DESIGNERS' GUIDES TO THE EUROCODES

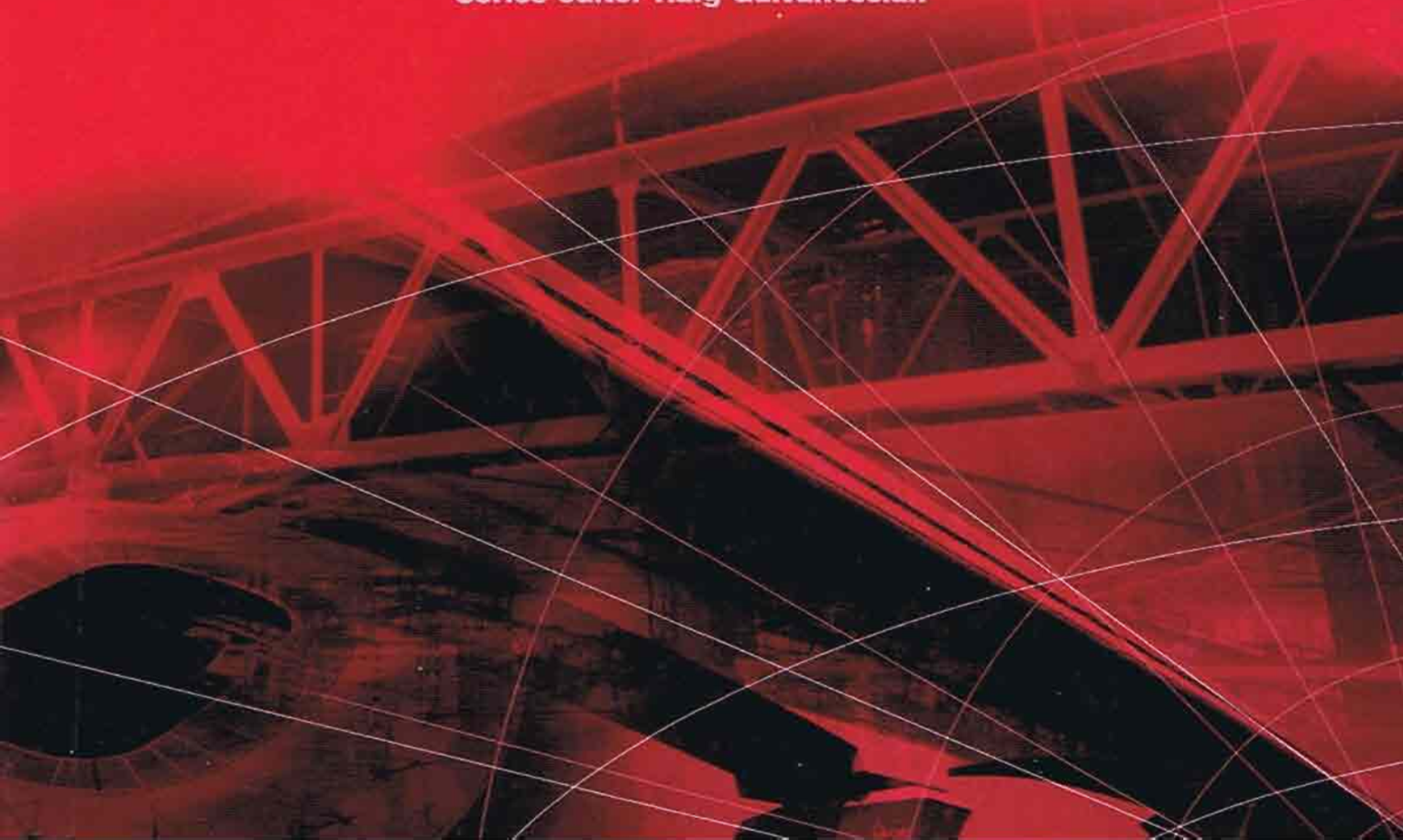
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# Designers' Guide to EN 1998-1 and EN 1998-5

## Eurocode 8: Design of structures for earthquake resistance. General rules, seismic actions, design rules for buildings, foundations and retaining structures

**Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioli, Paolo Pinto and Andre Plumier**

**Series editor Haig Gulvanessian**



DESIGNERS' GUIDES TO THE EUROCODES

# DESIGNERS' GUIDE TO EN 1998-1 AND EN 1998-5 EUROCODE 8: DESIGN OF STRUCTURES FOR EARTHQUAKE RESISTANCE

## GENERAL RULES, SEISMIC ACTIONS, DESIGN RULES FOR BUILDINGS, FOUNDATIONS AND RETAINING STRUCTURES

M. FARDIS, E. CARVALHO, A. ELNASHAI, E. FACCIOLI, P. PINTO and A. PLUMIER

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# Preface

## Aim of this guide

This Designers' Guide to EN 1998-1 and EN 1998-5 covers the rules for the seismic design of buildings and foundations, followings in a loose way the contents of these two Eurocodes. It summarizes the important points of these two Eurocodes without repeating them, and provides comments and explanations for their application, as well as background information. However, it does not elaborate on all clauses of these two Eurocodes, neither does it follow strictly the sequence of clauses.

## Layout of this guide

All cross-references in this guide to sections, clauses, subclauses, paragraphs, annexes, figures, tables and expressions of EN 1998-1 and EN 1998-5 are in italic type, which is also used where text from EN 1998-1 and EN 1998-5 has been directly reproduced (conversely, quotations from other sources, including other Eurocodes, and cross-references to sections, etc., of this guide, are in roman type). Expressions repeated from EN 1998-1 and EN 1998-5 retain their numbering; other expressions have numbers prefixed by D (for Designers' Guide), e.g. equation (D3.1).

## Acknowledgements

This guide would not have been possible without the successful completion of EN 1998-1 and EN 1998-5. Those involved in the process were:

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- The three project teams of CEN/TC250/SC8 that worked for the conversion from the ENVs to ENs, namely PT1 and PT2 for EN 1998-1, under the chairmanship of Carlos Sousa Oliveira and Jack Bouwkamp, respectively, and PT3 for EN 1998-5, under the chairmanship of Ezio Faccioli.

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CHAPTER 1

# Introduction

## 1.1. Scope of Eurocode 8

Eurocode 8, *Design of Structures for Earthquake Resistance*, covers, as its title suggests, the earthquake-resistant design and construction of buildings and other civil engineering works in seismic regions. Its stated purpose is to protect human life and property in the event of earthquakes and to ensure that structures which are important for civil protection remain operational.

*EN 1998-1: clauses 1.1.1(1), 1.1.1(2), 1.1.1(4), 1.1.3(1)*

Eurocode 8 has six parts, listed in Table 1.1. Among them, only Parts 1 (EN 1998-1, *General Rules, Seismic Actions and Rules for Buildings*)[1] and 5 (EN 1998-1, *Foundations, Retaining Structures and Geotechnical Aspects*)[2] are covered in this Designers' Guide.

The scope of Eurocode 8 does not (fully) cover special edifices, notably nuclear power plants, offshore structures and large dams.

## 1.2. Scope of Eurocode 8 – Part 1

Although its main object is buildings, EN 1998-1 also includes the general provisions for the other parts of Eurocode 8 to build on:

*EN 1998-1: clause 1.1.2*

- performance requirements
- seismic action
- analysis procedures and general concepts and rules applicable to all structures beyond buildings.

**Table 1.1.** Eurocode 8 parts and key dates (achievement or expectation, as of January 2005)

| Eurocode 8 part | Title | Approval by SC8 for formal voting | Availability from CEN of approved EN in English, French, and German to CEN members |
|---|---|---|---|
| EN 1998-1 | *General Rules, Seismic Actions, Rules for Buildings* | July 02 | Dec. 04 |
| EN 1998-2 | *Bridges* | Sept. 03 | Oct. 05 |
| EN 1998-3 | *Assessment and Retrofitting of Buildings* | Sept. 03 | June 05 |
| EN 1998-4 | *Silos, Tanks, Pipelines* | March 05 | June 06 |
| EN 1998-5 | *Foundations, Retaining Structures, Geotechnical aspects* | July 02 | Nov. 04 |
| EN 1998-6 | *Towers, Masts, Chimneys* | July 04 | June 05 |

**Table 1.2.** Eurocode 8 parts in Eurocode packages – estimated dates of withdrawal of conflicting national standards

| Package No. and subject | | Expected withdrawal of conflicting national standards | Eurocode 8 part | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 | 5 | 6 |
| 2/1 | Concrete buildings | March 2010 | ✓ | | ✓ | | ✓ | |
| 3/1 | Steel buildings | March 2010 | ✓ | | ✓ | | ✓ | |
| 4/1 | Composite (steel–concrete) buildings | March 2010 | ✓ | | ✓ | | ✓ | |
| 5/1 | Timber buildings | March 2010 | ✓ | | ✓ | | ✓ | |
| 6 | Masonry buildings | March 2010 | ✓ | | ✓ | | ✓ | |
| 7 | Aluminium structures | March 2010 | ✓ | | | | ✓ | |
| 2/2 | Concrete bridges | March 2010 | ✓ | ✓ | | | ✓ | |
| 3/2 | Steel bridges | March 2010 | ✓ | ✓ | | | ✓ | |
| 4/2 | Composite bridges | March 2010 | ✓ | ✓ | | | ✓ | |
| 5/2 | Timber bridges | March 2010 | ✓ | ✓ | | | ✓ | |
| 2/3 | Concrete liquid retaining and containment structures | March 2010 | ✓ | | | ✓ | ✓ | |
| 3/3 | Steel silos, tanks and pipelines | March 2010 | ✓ | | | ✓ | ✓ | |
| 3/4 | Steel piling | March 2010 | ✓ | | | | ✓ | |
| 3/5 | Steel cranes | March 2010 | ✓ | | | | ✓ | |
| 3/6 | Steel towers and masts | March 2010 | ✓ | | | | ✓ | ✓ |

EN 1998-1 covers in separate sections the design and detailing rules for buildings constructed with the main structural materials:

- concrete
- steel
- composite (steel–concrete)
- timber
- masonry.

It also covers seismic design of buildings using base isolation.

## 1.3. Scope of Eurocode 8 – Part 5

*EN 1998-5: clauses 1.1(1), 1.1(2)*

EN 1998-5 establishes the requirements, criteria and rules for the siting and foundation soil of structures for earthquake resistance. It covers the design of different foundation systems and earth-retaining structures under seismic actions, as well as the special issue of soil–structure interaction. It applies to all types of earthquake-resistant structures, beyond buildings. In that sense, along with *Sections 2* and *3* of EN 1998-1 that define the performance requirements and the seismic action, EN 1998-5 provides the 'foundation' for the rest of Eurocode 8 (the other five parts).

## 1.4. Use of Eurocode 8 – Parts 1 and 5 with the other Eurocodes

*EN 1998-1: clauses 1.1.1(3), 1.2.1, 1.2.2(1)*

Eurocode 8 is not a standalone code. It will be applied along with the other relevant Eurocodes, as part of Eurocode packages. Each package will refer to a specific type of civil engineering structure and construction material. The first column of Table 1.2 lists all Eurocode packages. To be self-sufficient for design, each package will also include the necessary parts of EN 1990, *Eurocode: Basis of Structural Design*, of EN 1991, *Eurocode 1: Actions on Structures*, and EN 1997, *Eurocode 7: Geotechnical Design*. Packages will contain the appropriate parts of Eurocode 8 as shown in Table 1.2.

## 1.5. Assumptions – distinction between Principles and Application Rules

Eurocode 8 refers to EN 1990[3] for assumptions and for the distinction between Principles and Application Rules. Accordingly, reference is also made here to the Designers' Guides on other Eurocodes for elaboration.

*EN 1998-1: clauses 1.3, 1.4*

## 1.6. Terms and definitions – symbols

Terms and symbols are defined in the chapter of this guide in which they first occur.

