.

2) P For ultimate limit state verifications the partial safety factors for material properties  $\gamma_c$  and  $\gamma_s$  shall take into account the possible strength degradation of the materials due to the cyclic deformations.

3) If more specific data are missing, the values of the partial safety factors  $\gamma_c$  and  $\gamma_s$  given in Part 1-1 of Eurocode 2 for the fundamental load combination should be applied, assuming that due to the local ductility provisions the ratio between the residual strength after degradation and the initial one is roughly equal to the ratio between the  $\gamma_M$ -Values for accidental and fundamental load combinations.

4) If the strength degradation is appropriately accounted in the evaluation of the material properties, the  $\gamma_M$ -Values corresponding to the accidental load combination may be used.

## 2.6 Provisions for anchorages and splices

### 2.6.1 General

1) P The provisions given in 2.6.2 and 2.6.3 apply for all ductility classes.

2) P In cases where “hoops” are required as transverse reinforcement in beams, columns or walls, closed stirrups with 135° bent-in ends and  $10 d_{bw}$  long hooks shall be used.

## 2.6.2 Anchorage of reinforcement

### 2.6.2.1 Columns

1) P When calculating the anchorage length  $l_{b, net}$  of column bars which contribute to the flexural strength of elements in critical regions, the ratio of the required over the actually provided area of reinforcement  $A_{s, req}/A_{s, prov}$  in exp. (5.4) of Part 1-1 of Eurocode 2 shall be taken as equal to 1.

2) P The anchorage length  $l_{b, net}$  of column bars anchored within beam-column joints shall be measured from a point at a distance equal to  $k_b \cdot d_{bL}$  from the face of the beam (see Figure 2.1) in order to take into account the yield penetration due to the cyclic post-elastic deformations, where

$$k_b = \begin{cases} 10 & \text{for DC"H"} \\ 5 & \text{for DC"M"} \\ 0 & \text{for DC"L"} \end{cases} \quad (2.7)$$

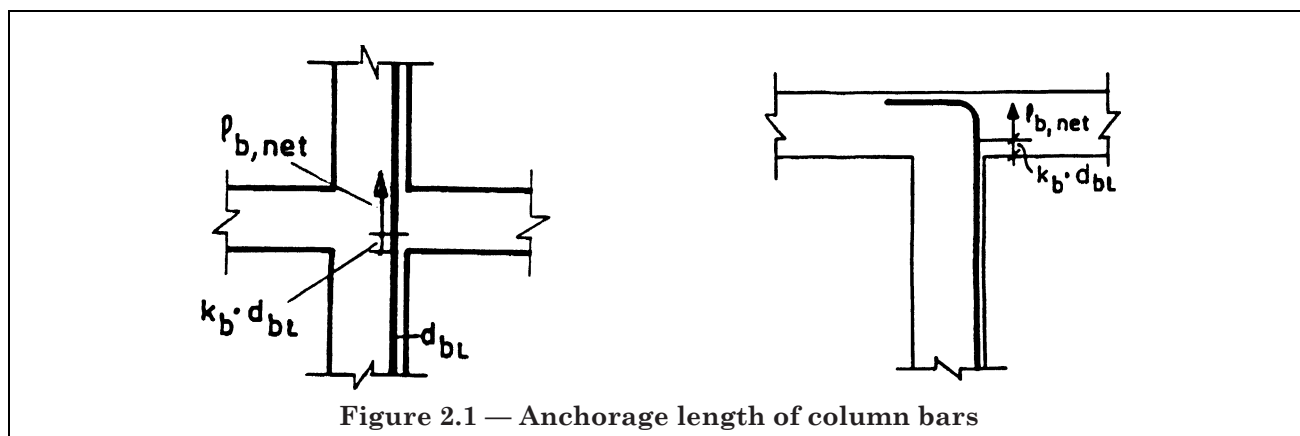


Figure 2.1 — Anchorage length of column bars

3) P If — under the seismic load combination — the axial force in a column can be tensile, the anchorage lengths shall be increased by 50 % with respect to the values specified in Part 1-1 of Eurocode 2.

### 2.6.2.2 Beams

1) P Longitudinal reinforcement of beams bent in joints for anchorage shall always be placed inside the corresponding hoops provided in the columns.