*EN 1998-1: clauses 1.5, 1.6*

CHAPTER 2

# Performance requirements and compliance criteria

## 2.1. Performance requirements for new designs in Eurocode 8 and associated seismic hazard levels

As a European standard (EN), Part 1 of Eurocode 8 provides for a two-level seismic design with the following explicit performance objectives: *Clause 2.1(1)*

- No-(local-)collapse: protection of life under a rare seismic action, through prevention of collapse of the structure or its parts and retention of structural integrity and residual load capacity after the event. This implies that the structure is significantly damaged, and may have moderate permanent drifts, but retains its full vertical load-bearing capacity and sufficient residual lateral strength and stiffness to protect life even during strong aftershocks. However, its repair may be uneconomic.
- Damage limitation: reduction of property loss, through limitation of structural and non-structural damage in frequent earthquakes. The structure itself has no permanent drifts; its elements have no permanent deformations, retain fully their strength and stiffness, and do not need repair. Non-structural elements may suffer some damage, which can be easily and economically repaired at a later stage.

The no-(local-)collapse performance level is achieved by dimensioning and detailing structural elements for a combination of strength and ductility that provides a safety factor between 1.5 and 2 against substantial loss of lateral load resistance. The damage limitation performance level is achieved by limiting the overall deformations (lateral displacements) of the system to levels acceptable for the integrity of all its parts (including non-structural ones).

The two explicit performance levels – (local) collapse prevention and damage limitation – are pursued under two different seismic actions. The seismic action under which (local) collapse should be prevented is termed the design seismic action, whilst the one under which damage limitation is pursued is often termed the serviceability seismic action. Within the philosophy of national competence on issues of safety and economy, the hazard levels for these two seismic actions are left for national determination. For structures of ordinary importance the recommendation in EN 1998-1 is for:

- a design seismic action (for local collapse prevention) with 10% exceedance probability in 50 years (mean return period: 475 years)
- a serviceability seismic action (for damage limitation) with 10% exceedance probability in 10 years (mean return period: 95 years).

The design seismic action for structures of ordinary importance is the reference seismic action; its mean return period is termed the reference return period, and denoted by $T_{NCR}$. The ratio, $\nu$, of the serviceability seismic action (for damage limitation) to the design seismic action (for local collapse prevention) reflects the difference in hazard levels, and is a nationally determined parameter (NDP).

*Clauses 2.1(2), 2.1(3), 2.1(4), 4.2.5(1), 4.2.5(2), 4.2.5(3), 4.2.5(4), 4.2.5(5)*

Enhanced performance of essential- or high-occupancy facilities is achieved not by upgrading the performance level, as often specified in US codes, but by modifying the hazard level (the mean return period) for which local-collapse prevention or damage limitation is pursued. For essential- or high-occupancy structures the seismic action should be increased, by multiplying the reference seismic action by an importance factor, $\gamma_I$. By definition, $\gamma_I = 1.0$ for structures of ordinary importance (i.e. for the reference return period of the seismic action).

For buildings, the recommended value of the NDP importance factor $\gamma_I$ is 1.2, if collapse of the building may have unusually severe social or economic consequences (high-occupancy buildings, such as schools, or public assembly halls, facilities housing institutions of cultural importance, such as museums, etc.). These are termed buildings of Importance Class III. Buildings which are essential for civil protection in the immediate post-earthquake period, such as hospitals, fire or police stations and power plants, belong in Importance Class IV; the recommended value of the NDP importance factor for them is $\gamma_I = 1.4$. A value of $\gamma_I$ equal to 0.8 is recommended for buildings of minor importance for public safety (Importance Class I: agricultural buildings, etc.). All other buildings are considered to be of ordinary importance, and are classified as Importance Class II.

*Clauses 4.4.3.2(2), 2.2.3(2)*

For buildings of ordinary or lower importance (Importance Classes I and II) a value of 0.5 is recommended for the ratio $\nu$ of the serviceability seismic action (for damage limitation) to the design seismic action (for local collapse prevention). For buildings of importance above ordinary (Importance Classes III and IV) a value of 0.4 is recommended for $\nu$. This gives about the same level of property protection for ordinary and high-occupancy buildings (Importance Classes II and III), 15–20% less property protection for buildings of low importance and 15% higher protection for essential facilities. This additional margin may allow help facilities important for civil protection to maintain a minimum level of operation of vital services during or immediately after a frequent event.

*Clause 2.1(4)*

Despite the fact that EN 1998-1 recommends specific values for the NDPs – the importance factor of structures of other than ordinary importance, $\gamma_I$, and the ratio of the serviceability seismic action to the design seismic action, $\nu$ – the nationally or regionally used values should reflect, in addition to national choice regarding the levels of safety and protection of property, also the regional seismo-tectonic environment. Eurocode 8 gives in a note the approach that may be used to determine the ratio of the seismic action at two different hazard levels. More specifically, the usual approximation of the annual rate of exceedance, $H(a_g)$, of the peak ground acceleration $a_g$ as $H(a_g) \sim k_o a_g^{-k}$ is mentioned, with the value of the exponent $k$ depending on seismicity, but being generally of the order of 3. Then, the Poisson assumption for earthquake occurrence gives a value of $\sim(T_L/T_{LR})^{1/k}$ for the value by which the reference seismic action needs to be multiplied to achieve the same probability of exceedance in $T_L$ years as in the $T_{LR}$ years for which the reference seismic action is defined (here, the index L denotes 'lifetime'). This value is the importance factor $\gamma_I$, or the conversion factor to the serviceability seismic action, $\nu$. Alternatively, the value of the multiplicative factor, $\gamma_I$ or $\nu$, to be applied on the reference seismic action in order to achieve a value of the probability of exceedance of the seismic action, $P_L$, in $T_L$ years other than the reference probability $P_{LR}$, over the same $T_L$ years, may be estimated as $\sim(P_{LR}/P_L)^{1/k}$. For Importance Classes III and IV, $T_{LR} < T_L$ and $P_{LR} > P_L$; then $\gamma_I > 1$. For Importance Class I and for the serviceability seismic action, $T_{LR} > T_L$ and $P_{LR} < P_L$; then the importance factor $\gamma_I$ of low-importance facilities and the factor $\nu$ result in values of less than 1. It is noted that the combination of 0.4 and 0.5 for the values recommended for the ratio $\nu$ of a serviceability seismic action with a recommended mean return period of 95 years to the design seismic action with a recommended mean return period of 475 years is consistent with a value of the

exponent $k$ for the decay of the annual rate of exceedance of the peak ground acceleration, $H(a_g)$, with a value of $k$ around 2.

Although not explicitly stated, an additional performance objective in buildings designed for energy dissipation is prevention of global collapse during a very strong and rare earthquake (with a mean return period in the order of 2000 years). Although structural elements can still carry their tributary gravity loads after such an event, the structure may be heavily damaged, have large permanent drifts, retain little residual lateral strength or stiffness and may collapse after a strong aftershock. Moreover, its repair may be unfeasible or economically prohibitive. This implicit performance objective is pursued through systematic and across-the-board application of the capacity design concept, which allows full control of the inelastic response mechanism.

*Clauses 2.2.1(2), 2.2.4.1(2), 4.4.2.3(2), 4.4.2.6(2)*

## 2.2. Compliance criteria for the performance requirements and their implementation

### 2.2.1. Compliance criteria for damage limitation

An earthquake represents for the structure a demand to accommodate a given energy input or given imposed dynamic displacements. Seismic damage to structural elements, or even to non-structural ones that follow the deformations of the structure, is due to deformations induced by the seismic response. Consistent with this reality, Eurocode 8 states that compliance criteria for the damage limitation limit state (i.e. performance level) should be expressed in terms of deformation limits. For equipment mounted or supported on the structure, limits relevant to damage may be expressed in terms of response accelerations at the positions of the equipment supports.

*Clauses 2.2.1(1), 2.2.3(1)*

### 2.2.2. Compliance criteria for the no-(local-)collapse requirement

The no-(local-)collapse performance level is considered as the ultimate limit state against which the structure should be designed according to the EN 1990 on the basis of structural design.[3] Unlike the damage limitation limit state, which is verified on the basis of deformation-based criteria, design for the no-(local-)collapse ultimate limit state is force-based. This is against the physical reality showing that it is the deformation that causes a structural member to lose its lateral load resistance and it is lateral displacements (and not lateral forces) that cause structures to collapse under their own weight. Force-based seismic design is well established, because structural engineers are familiar with force-based design for other types of action (such as gravity and wind loads), because static equilibrium for a set of prescribed external loads represents a robust basis of analysis methods and, last but not least, because tools for verification of structures for seismic deformations are not yet fully developed for practical application. This last statement refers both to non-linear analysis methods for the calculation of deformation demands and to the methods for the estimation of deformation capacities of structural members.

*Clauses 2.2.1(1), 2.2.2(1), 2.2.2(2)*

#### *2.2.2.1. Design for energy dissipation and ductility*

Fulfilment of the no-(local-)collapse requirement under the design seismic action does not mean that the structure has to remain elastic under this action: this would require it to be designed for lateral forces of the order of 50% or more of its weight. Although technically feasible, designing a structure to respond elastically to its design seismic action is economically prohibitive. It is also unnecessary, as an earthquake is a dynamic action, representing for a structure a certain total energy input and a demand to tolerate certain displacements and deformations, but not a demand to withstand specific forces. So, Eurocode 8 allows a structure to develop significant inelastic deformations under its design seismic action, provided that the integrity of individual members and of the structure as a whole is not endangered. This is termed seismic design for energy dissipation and ductility.

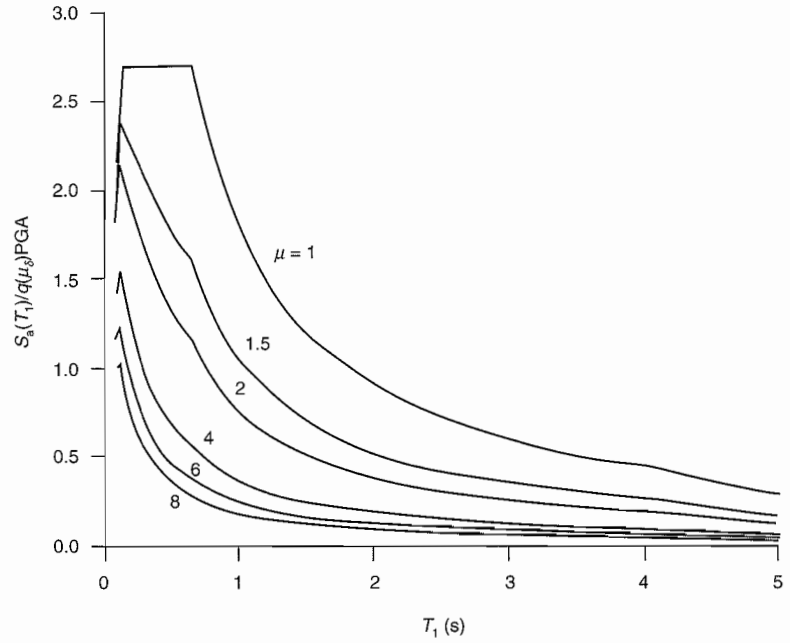



**Fig. 2.1.** Inelastic spectra for $T_C = 0.6$ s, normalized to peak ground acceleration (PGA), according to Vidic *et al.*[4] and equations (D2.1) and (D2.2)

The foundation of force-based seismic design for ductility is the inelastic response spectrum of a single-degree-of-freedom (SDOF) system which has an elastic–perfectly plastic force–displacement curve, $F$–$\delta$, in monotonic loading. For a given period, $T$, of the elastic SDOF system, the inelastic spectrum relates to:

- the ratio $q = F_{el}/F_y$ of the peak force, $F_{el}$, that would develop if the SDOF system were linear-elastic to the yield force of the system, $F_y$
- the maximum displacement demand of the inelastic SDOF system, $\delta_{max}$, expressed as a ratio to the yield displacement, $\delta_y$ (i.e. as the displacement ductility factor, $\mu_\delta = \delta_{max}/\delta_y$).

For example, Eurocode 8 has adopted the inelastic spectra proposed in Vidic *et al.*:[4]

$$\mu_\delta = q \qquad \text{if } T \geq T_C \tag{D2.1}$$

$$\mu_\delta = 1 + (q-1)\frac{T_C}{T} \qquad \text{if } T < T_C \tag{D2.2}$$

where $T_C$ is the transition period of the elastic spectrum, between its constant spectral pseudo-acceleration and constant spectral pseudovelocity ranges (Fig. 2.1). Equation (D2.1) expresses the well-known Newmark 'equal displacement rule', i.e. the empirical observation that in the constant spectral pseudovelocity range the peak displacement response of the inelastic and of the elastic SDOF systems are about the same.

With $F$ being the total lateral force on the structure (the base shear, if the seismic action is in the horizontal direction), the ratio $q = F_{el}/F_y$ is termed in Eurocode 8 the behaviour factor. In North America the same quantity is termed the force reduction factor or the response modification factor, and denoted by $R$. It is used in Eurocode 8 as a universal reduction factor on the internal forces that would develop in the elastic structure for 5% damping, or, equivalently, on the seismic inertia forces that would develop in this elastic structure, causing in turn the seismic internal forces. With this 'stratagem', the seismic internal forces for which the members of the structure should be dimensioned can be calculated through linear elastic analysis. As a price to pay, the structure has to be provided with the capacity to sustain a peak global displacement at least equal to its global yield displacement multiplied by the displacement

ductility factor, $\mu_\delta$, that corresponds to the value of $q$ used for the reduction of elastic force demands (e.g. according to equations (D2.1) and (D2.2)). This is termed ductility capacity or energy dissipation capacity – as it has to develop through cyclic response in which the members and the structure as a whole dissipate part of the seismic energy input through hysteresis.

Not all locations or parts of a structure are capable of ductile behaviour and hysteretic energy dissipation. A special instrument, termed capacity design, is used in Eurocode 8 to provide the necessary hierarchy of strengths between adjacent structural members or regions and between different mechanisms of load transfer within the same member, and ensures that inelastic deformations will take place only in those members, regions and mechanisms capable of ductile behaviour and hysteretic energy dissipation, while the rest stay in the elastic range of response. The regions of members entrusted for hysteretic energy dissipation are termed dissipative zones. They are designed and detailed to provide the required ductility and energy dissipation capacity.

*Clauses 2.2.4.1(2), 2.2.4.1(3)*

Before being designed and detailed for the necessary ductility and energy dissipation capacity, dissipative zones should first be dimensioned to provide a design value of force resistance, $R_d$, at least equal to the design value of the action effect due to the seismic design situation, $E_d$, from the analysis:

*Clauses 2.2.2(1), 2.2.2(5), 4.4.2.2(1)*

$$E_d \leq R_d \tag{D2.3}$$

The value to be used for $E_d$ in equation (D2.3) is obtained from the application of the seismic action together with the quasi-permanent value of the other actions included in the seismic design situation (i.e. the nominal value of the permanent loads and the quasi-permanent value of imposed and snow loads, see Section 4.4.1). Normally, linear analysis is used, and the value of $E_d$ may then be found by superposition of the seismic action effects from an analysis for the seismic action alone to the action effects from the analysis for the other actions in the seismic design situation. Second-order effects should be taken into account in the calculation of $E_d$.

The value of $R_d$ in equation (D2.3) should be calculated according to the relevant rules of the corresponding material Eurocode (unless these rules do not apply under inelastic cyclic loading and Eurocode 8 specifies alternative rules). It should be based on the design values of material strengths, i.e. the characteristic values, $f_k$, divided by the partial factor $\gamma_M$ of the material. Being key safety elements, the partial factors, $\gamma_M$, are NDPs with values defined in the National Annexes of Eurocode 8. Eurocode 8 itself does not recommend the values of $\gamma_M$ to be used in the seismic design situation – it just notes the options of choosing the value $\gamma_M = 1$ appropriate for the accidental design situations, or the same values as for the persistent and transient design situation. This latter option is very convenient for the designer, as he or she may then dimension the dissipative zone to provide a design value of force resistance, $R_d$, at least equal to the largest design value of the action effect due to the persistent and transient or the seismic design situation. With the former choice, the dissipative zone will have to be dimensioned once for the action effect due to the persistent and transient design situation and then for that due to the seismic design situation, each time using different values of $\gamma_M$ for the resistance side of equation (D2.3).

All regions and mechanisms not designated as dissipative zones are designed to provide a design value of force resistance, $R_d$, at least equal to an action effect, $E_d$, which is not obtained through analysis but through capacity design.

*Clause 2.2.4.1(2)*

The foundation is of paramount importance for the whole structure. Moreover, the foundation is difficult to inspect for seismic damage and even more difficult to repair or retrofit. Therefore, it is ranked at the top of the hierarchy of strengths in the entire structural system, and should be designed to remain elastic, while inelastic deformations and hysteretic energy dissipation takes place in the superstructure it supports.

*Clause 2.2.2(4)*

#### 2.2.2.2. *Seismic design for strength instead of ductility*

For buildings, Eurocode 8 gives the option of seismic design for strength alone, without observing any provisions for ductility and energy dissipation capacity. In this option the

*Clauses 2.2.1(3), 3.2.1(4)*

building is designed in accordance with Eurocodes 2 to 7, simply considering the seismic action as a lateral loading like wind. The seismic lateral forces are derived from the design response spectrum using a behaviour factor, $q$, of 1.5, at most (or possibly 2 for steel or composite buildings). Moreover, certain minimum requirements for ductility of the materials (or of steel sections) should be observed as well. As design seismic forces are derived with a value of the behaviour factor, $q$, greater than 1.0, structures designed for strength alone, without engineered ductility and energy dissipation capacity, are termed low-dissipative instead of non-dissipative.

Eurocode 8 states that the option of low-dissipative seismic design for strength alone is not recommended except in cases of low seismicity. Although it leaves it to the National Annex to decide which combination of categories of structures, ground types and seismic zones in a country correspond to the characterization as cases of low seismicity, it recommends (in a note) as a criterion either the value of the design ground acceleration on type A ground (i.e. on rock), $a_g$, or the corresponding value, $a_gS$, over the ground type of the site (the soil factor, $S$, is discussed in Section 3.2.2.2). Moreover, it recommends a value of $0.08g$ for $a_g$, or of $0.10g$ for $a_gS$, as the threshold for the low-seismicity cases. It should be recalled that the value of $a_g$ includes the importance factor $\gamma_I$.

*Clause 4.4.1(2)* For buildings, low-dissipative seismic design according to the first paragraph of this subsection – for strength alone without engineered ductility – is allowed in a specific case that may not necessarily fall within the category of low seismicity: when in the horizontal direction considered, the total base shear over the entire structure at the base level (the foundation or top of a rigid basement) due to the seismic design situation calculated with a behaviour factor equal to the value applicable to low-dissipative structures (see the first paragraph of this subsection) is less than that due to the design wind action, or any other relevant action combination for which the building is designed on the basis of a linear elastic analysis.

*Clause 10.10(5)* In buildings designed with seismic isolation, and irrespective of the classification of the building as a low-seismicity case or not, design of the superstructure above the level of the isolation (the 'isolation interface') as low-dissipative with a value of the behaviour factor, $q$, not greater than 1.5 is the rule imposed by EN 1998-1 rather than the exception.

#### *2.2.2.3. The balance between strength and ductility – ductility classification*

*Clause 2.2.2(2)* The option described in the previous subsection, namely design for strength alone, without engineered ductility and energy dissipation capacity, is an extreme, recommended by Eurocode 8 only for special cases. However, within the fundamental case of seismic design, namely that of design for ductility and energy dissipation capacity, the designer is normally given the option to design for more strength and less ductility, or vice versa. For buildings of concrete, steel, composite (steel–concrete) or timber construction, this option is exercised through the ductility classification introduced by Eurocode 8 in the corresponding material- specific chapters.

## 2.3. Exemption from the application of Eurocode 8

*Clauses 2.2.1(4), 3.2.1(5)* Eurocode 8 itself states that its provisions need not be applied in cases of very low seismicity. As for cases of low seismicity, which combination of categories of structures, ground types and seismic zones in a country will qualify as cases of very low seismicity is left to the National Annex. However, it recommends (in a note) the same criterion as for the cases of low seismicity: either the value of the design ground acceleration on type A ground (i.e. on rock), $a_g$, or the corresponding value, $a_gS$, over the ground type of the site. It goes on to recommend a value of $0.04g$ for $a_g$, or of $0.05g$ for $a_gS$, as the threshold for the very low-seismicity cases. As the value of $a_g$ includes the importance factor $\gamma_I$, certain structures in a region may be exempted from the application of Eurocode 8, while others (those housing essential or high-occupancy facilities) may not be. This is consistent with the notion that exemption from

the application of Eurocode 8 is due to the inherent lateral force resistance of any structure designed for non-seismic loadings, neglecting any contribution from ductility and energy dissipation capacity. Given that Eurocode 8 considers that, due to overstrength, any structure is entitled to a behaviour factor, $q$, at least equal to 1.5, implicit in the value of $0.05g$ for $a_g S$ recommended for the threshold for very low-seismicity cases is an assumed inherent lateral force capacity of $0.05 \times 2.5/1.5 = 0.083g$. This is indeed a reasonable assumption.

If a National Annex states that the entire national territory is considered to be a case of very low seismicity, then Eurocode 8 (all six parts) will not apply at all in that country.

CHAPTER 3

# Seismic actions

## 3.1. Ground conditions

The earthquake response of structures is significantly affected by the underlying soil condition. In this section, general guidelines and requisites for ground conditions are provided. The properties of the ground type at a given site can be characterized through adequate geotechnical investigations, *in situ* and/or in the laboratory. Rules for the identification of ground types are given in a simplified fashion in *clause 3.1.1* of EN 1998-1. Guidance for soil investigations and classification can be found in *clause 4.2* of EN 1998-5. *Clause 3.1.1*

Earthquakes may cause several types of ground effects, which can be either direct or indirect (Fig. 3.1); large ground deformations are a serious contributor to the damage of structural systems.

Generally, seismic hazard due to direct ground effects can be prevented by defining requisites for sound construction sites. *Clause 3.1.1(3)* of EN 1998-1 states that the occurrence of ground rupture, slope instability and permanent settlements caused by liquefaction or

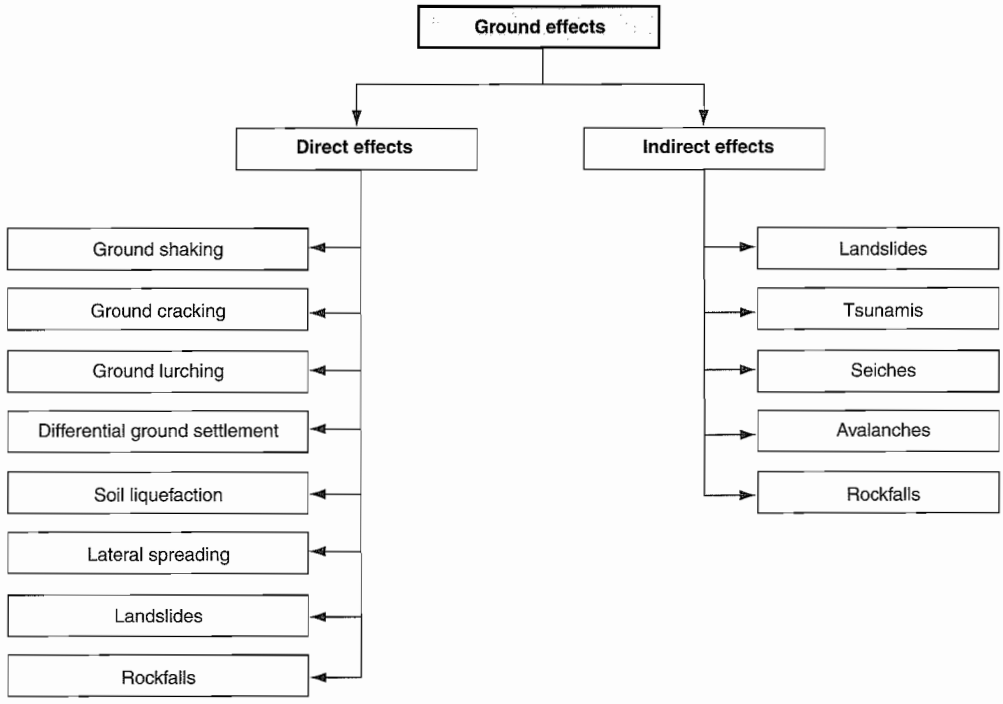



**Fig. 3.1.** Direct and indirect ground effects caused by earthquakes

densification in the event of an earthquake should be avoided by means of thorough investigation carried out in compliance with *Section 4* of EN 1998-5.