2) P In order to prevent bond failure the diameter  $d_{bL}$  of the longitudinal bars of beams anchored along the beam-column joint shall be appropriately limited according to the following expressions<sup>6)</sup>:

a) for interior beam-column joints:

$$\frac{d_{bL}}{h_c} \leq \frac{7,5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot \frac{1 + 0,8 \cdot \nu_d}{1 + k_D \cdot \rho' / \rho_{max}} \quad (2.8a)$$

b) for exterior beam-column joints:

$$\frac{d_{bL}}{h_c} \leq \frac{7,5 \cdot f_{ctm}}{\gamma_{Rd} \cdot f_{yd}} \cdot (1 + 0,8 \cdot \nu_d) \quad (2.8b)$$

<sup>6)</sup> Simplified expressions are given in the relevant clauses 2.7.2.2.1, 2.7.3.2.1 and 2.7.4.2.1.

where

- $h_c$  width of the column parallel to the bars,
- $f_{ctm}$  mean value of the tensile strength of concrete,
- $f_{yd}$  design value of the yield strength of steel,
- $v_d$  normalized design axial force in the column taken with its minimum value for the seismic load combination ( $v_d = N_{Sd}/f_{cd} \cdot A_c$ ),
- $k_D$  factor reflecting the ductility class [see 2.3.2.1 6)],
- $\rho'$  compression steel ratio of the beam bars passing through the joint,
- $\rho_{max}$  maximum allowed tension steel ratio (see 2.7),
- $\gamma_{Rd} = 1,25/1,15/1,00$  for DC“H”/DC“M”/DC“L” (value of the overstrength ratio of longitudinal steel in the beam, depending on the ductility class).

3) P If paragraph 2) cannot be satisfied in exterior beam-column joints because of the inadequate width  $h_c$  of the column parallel to the bars, the following additional measures shall be taken to ensure the anchorage of the longitudinal reinforcement of beams:

- a) Provision of horizontal beam-haunches or slab haunches [see Figure 2.2 (a)].
- b) Provision of anchorage plates welded to the ends of the bars [see Figure 2.2 (b)].
- c) Provision of bends with a minimum length of  $10 d_{bL}$  and of a transverse reinforcement made of short bars placed and tightly connected under the bends of a group of bars [see Figure 2.2 (c)].

4) P Top or bottom bars, passing through interior joints, shall end at a distance  $l_{cr}$  (length of the critical region, see 2.7.1.3.1) out-side the joint.

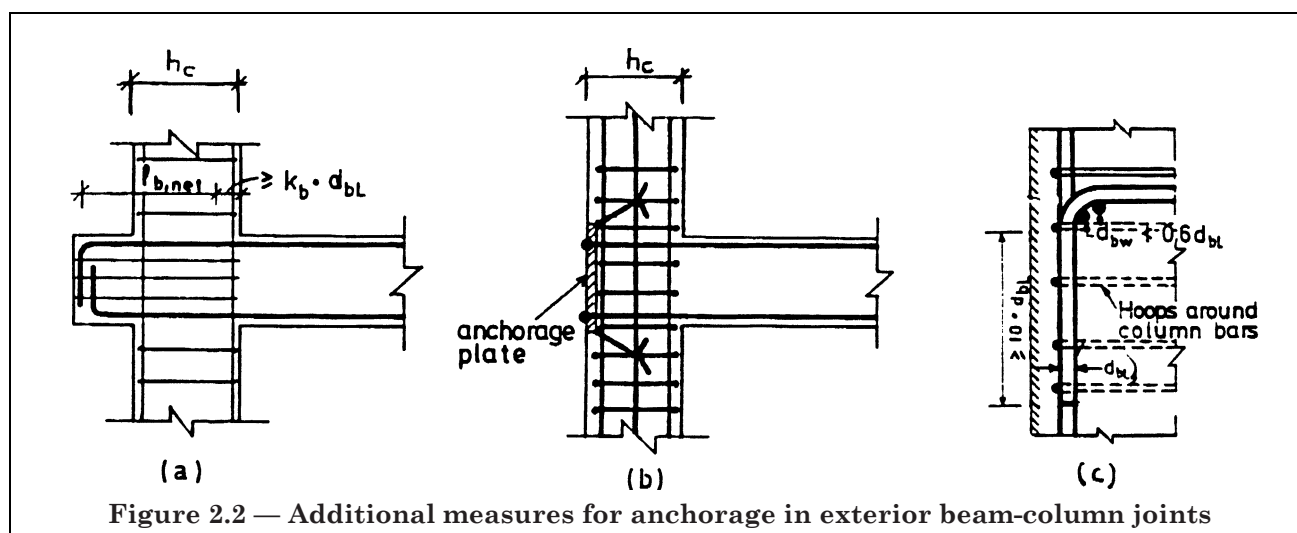


Figure 2.2 — Additional measures for anchorage in exterior beam-column joints

5) P In order to take advantage of the favourable effect of the column compression force on the bond of the horizontal bars passing through the joint (see Figure 2.3), the width  $b_w$  of the beam shall not be greater than:

$$b_w = \min \{b_c + h_w; 2b_c\} \quad (2.9)$$

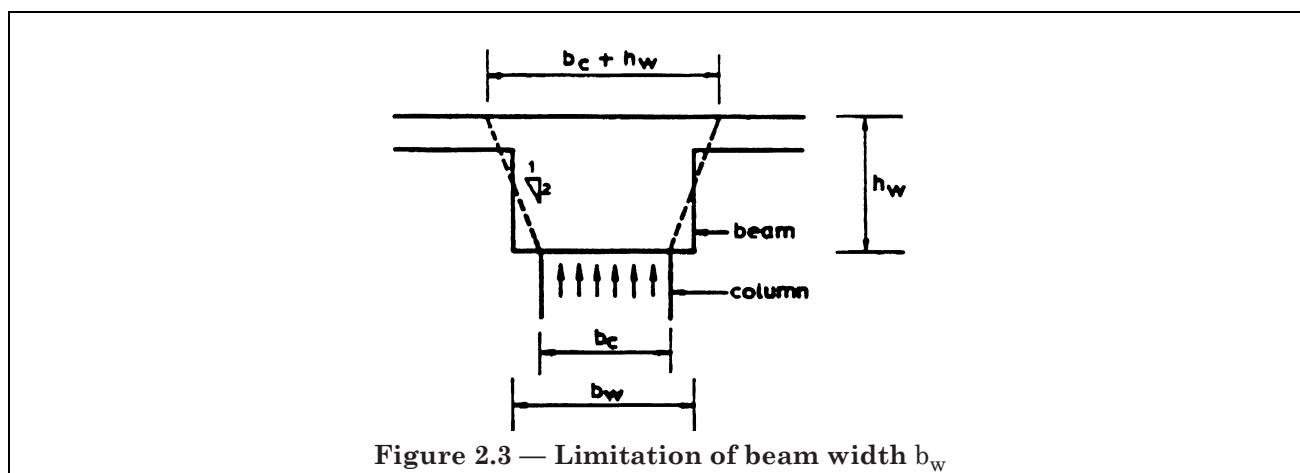


Figure 2.3 — Limitation of beam width  $b_w$

6) P The anchorage length of the bidiagonal reinforcement in coupling beams shall be 50 % longer than that calculated according to Part 1-1 of Eurocode 2 [see 2.11.2.2 3)].

### 2.6.3 Splicing of bars

- 1) P Splicing by welding is not allowed within the critical regions of structural elements.
- 2) P Splicing by mechanical couplers is allowed in columns and walls if these devices are covered by appropriate testing under conditions compatible with the ductility class selected.
- 3) P Splicing by overlapping is allowed within the critical regions of columns and walls except for the case of walls in DC“H” structures.
- 4) P The transverse reinforcement to be provided within the lap length shall be calculated according to clause 5.2.4.1.2 of Part 1-1 of Eurocode 2. However the following rules shall also be observed:

- a) For an arrangement of lap splices as in Figure 2.4 (a) the sum of the areas of all spliced bars  $\Sigma A_{sL}$  shall be used in the calculation of the transverse reinforcement.
- b) For an arrangement of lap splices as in Figure 2.4 (b) the area of the larger lapped longitudinal bar  $A_{sL}$  shall be used to calculate the transverse reinforcement.
- c) The spacing  $s$  of the transverse reinforcement to be placed along the laps shall not be greater than:
 
$$s = \min \{h/4; 100 \text{ mm}\} \quad (2.10)$$

where

$h$  minimal cross section dimension.

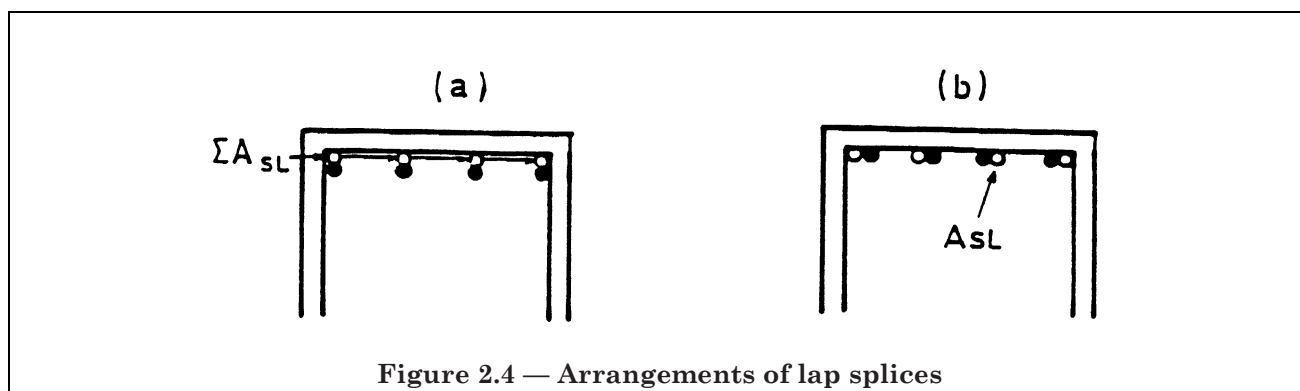


Figure 2.4 — Arrangements of lap splices

5) The transverse reinforcement  $A_{st}$  within the lap length of the longitudinal reinforcement of columns, spliced at the same location (as defined in EC2), or of the longitudinal reinforcement of boundary elements in walls, may be calculated from the following formula:

$$A_{st} = s \cdot (d_{bL}/50) \cdot (f_{yLd}/f_{ywd}) \quad (2.11)$$

where

- $A_{st}$  area of one leg of the transverse reinforcement,
- $d_{bL}$  diameter of the spliced bar,
- $s$  spacing of the transverse reinforcement,
- $f_{yLd}$  design value of the yield strength of the longitudinal reinforcement,
- $f_{ywd}$  design value of the yield strength of the transverse reinforcement.

## 2.7 Provisions for beams

### 2.7.1 General

#### 2.7.1.1 Design action effects

1) P For beams of DC“M” and DC“L” the design values of all action effects shall be obtained from the analysis of the structure for the seismic load combination according to clause 4.4 of Part 1-1. Redistribution of bending moments according to Part 1-1 of Eurocode 2 is permitted.

2) P For beams of DC“H” the design values of the bending moments shall be derived in the same way as specified in paragraph 1) above, whereas the design values of the shear forces shall be determined in accordance with the capacity design criterion as specified in 2.7.2.1.

#### 2.7.1.2 Design resistance evaluation and verification

##### 2.7.1.2.1 Bending resistance

1) The bending resistances are computed as specified in Eurocode 2, for the axial force resulting from the corresponding seismic combination.

2) The top reinforcement of the end cross-sections of T-shaped beams should be placed mainly within the width of the web. Only part of this reinforcement may be placed outside the width of the web but within the effective flange width  $b_{eff}$ .

NOTE The effective flange width is drastically reduced because of local plastification effects.

3) The effective flange width  $b_{eff}$  may be assumed as follows:

a) for beams framing into exterior columns:

$$b_{eff} = \begin{cases} b_c & \text{in the absence of a transverse beam} \\ & \text{(see fig. 2.5.a),} \\ b_c + 4 \cdot h_f & \text{if there is a transverse beam of} \\ & \text{similar dimension (see fig. 2.5.b).} \end{cases}$$

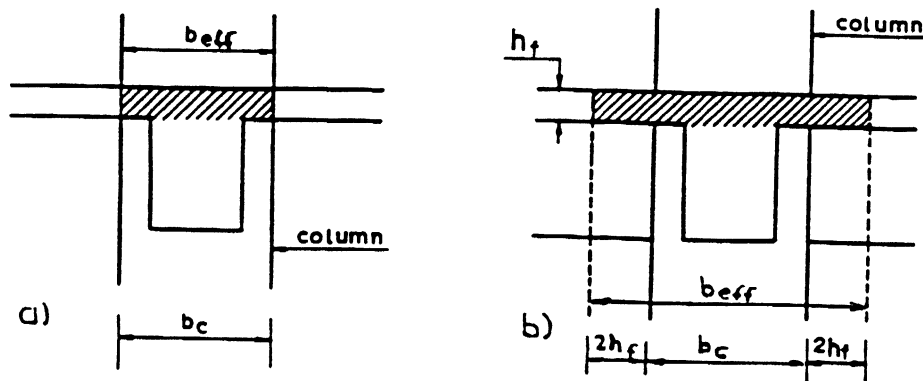


Figure 2.5 — Effective flange width  $b_{eff}$  for beams framing into exterior columns

b) for beams framing into interior columns:

the above lengths may be increased by  $2 h_f$  on each side of the beam.

- 4) For edge beams the reinforcement may be distributed over the distance from the face of the beam permitted in paragraph 3).
- 5) P Adequate anchorage and splicing of the beam reinforcement shall be ensured; the provisions of **2.6** are minimal measures to this end.
- 6) P Efficient transfer of cyclic moments from the beam to the column shall be achieved by limiting the eccentricity of the beam axis relative to that of the column into which it frames.
- 7) A deemed to satisfy rule for paragraph 6) is to limit the distance between the centroidal axes of the two members to less than  $b_c/4$ , see Figure 2.6.