### 3.1.1. Identification of ground types

*Clause 3.1.2(1)*

The influence of the local soil condition on the seismic response of structures can be quantified by defining ground types with different mechanical properties. Five ground types have been selected to identify the soil profiles. Alphabetical capital letters (A, B, C, D and E) are used for such profiles. *Table 3.1* of EN 1998-1 ('*ground types*') provides for each ground type a description of the stratigraphic profile and the parameters used to classify the soil. Three parameters have been used in *Table 3.1* of EN 1998-1. They are ranked as follows:

- average shear wave velocity ($v_{s,30}$)
- number of blows evaluated with the standard penetration test ($N_{SPT}$)
- undrained cohesive resistance ($c_u$).

It should be noted that the site classification can be based on the values of $v_{s,30}$ where available. Otherwise, values of $N_{SPT}$ are used. Values of $N_{SPT}$ and $c_u$ are typically used in geotechnical problems to define the mechanical properties of soil. For soils A and E, values of $N_{SPT}$ and $c_u$ have not been provided in *Table 3.1* of *clause 3.1.2* of EN 1998-1. Ground type B exhibits values of $N_{SPT}$ greater than 50, and $c_u$ greater than 250 kPa, while ground type D has $N_{SPT}$ less than 15 and $c_u$ less than 70. Intermediate values of $N_{SPT}$ and $c_u$ are used to classify ground type C.

The ground characterization in *Table 3.1* of EN 1998-1 ranges from rock or other rock-like geological formations (ground type A) to surface alluvium with a thickness varying between about 5 and 20 m (ground type E). Ground type A is characterized by a shear wave velocity $v_{s,30}$ of greater than 800 m/s. For smaller values of $v_{s,30}$ the stratigraphic profile comprises very dense sand, gravel or very stiff clay (ground type B, $360 \le v_{s,30} \le 800$ m/s). Ground type C includes deep deposits of dense or medium-dense sand, gravel or stiff clay; the shear wave velocity is $180 \le v_{s,30} \le 360$ m/s. If $v_{s,30} < 180$ m/s, the deposit consists of loose-to-medium cohesionless soil or of predominantly soft-to-firm cohesive soil.

*Clauses 3.1.2(2), 3.1.2(3)*

The shear wave velocity ($v_s$) is relatively straightforward to measure, either *in situ* or in the laboratory. The shear modulus ($G$) is directly related to $v_s$ by the relationship

$$v_s = \sqrt{\frac{G}{\rho}} \tag{D3.1}$$

in which $\rho$ is the mass density of the soil, which is often easily evaluated. Therefore, reliable estimates of $G$ depend on accurate measurements of $v_s$. Equation (D3.1) provides upper bounds for shear moduli ($G_{max}$): as the strain increases, the soil stiffness decreases. The modulus reduction factor, i.e. the ratio $G/G_{max}$, depends on several environmental and loading conditions. In addition, soil profiles are frequently heterogeneous; drill boreholes may be essential to identify different strata and their thickness. The latter can also be utilized to estimate shear wave velocities (up-hole, down-hole, cross-hole, bottom-hole and in-hole). Geotechnical characterization of the heterogeneous soil profile is performed by means of the average value of the velocity $v_s$.

The average shear wave velocity $v_{s,30}$ is defined in *equation (3.1)* of *clause 3.1.1.2*, in which $h_i$ and $v_i$ are the thickness (in metres) and the shear wave velocity (evaluated at shear strain level not greater than $10^{-6}$) of the $i$th formation or layer, in a total of $N$ layers, of the top 30 m. Recent comprehensive studies have shown that, during strong shaking, soil strains can be as high as $\gamma \cong 5 \times 10^{-3}$ or even more, leading to shear modulus ratio $G/G_{max} \cong 1/10$.[5]

*Clause 3.1.2(1), 3.1.2(4)*

Two special ground types, namely $S_1$ and $S_2$, have also been listed in *Table 3.1* of EN 1998-1. The former includes deposits consisting (or containing) a layer at least 10 m thick of soft clays/silts with a high plasticity index (PI > 40) and high water content. Ground type $S_2$ accounts for all the other soil profiles, and includes deposits of liquefiable soils and sensitive clays. Soil type $S_2$ is thus likely to fail under earthquake ground motion, which may cause severe structural damage. *Clause 3.1.2(4)* requires special studies for ground type $S_2$.

Similarly, soil type $S_1$ can generally produce anomalous seismic site amplification and soil–structure interaction effects (see *Section 6* of EN 1998-5) which significantly influence the earthquake characteristics and hence seismic action at the construction site. Soil $S_1$ exhibits, in fact, very low values of shear wave velocity, low internal damping and an abnormal range of linear behaviour. Liquefaction of soils leads to catastrophic failure. On level ground, it causes loss of bearing capacity of foundation systems, and on sloping ground it gives rise to flow conditions, although on very gentle slopes lateral spreading occurs. There may be some delay for the effects of liquefaction to appear on the surface. This circumstance generally occurs when the liquefied deposits are at some depth overlain by a relatively impermeable clay layer. It takes time for the water under high pressure at that depth to flow out through the clay layer thus affecting the pore pressures in the top layer and causing damage. The upper layer will first swell and then consolidate, while the liquefied layer will consolidate. The time and the pressure gradient will depend on the relative consolidation and swelling characteristics of the two layers.[6] Detailed geotechnical studies should assess the effects of the thickness of soil layer and shear wave velocity of the soft clay/silt layer and the variation of stiffness between layers of soil $S_1$ and the underlying materials on the response spectrum.

Using only the ground types included in *Table 3.1* of EN 1998-1 to assess the stratigraphy at a given construction site may result in extreme oversimplification. Consequently, further classification of the ground conditions can be made to conform more closely to the stratigraphy of the site and its deeper geology.

## 3.2. Seismic action

### 3.2.1. Seismic zones

*Clauses 3.2.1(1), 3.2.1(2)*

This section aims to define the seismic action used to perform structural analysis and to design building systems according to the rules specified in the relevant parts of Eurocode 8. Typical representations of seismic actions are described. These include basic (**spectrum based**) and alternative (**accelerograms**) formats. Also, expressions for combining the seismic action with other actions are given. Seismic zones are introduced along with the engineering seismological parameters utilized to define the hazard within each zone.

The estimation of future earthquake ground motions at a particular location can be carried out through the assessment of seismic hazard. There are a number of ways in which the hazard can be expressed; common approaches are either deterministic or probabilistic. A fairly comprehensive treatment of this subject is provided by Reiter[7] and Lee *et al.*[8]

The seismic hazard at a site can be represented by a hazard curve showing the exceedence probabilities associated with different levels of a given engineering seismology parameter, e.g. peak ground acceleration (PGA), velocity (PGV), displacement (PGD) and duration, for a given period of exposure. Alternatively, the return period associated with different levels of the selected parameter can be used, which results from the probability of being exceeded and the period of exposure. PGAs are widely employed for hazard curves. More recently, spectral ordinates at a given response period have been used to characterize a hazard. Earthquakes cause inertial forces on structures; hence the effects can be assessed if the structural mass and the PGA are known.

The seismic hazard can also be presented in the form of regional maps. National authorities should perform seismic hazard assessments to subdivide national territories into seismic zones as a function of the local hazard. The hazard within each zone is assumed to be constant. This assumption tends to be over-conservative in the case of directivity of the fault rupture.[8,9]

Hazard maps are derived by employing attenuation relationships. These are empirical expressions describing ground motion variation with magnitude and source–site distance. Such relationships account for the mechanisms of energy loss of seismic waves during their travel through a path (**soil hysteresis** and **scattering**). They permit the estimation of both the

ground motion at a site from a specified event and the uncertainty associated with the prediction. There is a large number of attenuation equations that have been developed by various researchers.[10] Those used for hazard maps are generally based on values of PGAs. The basic functional form for attenuation relationships is as follows:

$$\log(Y) = \log(b_1) + \log[f_1(M)] + \log[f_2(R)] + \log[f_3(M, R)] + \log[f_4(E_i)] + \log(\varepsilon) \qquad \text{(D3.2)}$$

where $Y$ is the ground motion parameter to be computed, for example the PGA, PGV or PGD, and $b_1$ is a calibration factor. The second to fourth terms on the right-hand side are functions ($f_i$) of the magnitude ($M$), source-to-site distance ($R$) and possible source, site and/or geologic structure effects ($E_i$). Uncertainty and errors are quantified through the parameter $\varepsilon$. Equation (D3.2) is an additive function based on the model for the ground motion regression equations defined by Campbell.[11] It also accounts for the statistical log-normal distribution of a ground parameter ($Y$). Peak ground motion parameters decrease as the epicentral distance increases. The attenuation depends, however, on the magnitude; these variations may be expressed through equation (D3.2). Figure 3.2 shows variations of peak ground horizontal acceleration with magnitude and the effects of focal depth. Revised attenuation relationships for European countries and some regions in the Middle East have been proposed by Ambraseys and co-workers[12,13] for different peak ground motion parameters.

In probabilistic seismic hazard assessment and for hazard maps, earthquakes are modelled as a Poisson process. The Poisson model is a continuous time, integer-value counting process with stationary independent increments.[14] This means that the number of events occurring in an interval of time depends only on the length of the interval and does not change in time (stationarity). The probability of an event occurring in the interval is independent of the history and does not vary with the site. Thus, each earthquake occurs independently of any other seismic event; that is, earthquakes have no memory. The Poisson model is defined by a single parameter ($\nu$) which expresses the mean rate of occurrence of seismic events exceeding a certain threshold, e.g. earthquakes of magnitude greater than $M$ over a given area.

The probability of earthquake occurrence modelled by the Poisson distribution is as follows:

$$P[N = n,\ T_L] = \frac{(\nu T_L)^n e^{-\nu T_L}}{n!} \qquad \text{(D3.3)}$$

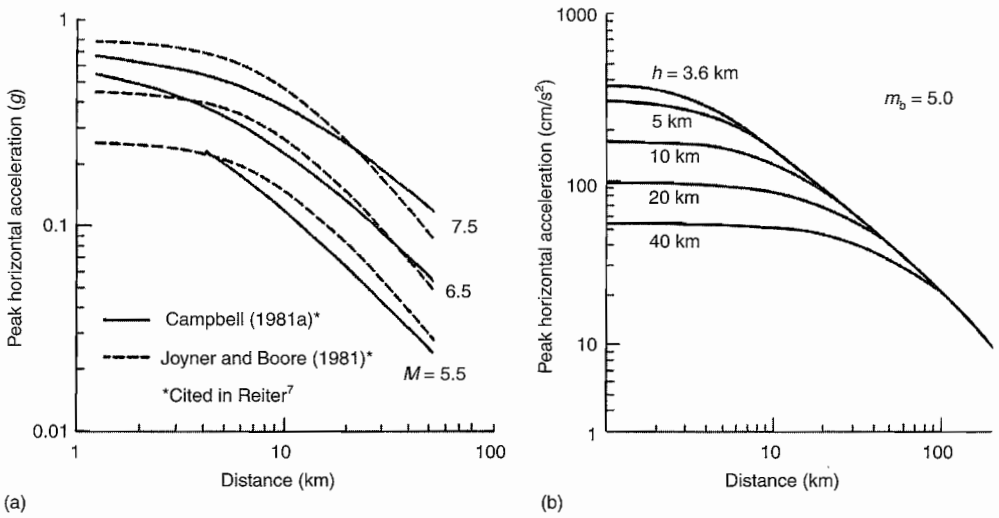



**Fig. 3.2.** Attenuation of peak ground horizontal acceleration: (a) effect of magnitude and (b) focal depth[7]

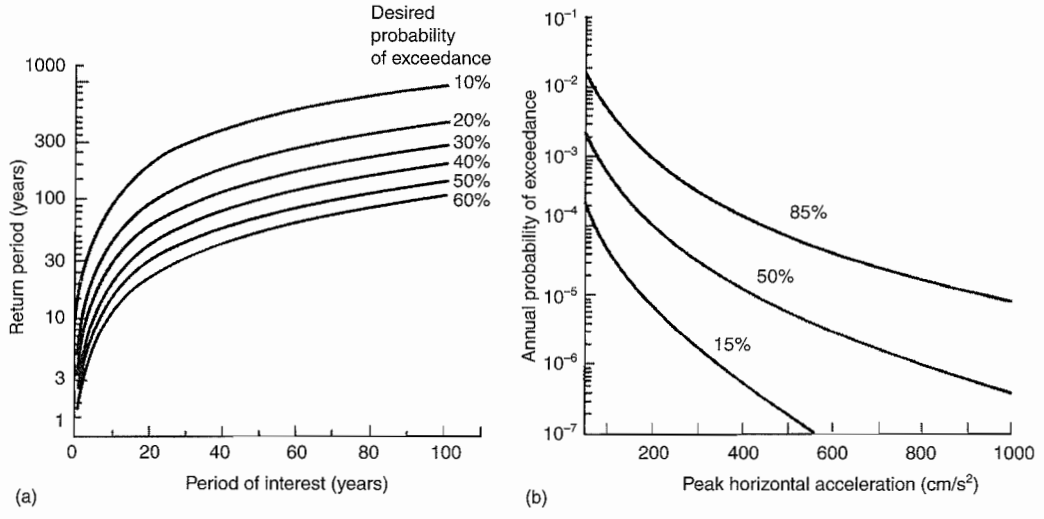



**Fig. 3.3.** Relationship between (a) return period, lifetime of the structure and desired probability of exceedance and (b) hazard curves for peak ground accelerations[7]

where $P = P[N = n, T_L]$ is the probability of having $n$ earthquakes with magnitude $m$ greater than $M$ over a reference time period $T_L$ in a given area. The value of $\nu$ corresponds to the expected number $N$ of occurrences per unit time for that area, i.e. the cumulative number of earthquakes greater than $M$.