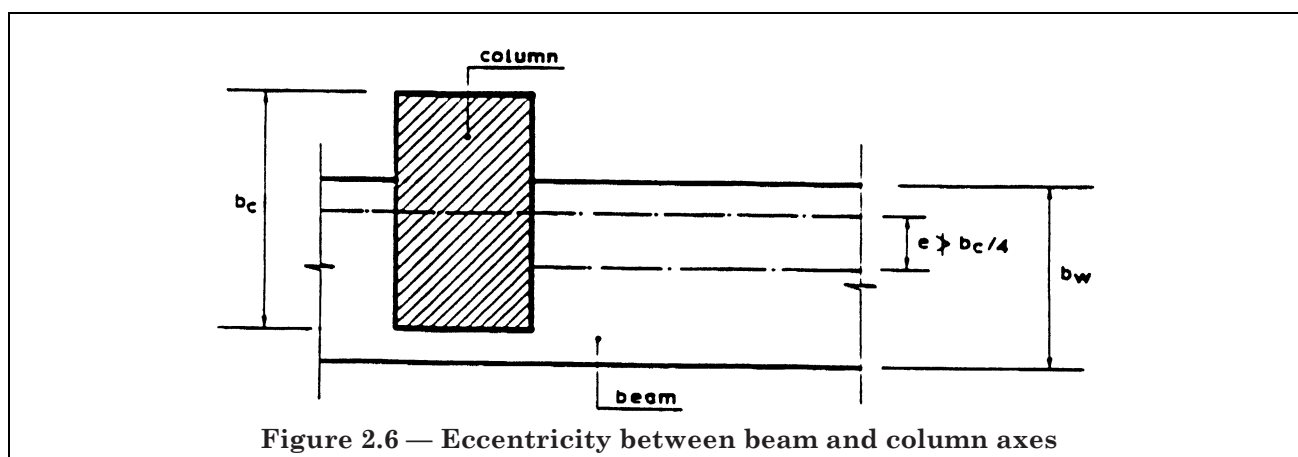


Figure 2.6 — Eccentricity between beam and column axes

#### 2.7.1.2.2 Shear resistance

- 1) P The shear resistance computations and verifications are carried out as in Eurocode 2, unless specified otherwise for each ductility class in the following clauses.

#### 2.7.1.3 Local ductility

- 1) P The regions of a beam up to a distance  $l_{cr}$  from an end cross-section where the beam frames into a joint, as well as from both sides of any other cross-section liable to yielding under the design seismic load combination, shall be considered as critical regions. The length  $l_{cr}$  is given for each ductility class in **2.7.2.3**, **2.7.3.3** and **2.7.4.3**.

- 2) Within the critical regions the local ductility requirement is deemed to be satisfied, if the following rules are met:

- a) The conditions for the transverse reinforcement, given for each ductility class in order to ensure an adequate confinement and to prevent local buckling of longitudinal bars, are satisfied.
- b) A reinforcement of not less than half the amount of the actual tension reinforcement is placed in the compression zone, in addition to the reinforcement needed for equilibrium.

- 3) Along the entire beam the necessary ductility conditions are deemed to be satisfied, if the following rules are met:

- a) The tension reinforcement ratio  $\rho$  is nowhere less than the minimum value  $\rho_{min}$ , derived as follows
 
$$\rho_{min} = 0,5 \cdot (f_{ctm}/f_{yk}) \quad (2.12)$$

and nowhere greater than the amount specified for each ductility class.

- b) The specific rules for the detailing of the reinforcement along the entire beam, given for each ductility class, are applied.

## 2.7.2 Special provisions for beams of DC“H”

### 2.7.2.1 Design action effects

1) P The design shear forces shall be determined in accordance with the capacity design criterion, considering the equilibrium of the beam under the appropriate transverse load and of a rationally selected adverse combination of the actual bending resistances of the end cross-sections, as follows (see Figure 2.7):

a) At each end cross-section, two values of the acting shear force shall be calculated, i.e. the maximum  $V_{Sd, max}$  and the minimum  $V_{Sd, min}$ , corresponding to the positive and negative resisting moments at hinges in the critical end regions.

b) These moments shall be evaluated considering the actual area of the tension steel as well as the possible occurrence of steel strengths higher than their design values. This latter fact is covered by increasing their nominal values with a  $\gamma_{Rd}$ -factor.

c) This  $\gamma_{Rd}$ -factor which is also intended to counterbalance the partial safety factor  $\gamma_s$  of steel (the same as used in the fundamental load combination) and to partially cover the hardening effects (see comments to Table 2.1) may be taken equal to 1,25.