Recurrence relationships express the likelihood of earthquakes of a given size occurring in the given source during a specified period of time, for example one year. Therefore, the expected number of earthquakes $N$ in equation (D3.3) can be estimated through statistical recurrence formulae. Gutenberg and Richter[15] developed the following frequency–magnitude relationship:

$$\log \nu = a - bM \tag{D3.4}$$

in which $a$ and $b$ are model constants that can be evaluated from seismological observational data through a least-square fit. They describe the seismicity of the area and the relative frequency of earthquakes of different magnitudes, respectively.

From equation (D3.3), the probability of at least a seismic event exceeding a certain threshold can be expressed as the complement of no such occurrence (i.e. $n = 0$):

$$P[m > M,\ T_L] = 1 - e^{-\nu T_L} \tag{D3.5}$$

The return period, $T_R$, of seismic events that exceed a certain threshold can be estimated as the average time between such occurrences: *Clause 3.2.1(3)*

$$T_R = 1/\nu = -T_L/\ln(1 - P) \tag{D3.6}$$

Low-magnitude earthquakes occur more often than high-magnitude events and are generally expected to produce less damage. Longer return periods lead to a lower probability of earthquake occurrence, which is often associated with a higher potential for economic loss (owing to a lack of seismic design provisions). The relationship between the return period $T_R$, the lifetime of the structure, $T_L$, and the probability of exceedance of earthquakes with a magnitude $m$ greater than $M$, P[$m > M$, $T_L$], is plotted in Fig. 3.3a. Variations of the peak horizontal acceleration with the annual probability of exceedance are also included for the three percentiles 15, 50 and 85 in Fig. 3.3b.

The design seismic action on rock (or, in Eurocode 8 terms, on type A ground) for structures of ordinary importance is the 'reference' seismic action. In EN 1998-1 the hazard is defined through the value of the *'reference peak ground acceleration'* on type A ground, denoted by $a_{gR}$. This parameter will be derived from zonation maps in the National Annexes.

Reference peak ground accelerations chosen by national authorities correspond to the reference return period ($T_{NCR}$) of the design seismic action (for the no-collapse requirement) of structures of ordinary importance. For such structures it also corresponds to the reference probability of exceedance in $T_L = 50$ years ($P_{NCR}$). As explained in Section 2.1 of this guide, the reference values (period and probability of exceedance) selected by national authorities apply to the design of ordinary structures to fulfil the no-(local-)collapse requirement. Structures other than ordinary ones are designed to fulfil the no-(local-) collapse requirement under a design ground acceleration equal to

$$a_g = \gamma_I a_{gR} \tag{D3.7}$$

The value of the importance factor $\gamma_I$ in equation (D3.7) is by definition equal to 1.0 for structures of ordinary importance. Its recommended values for buildings other than ordinary ones have been given in Section 2.1. Values of $\gamma_I$ other than 1.0 are considered to correspond to mean return periods other than the reference, $T_{NCR}$. It is noted, though, that ground motions with different $T_R$ values generally exhibit different seismological characteristics, especially frequency content and duration, in addition to a different PGA as per equation (D3.7).

### 3.2.2. Basic representation of the seismic action

#### *3.2.2.1. General*

*Clause 3.2.2.1*

Methods for evaluating earthquake input for different hazard levels include zonation map-based procedures and site-specific studies. The latter are primarily employed for large projects, such as long bridges, nuclear power plants and/or when site amplification effects, e.g. on soft soil sites, are expected. It is also the only approach feasible in the assessment of geographically distributed systems subjected to spatially varying ground motion. Soft soil sites filter out short periods and amplify longer periods, and may cause large amplification of response quantities. Map-based procedures, such as those normally provided by national authorities in Europe, use maps of the PGA to define the seismic input at one or more different hazard levels and under different site conditions. Consequently, the earthquake ground motion at a given site is described by the response spectrum, which may be elastic (theoretical response of a single degree of freedom (SDOF) system in the elastic range), inelastic (theoretical response of an SDOF system with inelastic load– deformation characteristics), or design (smoothed and adjusted spectrum taking into account non-theoretical features and requirements for safe design, e.g. providing a minimum base shear for long-period structures).

A response spectrum is a plot of the maxima of the acceleration ($a$), velocity ($v$) or displacement ($d$) response for an SDOF system with various natural periods when subjected to an earthquake ground motion. A family of curves is usually calculated for any given excitation, showing the effect of variation of the structural damping. The latter is expressed in terms of the equivalent linear viscous damping ratio ($\xi$). For many practical structural applications it can be time saving to employ the maxima of the above response parameters ($a$, $v$ and/or $d$) rather than their values at each instant during the response; these maxima are 'spectral values'. Response spectra can be computed from earthquake accelerograms, either natural or artificial, by means of several computer programs. Freeware versions of software such as SEISMOSIGNAL and USEE are available on the Internet at http://www.seismolinks.com and http://mae.ce.uiuc.edu/usee, respectively. These programs implement numerical algorithms to integrate the equation of motion for an SDOF system for given constitutive action–deformation relationships and damping ratios $\xi$. Time steps and parameters of the integration schemes, for example the Newmark and/or Wilson method, can be specified for the earthquake record being considered.

The elastic response spectra can be derived analytically by means of Duhamel's integral, which provides the total displacement response of an SDOF system subjected to earthquake loading. The maximum value of the displacement $S_d$ (**spectral displacement**), can be derived by using principles of structural dynamics. Displacement response spectra are essential for

modern seismic design approaches, such as displacement-based design.[16] Considerable analytical efforts have been undertaken (e.g. see Bommer and Elnashai[17], Tolis and Faccioli[18] and Borzi *et al.*[19]). Such spectra are discussed in informative *Annex A* of EN 1998-1.

The maximum velocity ($S_v$) and acceleration ($S_a$) can be derived directly from $S_d$. The values of $S_v$ and $S_a$ are spectral pseudo-velocity and pseudo-acceleration, respectively. The prefix 'pseudo' indicates that such values do not correspond to the actual peak spectral velocity and acceleration. The real values would be calculated by differentiation, while pseudo-values are obtained by assuming simple harmonic oscillation. For the practical range of damping in earthquake engineering ($0.5 \leq \xi \leq 10\%$) and for low-to-medium period structures ($0.2 \leq T \leq 1.0$ s), the pseudo-velocity spectrum is a close approximation of the true relative velocity spectrum. However, for structures with supplemental damping ($\xi > 15$–$20\%$), e.g. with passive, active and/or semi-active vibration control devices, the differences between maximum absolute acceleration and $S_a$ increase as a function of the natural period $T$.[20]

Acceleration response spectra are related directly to the base shear used in the seismic design, and hence they are implemented in force-based codes, such as Eurocode 8. Response acceleration spectra can be computed for the three translational components of earthquake ground motions, e.g. horizontal (longitudinal and transversal) and vertical. Horizontal and vertical spectra are influenced by different frequency contents and ground accelerations. Their shapes and values are discussed in *clauses 3.2.2.2* and *3.2.2.3*, respectively. For a given component of ground motion, the response spectra depend significantly on the relative distance between the seismic source and the observation site. For example, Fig. 3.4 shows the response spectra for the 1940 El Centro and the 1994 Northridge earthquakes, which are representative of strong motions registered close to and far from the seismic source, respectively. Differences in shape between far- and near-source response spectra are due to the frequency content of the input motion. The former are generally broad-band signals, while the latter are narrow-band, pulse-like records. For distant earthquakes, the rupture can be assumed to be uniform and instantaneous; the ground motion at the site is influenced to a lesser extend by the source seismological characteristics. This assumption cannot be safely made for near-source earthquake ground motion; site-specific studies are warranted. Vertical components of ground motion show typical features of near-fault records, as discussed further in *clause 3.2.2.3*.

Elastic spectra are useful tools for structural design and assessment. They do not, however, account for the inelasticity and stiffness and strength degradation experienced during severe earthquakes. Structural systems are not designed to resist earthquake forces in their elastic range, excepting a very few cases of safety-critical installations (e.g. nuclear power plants, which are not covered in Eurocode 8). Concepts of energy absorption and plastic redistribution are used to reduce the elastic seismic forces by as much as 80%. As already described in Section 2.2.2.1, the inelastic behaviour of structures is quantified through the behaviour factor ($q$), provided in the relevant parts of Eurocode 8. High $q$ values correspond to large inelastic deformations; for linearly elastic systems, the behaviour factor is unity. Thus, inelastic spectra for a target level of inelasticity were estimated by dividing the ordinates of the elastic spectra by the $q$ factors (e.g. see Newmark and Hall,[21] Borzi and Elnashai,[22] and many others), as discussed in *clause 3.2.2.5*.

The reduction of the elastic spectra by the use of $q$ factors given in *clause 3.2.2.5* of EN 1998-1 is the most commonly used approach to derive inelastic spectra. The approach is employed in Eurocode 8 to evaluate design base shears. If other design requirements are included (e.g. minimum base shear, to safeguard against force increase as the structure yields) the inelastic spectrum becomes a design spectrum. However, this approach makes use of static concepts to scale the elastic spectrum, obtained from dynamic analysis. It is, as such, insensitive to characteristics of the earthquake motion which affect the hysteretic damping. More accurate results can be obtained by non-linear dynamic analysis of SDOF systems subjected to earthquake input.[11,23,24]

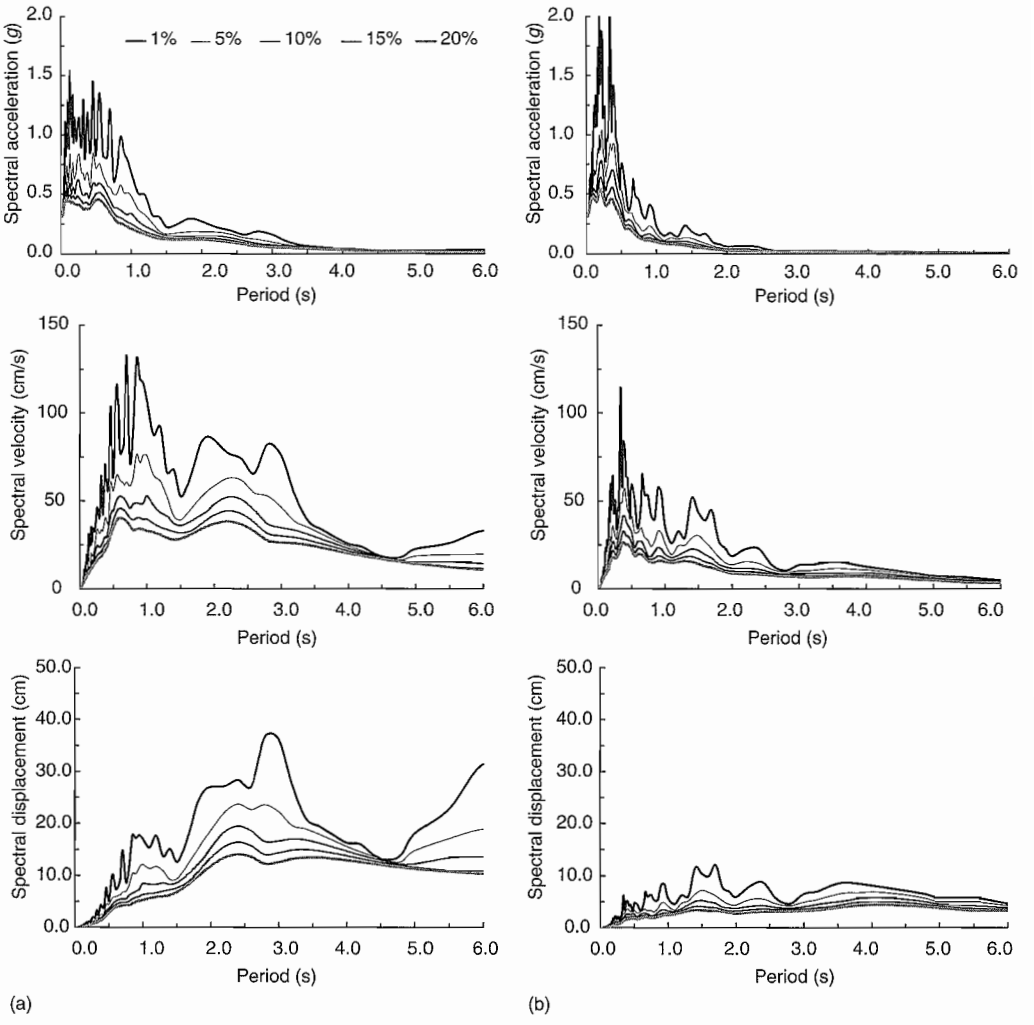



**Fig. 3.4.** Elastic response spectra for (a) the 1940 ElCentro and (b) the 1994 Northridge earthquakes for various damping (1, 5, 10, 15 and 20%): acceleration (top), velocity (middle) and displacement (bottom)

Generally, elastic spectra are derived from a specific ground motion, and hence they have the physical meaning of the maximum response of the SDOF system under earthquake ground motion. They possess irregular shapes due to peaks and valleys, and are not suitable for design because of the difficulties encountered in determining the exact frequencies and modal shapes during severe earthquakes when the dynamic response is likely to be highly non-linear.[25] Conversely, design spectra are more appropriate for design purposes. They are derived from statistical analyses using either the mean, median (50th percentile probability level), or the median plus one standard deviation (84th percentile probability level) of the ground motion parameters for the records chosen. Such spectra are often modified by engineering judgement. Their ordinates may, however, not have any physical meaning. Design response spectra can be defined for each component of the earthquake motion: longitudinal, transverse and vertical.

In EN 1998-1, elastic response spectra are presented as smooth curves and/or straight lines; therefore, they correspond to elastic design spectra.

Spectra with similar shapes are used for the limit states of damage limitation (serviceability limit state) and collapse prevention (ultimate limit state). It is thus assumed that moderate-

to-intense earthquakes possess the same frequency content. Note that *clause 3.2.2.1(4)* specifies that for the three components of the seismic action, one or more alternative shapes of response spectra can be adopted, depending on the seismic source and the earthquake magnitude.

### 3.2.2.2. *Horizontal elastic response spectrum*

*Clauses 3.2.2.2(1), 3.2.2.2(2)*

Horizontal (longitudinal and transverse) components of ground motion are mainly caused by secondary shear S waves. The wavelength of these seismic waves is longer than that of primary P waves, which means that the former are associated with lower frequencies (higher periods).

Horizontal components of the seismic action are defined in Eurocode 8 through the horizontal elastic response spectrum given by *equations (3.2)–(3.5)* in EN 1998-1, where $S_e(T)$ is the value of the elastic response spectrum for the vibration period $T$ of a linear SDOF system. The design acceleration ($a_g$) in the above equations is that corresponding to ground type A according to equation (D3.7) above. The soil effects are, in fact, accounted for through the soil factor $S$. The latter is by definition equal to 1.0 for ground type A. Recommended values of $S$ for the other standard ground types are provided in *Table 3.2* of EN 1998-1, along with the corner periods ($T_B$, $T_C$ and $T_D$). The period $T_D$ defines the beginning of the constant displacement response range of the spectrum; this period does not change significantly with the ground type.

To avoid overestimation of spectral ordinates in those areas of Europe affected only by moderate-magnitude earthquakes whilst still only mapping a single ground-motion parameter, e.g. PGA, two types of spectra are recommended in Eurocode 8 for the horizontal elastic response spectrum: type 1 and type 2 (see Figs 3.6, 9.3, 9.5 and 9.6 of this guide). These spectra are classified as a function of the magnitude of the earthquakes that contribute most to the seismicity at the given site, for the purpose of the probabilistic seismic hazard. If the earthquake that contributes most possesses a surface wave magnitude ($M_S$) not greater than 5.5, the type 2 spectrum may be adopted (**moderate-seismicity context**). Alternatively, the type 1 spectrum should be used (**high-seismicity context**). It is instructive to note that for earthquakes characterized by $M_S > 5.5$ (type 1) the maximum spectral amplifications occur at frequencies lower than the corresponding of the spectra of type 2.

*Clauses 3.2.2.2(3), 3.2.2.2(4)*

Horizontal elastic response spectra derived from *equations (3.2)* to *(3.5)* in EN 1998-1 are generally provided for values of damping ($\xi$) equal to 5%. A correction factor $\eta$ is also utilized in *equations (3.2)–(3.5)*; its expression is given in *equation (3.6)* in EN 1998-1. A minimum threshold value of the damping correction factor is also provided ($\eta = 0.55$). Such a value corresponds to a maximum equivalent viscous damping of about 30% ($\xi = 28\%$). Note that for $\xi = 5\%$ the factor $\eta = 1.0$. Generally, it is not straightforward to quantify the total damping associated with the different mechanisms in structural systems, as shown in Fig. 3.5. Therefore, an equivalent viscous damping ratio $\xi$ is employed. Indicative values of $\xi$ from the literature are provided in Table 3.1, as a function of the material of construction and of the limit state to be assessed.

The spectra provided in Fig. 3.6 are for 5% damping and normalized by $a_g$. If values of viscous damping different from 5% are deemed necessary, these values are then specified in the relevant parts of Eurocode 8.

*Clauses 3.2.2.2(5), 3.2.2.2(6)*

*Equations (3.2)–(3.5)* in EN 1998-1 can also be employed to derive the horizontal elastic displacement response spectrum in a simplified fashion. Acceleration spectral values may be converted in displacement values through *equation (3.7)* in EN 1998-1. That relationship normally holds for periods of vibration ($T$) not greater than 4.0 s. For very flexible structures, i.e. structures with $T > 4.0$ s, a more refined definition of elastic displacement response spectrum should be sought. For the type 1 spectrum, Eurocode 8 provides such a definition in informative *Annex A* of EN 1998-1. For periods $T > 4.0$ s, the elastic acceleration spectrum can thus be derived from the elastic displacement spectrum by inverting *equation (3.7)*.

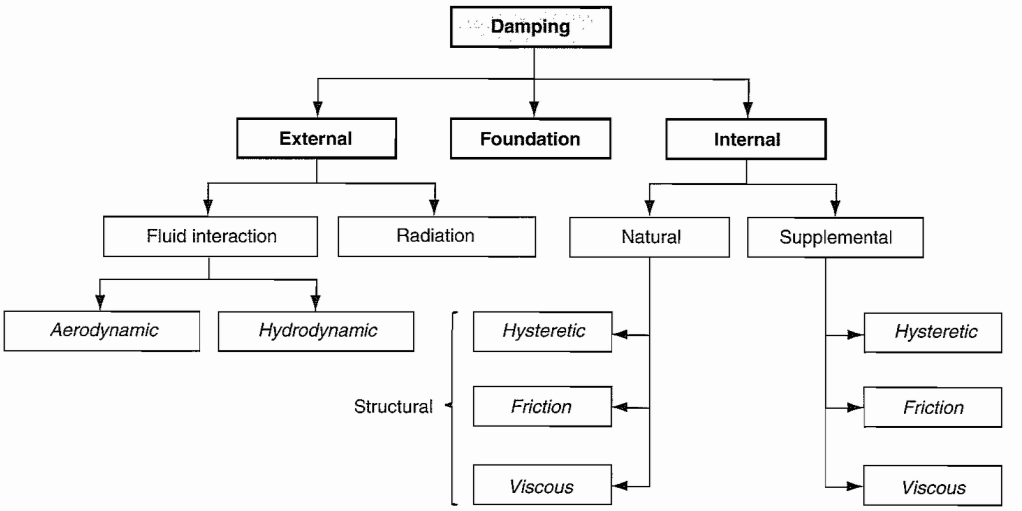



**Fig. 3.5.** Sources of damping mechanisms in structural systems

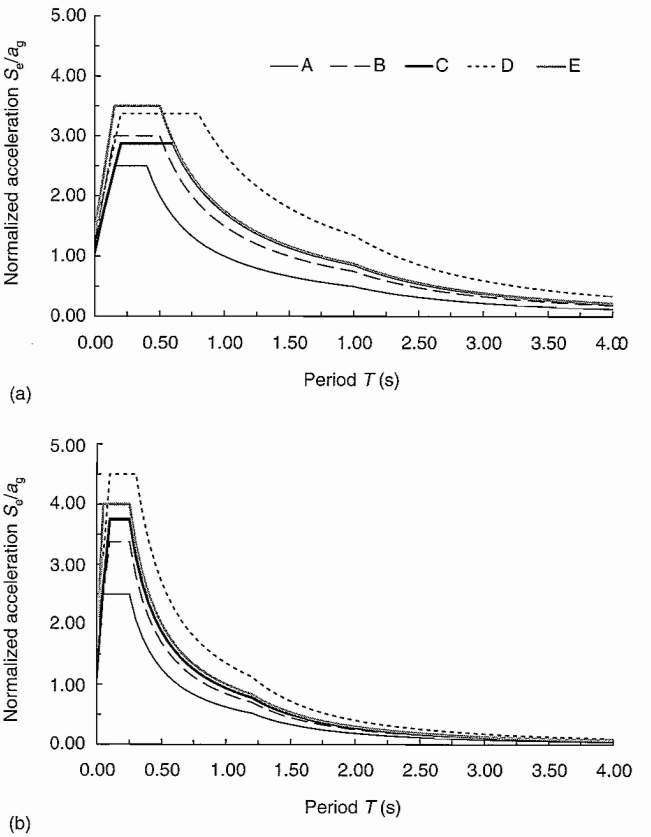



**Fig. 3.6.** Elastic response spectra for different soil conditions: (a) type 1 and (b) type 2

### *3.2.2.3. Vertical elastic response spectrum*

*Clause 3.2.2.3(1)* The vertical component of earthquake ground motion has generally been neglected in the earthquake-resistant design of structures.[27] This is gradually changing due to the increase in near-source records obtained recently, coupled with field observations confirming the potential damaging effects of high vertical vibrations. The vertical component of ground motion is mainly associated with the arrival of vertically propagating compressive P waves,

**Table 3.1.** Values of viscous damping for different construction materials[26]

| Material | Damping (%) |
|---|---|
| Reinforced concrete | |
| Small amplitudes (uncracked) | 0.7–1.0 |
| Medium amplitudes (fully cracked) | 1.0–4.0 |
| Large amplitudes (fully cracked) but no yielding of reinforcement | 5.0–8.0 |
| Prestressed concrete (uncracked) | 0.4–0.7 |
| Lightly stressed concrete (slightly cracked) | 0.8–1.2 |
| Composite | 0.2–0.3 |
| Steel | 0.1–0.2 |

whilst secondary shear S waves are the main cause of horizontal components. The wavelength of P waves is shorter than that of S waves, which means that the former are associated with higher frequencies.