### 2.7.2.2 Design resistance evaluation and verification

#### 2.7.2.2.1 Anchorage of longitudinal bars

1) Clause 2.6.2.2 2) is deemed to be satisfied, if

a) **for interior beam-column joints:**

$$d_{bL} \leq 4,0 \cdot (f_{ctm}/f_{yd}) \cdot (1 + 0,8 \cdot v_d) \cdot h_c \quad (2.13a)$$

b) **for exterior beam-column joints:**

$$d_{bL} \leq 6,0 \cdot (f_{ctm}/f_{yd}) \cdot (1 + 0,8 \cdot v_d) \cdot h_c \quad (2.13b)$$

#### 2.7.2.2.2 Shear resistance

1) P For the shear resistance evaluation the term  $V_{cd}$  shall be taken into account as follows:

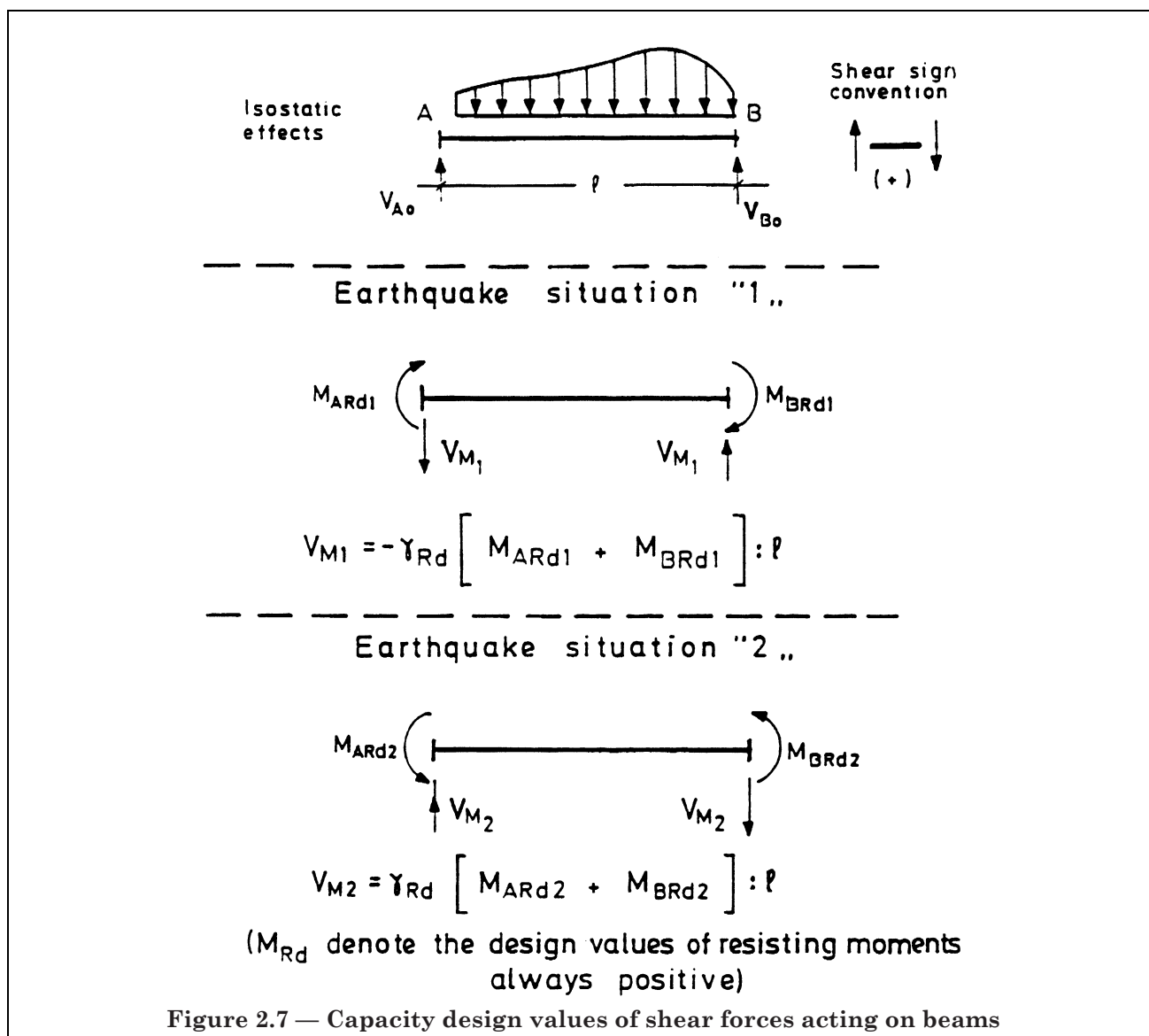
a) **in the critical regions:**

$$V_{cd} = 0$$

b) **outside the critical regions:**

$V_{cd}$  is calculated as in Eurocode 2

2) P The value of  $V_{Rd2}$  shall be taken as in clause 4.3.2.4.3 of Part 1-1 of Eurocode 2.