The vertical component of the seismic action can be defined through the vertical elastic response spectrum. The commonly used approach of taking the vertical spectrum as two-thirds of the horizontal, without a change in frequency content, has been superseded.[28] The vertical elastic spectrum is provided in *equations* (*3.8*)–(*3.11*) of EN 1998-1. Three corner periods, i.e. $T_B$, $T_C$ and $T_D$, as well as the vertical component of ground acceleration $a_{vg}$ should be provided by national authorities. Nevertheless, as for the horizontal elastic response spectrum (see Section 3.2.2.2), two types of vertical spectral shapes are recommended: type 1 and type 2. The latter should be employed when the earthquake that contributes most to the seismic hazard for the site has magnitude $M_S < 5.5$. The values of $T_B$, $T_C$ and $T_D$ recommended by Eurocode 8 are provided in *Table 3.4* of EN 1998-1, and are the same for type 1 and type 2 spectra. The values of the control periods for vertical spectra are smaller than those for horizontal spectra; the former are, in fact, characterized by a higher frequency content than that of horizontal components. In addition, for intense earthquakes, e.g. those with $M_S > 5.5$, the vertical component of the peak ground acceleration ($a_{vg}$) can be as high as the horizontal counterpart ($a_g$): $a_{vg} = 0.9a_g$. In turn, for earthquakes with $M_S < 5.5$, the recommended value of the ratio $a_{vg}/a_g$ is equal to 0.45.

Vertical spectra recommended in Eurocode 8 do not vary with the soil conditions (see also the values in *Table 3.4* of EN 1998-1). This is due to the lack of data on which soil effects on vertical spectra can be based. Recommended corner periods $T_B$ and $T_C$ are fixed to 0.05 and 0.15 s, respectively, according to Elnashai and Papazoglou.[28]

### 3.2.2.4. Design ground displacement

Ground displacements are generally difficult to estimate accurately because of errors in signal processing in the filtering and integration of analogue records.[11] Special studies may be performed to evaluate the design ground displacement ($d_g$) for a given construction site. Attenuation relationships for ground displacements and design spectra for European regions were derived by Bommer and Elnashai[17] and Tolis and Faccioli.[18] Alternatively, a simplified relationship to compute $d_g$ is provided in *equation* (*3.12*) in EN 1998-1, in terms of the design ground acceleration, $a_g$, the soil factor, $S$, and the corner periods of the response spectrum, $T_C$ and $T_D$, as defined in *clause 3.2.2.2*.

*Clause 3.2.2.4(1)*

### 3.2.2.5. Design spectrum for elastic analysis

Response spectra scaled down by behaviour factors ($q$) are provided in *equations* (*3.13*)–(*3.16*) in EN 1998-1, for both horizontal and vertical components of the ground motion. By calculating actions from the scaled spectra, inelastic analyses are avoided and the energy dissipation capacity of structural systems is accounted for in a simple manner, by performing an elastic analysis based on design spectrum. Design spectra reduced by $q > 1$ can be utilized to evaluate design forces for structural systems responding inelastically under earthquake loading.

*Clauses 3.2.2.5(1), 3.2.2.5(2), 3.2.2.5(3), 3.2.2.5(4), 3.2.2.5(5)*

In the relationships of *clause 3.2.2.5*, $S_d(T)$ is the design spectrum, $q$ is the behaviour factor and $\beta$ is the lower bound factor for the horizontal design spectrum. The last is provided by national authorities; the recommended value proposed in EN 1998-1 is 0.2. The value of the accelerations $a_g$ and $a_{vg}$ of the soil factor $S$ and of the corner periods $T_C$ and $T_D$ are those specified in *clauses 3.2.2.2* and *3.2.2.3* for the horizontal and vertical components of ground motions, respectively.

Upper bounds for the $q$ factors, which also account for the influence of viscous damping ($\xi$) other than 5%, and are to be used for horizontal and vertical components of ground motions, are given for the various materials and structural systems and according to the relevant class of ductility in the relevant parts of Eurocode 8.

*Clauses 3.2.2.5(6), 3.2.2.5(7)*

For the vertical components of the seismic action, the recommended maximum value of the $q$ factor is 1.5 for all materials and structural systems. It is important to note that energy absorption and plastic redistribution for the vertical inelastic response is inherently lower than in the case of the transverse response.[28,29] However, the adoption of $q$ values greater than 1.5 in the vertical direction is permitted by *clause 3.2.2.5(7)* in EN 1998-1, provided that the energy absorption mechanisms are identified and their absorption capacity is quantified by advanced analysis (possibly including testing).

*Clause 3.2.2.5(8)*

The spectra given by *equations (3.13)–(3.16)* are not appropriate for the design of structures with base isolation and/or energy dissipation devices. In these cases, special studies are required to derive the spectra to employ in the structural analysis and design.

### 3.2.3. Alternative representations of the seismic action

*3.2.3.1. Time–history representation*

*Clauses 3.2.3.1.1(1), 3.2.3.1.1(3)*

There are three approaches to obtain earthquakes or earthquake-like ground motion records (acceleration versus time functions) for the purposes of assessment by advanced analysis. Natural records of earthquakes have increased exponentially in the past decade or so, leading to the availability of high-quality strong-motion (acceleration) data for different locations around the world, archived by many agencies. Another approach is to generate a signal that fits, with a certain degree of approximation, a target spectrum. Finally, use of mathematical source models (point, line or area source representations) to generate strong-motion-like time series is increasing in popularity since the ensuing records resemble natural records more than records generated to fit a target spectrum.

Records of a quantity versus time as described above may also be expressed as ground velocity or displacement.

*Clause 3.2.3.1.1(2)*

For planar structural models, horizontal and vertical components of earthquake motions can be considered to act simultaneously. For spatial models, the seismic action should consist of three simultaneously acting accelerograms, two horizontal and one vertical. It is recommended that the most demanding combination of motions applied in all the different directions is used.

**Artificial accelerograms**

*Clauses 3.2.3.1.2(1), 3.2.3.1.2(2), 3.2.3.1.2(3), 3.2.3.1.2(4)*

Artificial accelerograms are an option for generating signals that satisfy engineering criteria unrelated to the physics of earthquake motion generation and propagation. Accelerograms can be mathematically simulated through random vibration theory. Both stationary and non-stationary random processes have been suggested.[30,31] Strong motions include transitional phases at the initial and final stages, respectively, moving from rest to maximum shaking and vice versa (**non-stationary processes**). Small earthquakes can also be described by such processes. By contrast, the middle portion, i.e. the nearly uniform part of the vibration, can be modelled by means of stationary processes, such as white noise.[32,33]

The most widely used approach is to develop a signal with a response spectrum that matches a target response spectrum with a predefined accuracy (e.g. 3–5% margin of error). The target spectrum is either a uniform hazard spectrum or a code spectrum. An example of such acceleration signals (a response-spectrum-compatible accelerogram) is shown in Fig. 3.7. It is noteworthy that the level of accuracy of the match depends on the number of iterations carried out during the generation process.

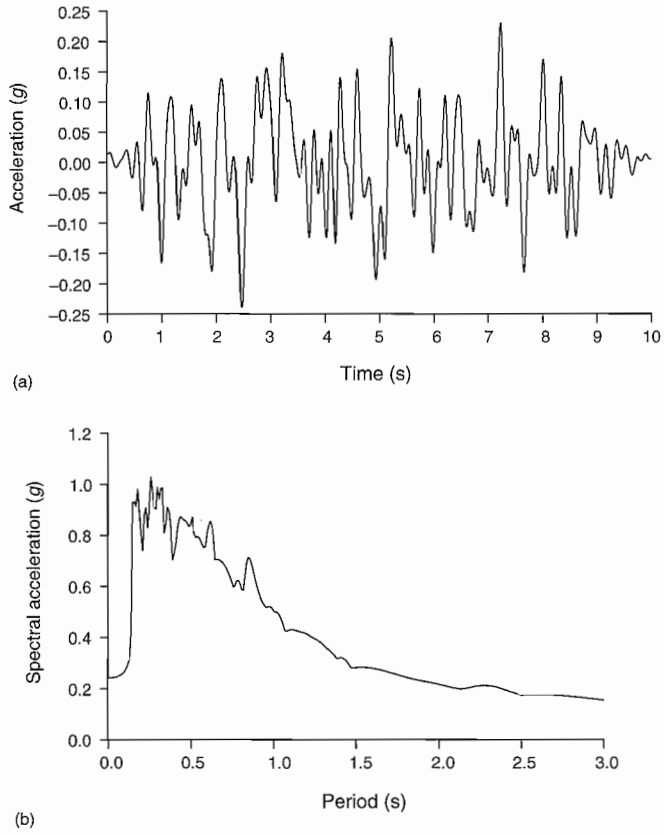



**Fig. 3.7.** (a) Acceleration artificial record matched to (b) a code spectrum

In EN 1998-1 it is required that artificial accelerograms should be generated so as to match the elastic response spectrum given in *clauses 3.2.2.2* and *3.2.2.3* for 5% viscous damping. Moreover, the duration of the generated records should be consistent with the magnitude and the other relevant seismological features consistent with the establishment of the ground acceleration ($a_g$), e.g. frequency content and duration. The stationary part of the accelerograms should possess a minimum duration of 10 s. A minimum number of three accelerograms should be generated; their mean PGA should not be less than the value $a_g S$ for a given site. In addition, mean 5% damping elastic spectral values should not be less than 90% of the corresponding value of the 5% damping elastic response spectrum. These requirements are meant to ensure deriving records that would provide conservative estimates of the response of structures and foundations. Relationships between magnitude and duration from the literature (e.g. Naeim,[25] among others) may be consulted.

Three elements are necessary to generate artificial (or synthetic) accelerograms:

- power spectral density
- a random phase angle generator
- an envelope function.

The simulated motion can be calculated as the sum of several harmonic excitations. Thus, the consistency of the artificial motion is assessed through an iterative algorithm which examines the frequency content. The latter check can be carried out either with the response spectrum of the signal or its power spectral density. A detailed description of the procedures for generating artificial records can be found in Clough and Penzien.[34] Several computer programs that generate such records have been developed (e.g. SIMQKE-1[35]). Inherent difficulties in the generation process are: (1) the assumption of the phase distribution between the various single-frequency waves and (2) the duration of the record. Therefore,

signals that match the same spectrum may look different and, more importantly, may lead to different structural response quantities. A closer fit between the spectrum of the generated signal and that of the target spectrum should be sought in the vicinity of the structural fundamental period (0.2–2.0 times the fundamental period). It should also be recognized that artificial records often exhibit a larger number of cycles than natural records. As a consequence, such records may lead to over-conservative demand quantities for inelastic systems.

### *3.2.3.2. Recorded or simulated accelerograms*

*Clauses 3.2.3.1.3(1), 3.2.3.1.3(2), 3.2.3.1.3(3)*

When using natural earthquake records, EN 1998-1 recommends the use of a minimum of three different accelerometer recordings, scaled to the required PGA. Otherwise, artificially generated records can be used, provided the distribution of frequencies associated with high energy is relevant to the fundamental period of the structure. This can be ensured by generating a record which conforms to an approved spectral shape.

It is instructive to note that the features of strong motions that affect structural response are many and their inter-relationship is complex. It is thus of importance to highlight the regional differences in strong-motion data and the criteria for the selection and scaling of natural records.

The ideal procedure for the selection of strong motion for use in analysis is to obtain records generated under conditions that are identical to those of the design earthquake scenario. Bolt[36] showed that if all the characteristics of the design earthquake could be matched to those of a previous earthquake, the probability of the characteristics of the record matching would be 100%. The design earthquake, however, is usually defined in terms of only a few parameters. Hence, it is difficult to guarantee that the selected records will closely model all of the characteristics of the design earthquake at the source, along the path and through the site to the surface. Furthermore, even if the design earthquake scenario was defined in all aspects, it is unlikely that a record could be found in the available data banks which would also match all of the characteristics. To select records with a reasonable probability of bracketing the response, it is necessary to identify the most important parameters that characterize the conditions under which an earthquake record is produced, and match as many of these as possible to the design earthquake scenario. It is emphasized that records giving seemingly consistent response parameters, i.e. with the lowest coefficients of variation, may yield much higher variations due to period shifts due to inelasticity.

The parameters that characterize the conditions under which strong-motion records are generated can be grouped into three sets representing the earthquake source, the path from the source to the bedrock under the recording site and the nature of the site. The important parameters in the above sets are as follows:

- **Source**: magnitude, rupture mechanism, directivity and focal depth
- **Path**: distance and azimuth
- **Site**: surface geology, topography and structures.