3) P With regard to the arrangement of shear reinforcement within the critical regions at each end of the beam, the following cases shall be distinguished depending on the algebraic value of the ratio  $\zeta = V_{Smin}/V_{Smax}$  between the minimum and maximum acting shear forces as derived from the analysis:

- a) If  $\zeta \geq -0,5$ , the shear resistance provided by the reinforcement shall be computed on the basis of the truss model in accordance with Eurocode 2.
- b) If  $\zeta < -0,5$ , i.e. when an almost full reversal of shear forces is expected, then:
  - 1) If  $|V_S|_{max} \leq 3 \cdot (2 + \zeta) \cdot \tau_{Rd} \cdot b_w \cdot d$  with  $\tau_{Rd}$ -values as in clause 4.3.2.3 of Part 1-1 of Eurocode 2, the same rule as in a) applies.
  - 2) If  $|V_S|_{max}$  exceeds the above limit value, bidiagonal reinforcement at  $\pm 45^\circ$  shall be provided as follows:
    - if  $|V_S|_{min} \leq 6 \cdot (2 + \zeta) \cdot \tau_{Rd} \cdot b_w \cdot d$ , half of  $V_{Smax}$  shall be resisted by stirrups and half by inclined reinforcement,

if  $|V_S|_{\min}$  exceeds the above limit value, the entire  $V_{S\max}$  shall be fully resisted by inclined reinforcement. In such a case, the verification is carried out by means of the condition:

$$V_{Sd} \leq \Sigma A_s \cdot f_{yd} \cdot \sqrt{2}$$

where

$A_s$  area of the inclined reinforcement in one direction, which crosses the potential sliding plane (i.e. the beam end cross-section).

### 2.7.2.3 Local ductility

1) P The length of the critical regions  $l_{cr}$  shall be taken equal to  $2,0 h_w$  (where  $h_w$  denotes the height of the beam, see Figure 2.8).

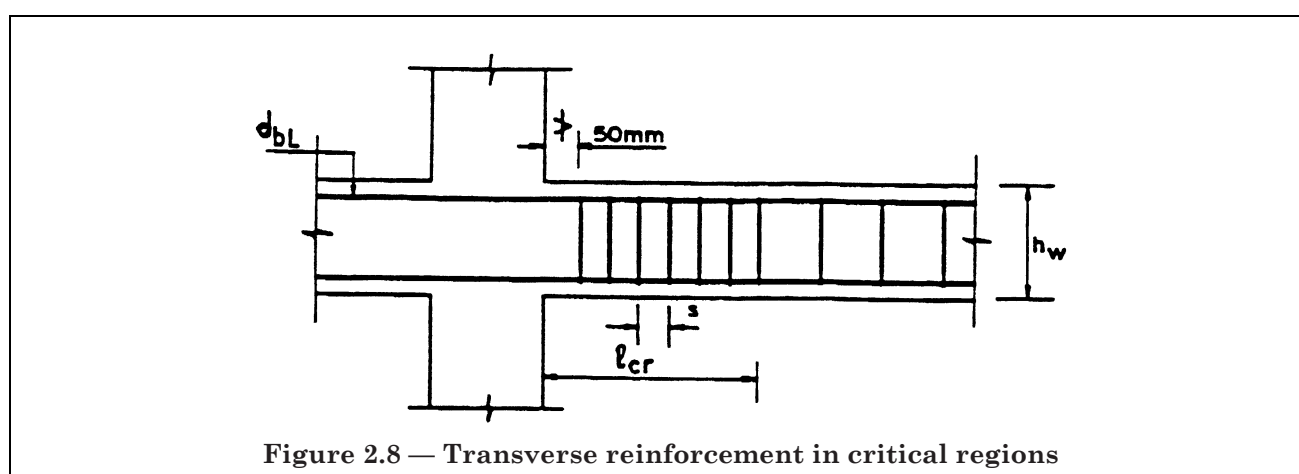


Figure 2.8 — Transverse reinforcement in critical regions

2) P Within the critical regions, hoops satisfying the following conditions shall be provided:

- The diameter  $d_{bw}$  of the hoops is not less than 6 mm.
- The spacing  $s$  of the hoops is not greater than:

$$s = \min \{h_w/4; 24 d_{bw}; 150 \text{ mm}; 5 d_{bL}\} \quad (2.14)$$

c) The first hoop is placed not more than 50 mm from the end cross-section of the beam (see Figure 2.8).

3) P Within the critical regions, the tension reinforcement ratio  $\rho$  shall not exceed the value  $\rho_{\max}$ , derived as follows:

$$\rho_{\max} = 0,35 \cdot \frac{f_{cd}}{f_{yd}} \cdot \frac{\rho'}{\rho} + 0,0015 \quad (2.15)$$

4) P At least two S400 bars with  $d_b = 14$  mm shall be provided both at the top and at the bottom sides along the entire length of the beam.

### 2.7.2.4 Specific measures

- P The minimum width of the beams shall be 200 mm.
- P The width to height ratio of the web shall satisfy the provision of clause 4.3.5.7 of Part 1-1 of Eurocode 2.
- P One fourth of the maximum top reinforcement at supports shall run along the entire beam length.

## 2.7.3 Special provisions for beams of DC“M”

### 2.7.3.1 Design action effects

- Paragraph 2.7.1.1 1) applies.

### 2.7.3.2 Design resistance evaluation and verification

#### 2.7.3.2.1 Anchorage of longitudinal bars

1) Clause 2.6.2.2 2) is deemed to be satisfied, if

a) **for interior beam-column joints:**

$$d_{bL} \leq 4,5 \cdot (f_{ctm}/f_{yd}) \cdot (1 + 0,8 \cdot v_d) \cdot h_c \quad (2.16a)$$

b) **for exterior beam-column joints:**

$$d_{bL} \leq 6,5 \cdot (f_{ctm}/f_{yd}) \cdot (1 + 0,8 \cdot v_d) \cdot h_c \quad (2.16b)$$

#### 2.7.3.2.2 Shear resistance

1) P For the shear resistance evaluation the term  $V_{cd}$  shall be taken into account as follows:

a) **in the critical regions:**

$V_{cd}$  is 40 % of the value calculated as in Eurocode 2

b) **outside the critical regions:**

$V_{cd}$  is calculated as in Eurocode 2

2) P The value of  $V_{Rd2}$  shall be taken as in clause 4.3.2.4.3 of Part 1-1 of Eurocode 2.

3) P With regard to the arrangement of shear reinforcement within the critical regions at each end of the beam, the following cases shall be distinguished depending on the algebraic value of the ratio  $\zeta = V_{Smin}/V_{Smax}$  between the minimum and maximum acting shear forces as derived from the analysis:

a) If  $\zeta \geq -0,5$ , the shear resistance provided by the reinforcement shall be computed on the basis of the truss model in accordance with Eurocode 2.

b) If  $\zeta < -0,5$ , i.e. when an almost full reversal of shear forces is expected, then:

1) If  $|V_S|_{max} \leq 4 \cdot (2 + \zeta) \cdot \tau_{Rd} \cdot b_w \cdot d$  with  $\tau_{Rd}$ -values as in clause 4.3.2.3 of Part 1-1 of Eurocode 2, the same rule as in a) applies.

2) If  $|V_S|_{max}$  exceeds the above limit value, bidiagonal reinforcement at  $\pm 45^\circ$  shall be provided as follows:

— if  $|V_S|_{min} \leq 8 \cdot (2 + \zeta) \cdot \tau_{Rd} \cdot b_w \cdot d$ , half of  $V_{Smax}$  shall be resisted by stirrups and half by inclined reinforcement,

— if  $|V_S|_{min}$  exceeds the above limit value, the entire  $V_{Smax}$  shall be fully resisted by inclined reinforcement. In such a case, the verification is carried out by means of the condition:

$$v_{Sd} \leq \sigma_{As} \cdot f_{yd} \cdot \sqrt{2}$$

where

$A_s$  area of the inclined reinforcement in one direction, which crosses the potential sliding plane (i.e. the beam end cross-section).

#### 2.7.3.3 Local ductility

1) P The length of the critical regions  $l_{cr}$  shall be taken equal to  $1,5 h_w$  (where  $h_w$  denotes the height of the beam, see Figure 2.8).

2) P Within the critical regions, hoops satisfying the following conditions shall be provided:

a) The diameter  $d_{bw}$  of the hoops is not less than 6 mm.

b) The spacing  $s$  of the hoops is not greater than:

$$s = \min \{h_w/4; 24 \cdot d_{bw}; 200 \text{ mm}; 7 \cdot d_{bL}\} \quad (2.17)$$

c) The first hoop is placed not more than 50 mm from the end cross-section of the beam (see Figure 2.8).