The above list is not exhaustive, but it does include the parameters that have been established as having a notable influence on ground motion characteristics.[7] These parameters influence different characteristics of the recorded motion in different ways and to different degrees. Hence, the most appropriate selection of parameters depends on which characteristics of the selected motion are considered most important from a system- response viewpoint.

### *3.2.3.3. Spatial model of the seismic action*

*Clauses 3.2.2.1(8), 3.2.3.2(1), 3.2.3.2(2)*

Structural systems frequently do not experience the same displacements at all ground–system contact points. This is referred to as asynchronous motion. Asynchronous motion is caused by the spatial variability of ground motions. The latter can be represented primarily by three mechanisms, namely:

- wave passage
- loss of coherence
- local site conditions.

The wave passage effect is due to the time delay of waves because of the finite velocity of seismic waves, while loss of coherence is caused by the reflection and refraction of waves in soil layers underlying the structure. The local properties of soil at the construction site may filter seismic ground motion, thus amplifying or damping wave amplitudes and modifying the frequency content.

If the plan dimensions of a structure are large with respect to the wavelength of the seismic waves, the foundation–structure system is subjected to non-uniform shaking. Asynchronous motion at the support points is a common problem in the design of long-span bridges, large dams and pipelines that extend over considerable areas. Conversely, for ordinary structures, the ground motion at the base can be considered coherent, provided that the structure is supported by rigid foundation systems. For structures in which the excitation at the support points is incoherent, spatial models of the seismic action should be used. Such models should employ the elastic spectra defined in *clauses 3.2.2.2* and *3.2.2.3*.

## 3.3. Displacement response spectra

This section relates to informative *Annex A* in EN 1998-1 on displacement spectra.

Recent analytical studies have dealt specifically with displacement spectra, e.g. in Europe.[17,18] Such studies were prompted by the development of seismic displacement-based assessment and design procedures.[16] The significance of explicitly deriving displacement spectra is that the derivation of displacements from velocity or acceleration using simple harmonic motion conversions may be inaccurate (as demonstrated in Fig. 3.8); the error is often small, but increases with period and for high levels of ground parameters on soft sites. A limitation on the use of current attenuation relationships for elastic displacement spectra is the fact that the majority of the available equations only provide spectral ordinates for 5% damping, with the exception of two studies[37,38] which predict spectral ordinates for damping ratios of 2, 5, 10 and 20%. However, these equations only predict spectral ordinates at response periods of up to 2.0 s. Since ductility-equivalent damping, employed in direct displacement-based design, may be up to 30% for fixed-base structures, dedicated displacement spectra for such applications are required.[18]

In view of the fact that the study reported by Bommer and Elnashai[17] was concerned with the long-period response spectrum and that small-magnitude earthquakes do not produce significant long-period radiation, it was decided to impose a lower magnitude limit on the data set presented by Ambraseys *et al.*[12] The final data set consisted of 183 accelerograms from 43 shallow earthquakes. For three of the recording stations, each of which contributed only one record, the site classification is unknown. For the remaining 180 accelerograms, the distribution amongst the three site classifications (rock, stiff soil and soft soil) as percentages is 25:51:24, which compares favourably to distribution of the original data set in Ambraseys *et al.*,[12] which is 26:54:20. Regression analyses were performed on the horizontal displacement spectral ordinates for damping ratios of 5, 10, 15, 20, 25 and 30% of critical damping. The regression model used for spectral $S_D$ ordinates (centimetres) was the same as that employed in Ambraseys *et al.*[12] for acceleration spectral ordinates. At each period, the larger spectral ordinate from the two horizontal components of each accelerogram was used as the dependent variable.

From inspection of a large number of displacement response spectra for different levels of damping $\xi$, it was concluded that a general, idealized smoothed format would be as shown in Fig. 3.9.

The smoothed spectrum for each damping level comprises an initial curve and four straight line segments. These branches are defined by four control periods along with their corresponding amplitudes. The amplitude corresponding to $T_F$ is the peak ground

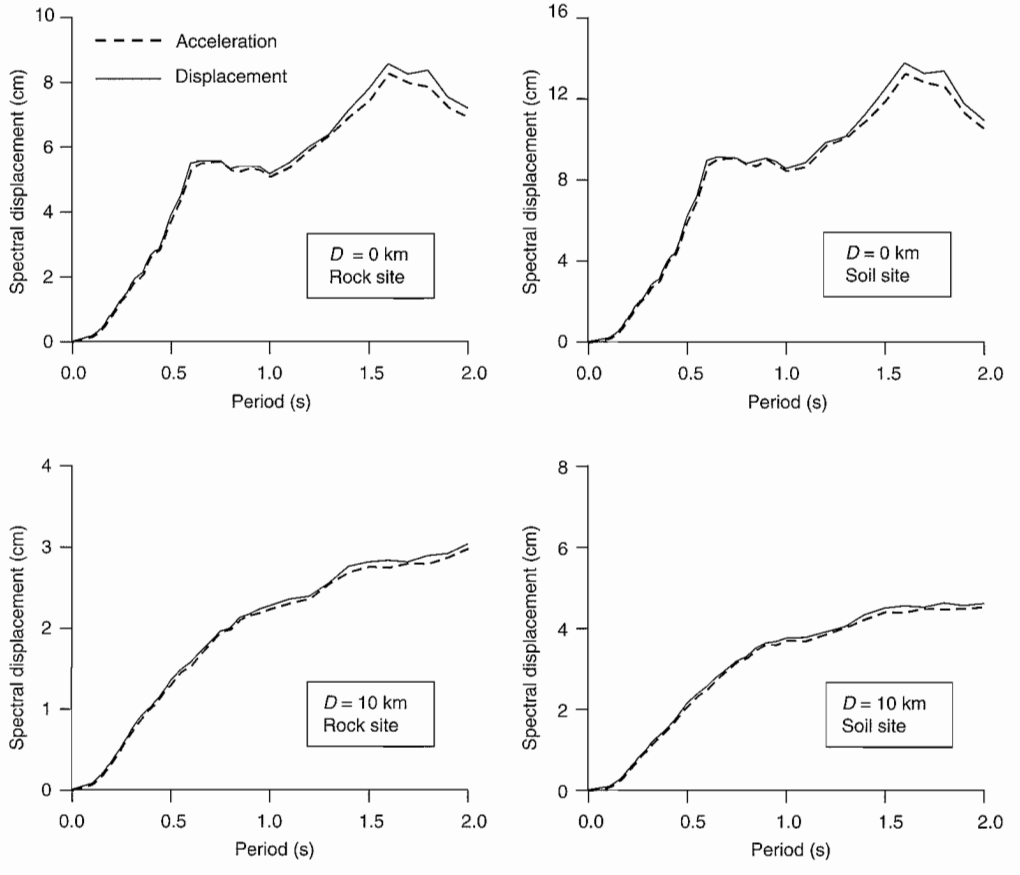



**Fig. 3.8.** Comparison of spectra obtained from acceleration or directly from displacement

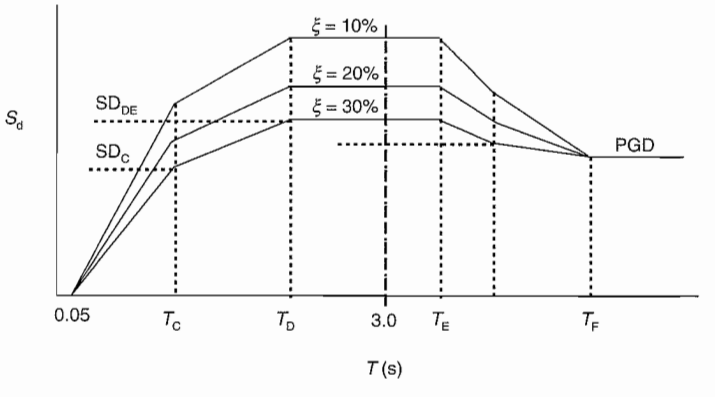



**Fig. 3.9.** Idealized displacement spectrum shape

displacement ($d_g$ or PGD). Up to the control period $T_E$, the ordinates $S_D$ are derived from *equations* (*3.2*) to (*3.5*), converting $S_e(T)$ to $S_{De}(T)$ through the pseudo-acceleration relationship given in *equation* (*3.7*) of EN 1998-1. Relationships to compute elastic displacement spectral values for periods greater than $T_E$ are provided in *equations* (*A.1*) and (*A.2*). In fact, the acceleration spectra expressed by *equations* (*3.1*) and (*3.4*) do not converge to the PGD at long periods.[17]

The values of the control periods $T_E$ and $T_F$ are given in *Table A1* of EN 1998-1 for the type 1 displacement spectrum and for the different ground types. Note that for all ground types

the value of $T_F$ is equal to 10 s. Values of $T_E$ vary between 4.5 s (ground type A) and 6 s (ground types C, D and E). For ground type B, the corner period $T_E = 5$ s. The shape and the corner periods proposed in EN 1998-1 were based on the work in Bommer and Elnashai[17] and Tolis and Faccioli.[18] However, only that part of the spectrum up to a period of 3 s was considered in the former study because longer periods would require use of hitherto unavailable digital recordings of a sufficiently large number. For displacement attenuation over longer periods, the reader is referred to Bommer and Elnashai,[17] where the 1995 Kobe strong motion was used to derive longer-period ordinates.

CHAPTER 4

# Design of buildings

## 4.1. Scope

This chapter covers the general rules for the seismic design of buildings using the structural materials encompassed by the Eurocodes. Accordingly, it deals essentially with the general conception of structures for buildings and its modelling and analysis for the purpose of checking the general requirements set forth in *Section 2* of EN 1998-1. *Clause 4.1.1*

This chapter loosely follows the contents of *Section 4* of EN 1998-1, but does not elaborate on all clauses of that section; neither does it strictly follow the sequence of clauses.

## 4.2. Conception of structures for earthquake resistant buildings

It is well known that a good seismic response of a building is much more easily achievable if its structural system possesses some characteristics that enable a clear and simple structural response under the action of the seismic event. *Clause 4.2.1(1)*

Such characteristics, being basic features of any structural system developed for a building, have to be considered and incorporated at the very earliest phases of the structural design i.e. at the conceptual design phase, which is the root of the design process and influences all other design activities and decisions.

Accordingly, the guiding principles for a good conceptual design are dealt with at the start of *Section 4*.

The aspects referred to in Eurocode 8 in this respect are: *Clause 4.2.1(2)*

- structural simplicity
- uniformity, symmetry and redundancy
- bi-directional resistance and stiffness
- torsional resistance and stiffness
- diaphragmatic behaviour at the storey level
- adequate foundations.

### 4.2.1. Structural simplicity

Structural simplicity implies that a clear and direct path for the transmission of the seismic forces is available. The seismic forces are associated with the different masses of a structure which are set in motion by its dynamic response to the seismic excitation. In buildings, an important part of their mass is located in the floor elements which act simultaneously as originators of the horizontal seismic forces and also as the elements that apply these forces to the vertical elements. These, in turn, have to transmit the forces to the ground at the foundation level. *Clause 4.2.1.1(1)*

Bearing in mind that, even for well-designed structures, a large-intensity earthquake will always be an extreme event which has the potential to drive the structure to its limits and to reveal all hidden weaknesses and defects, simple structures are at an advantage because their modelling, analysis, dimensioning, detailing and construction are subject to much less uncertainty and thus their seismic behaviour is much more consistent.

### 4.2.2. Uniformity, symmetry and redundancy

*Clauses 4.2.1.2(1), 4.2.1.2(3), 4.2.1.2(4)*

Uniformity, symmetry and redundancy are related characteristics which are normally correlated to structural simplicity

The advantage of structural uniformity in the seismic design context is that it allows the inertial forces created in the distributed masses of the building to be transmitted via short and direct paths, avoiding longer or indirect paths.

Structural uniformity of the building should be sought both in plan and in elevation. To achieve plan uniformity (and symmetry), it may be useful to subdivide the entire building into more uniform structural blocks through the use of seismic joints. These blocks will behave as dynamically independent units, but it should be checked that pounding of individual units is prevented, by providing appropriate width to these joints (as indicated in *clause 4.4.2.7* of EN 1998-1).

Furthermore, the in-plan uniformity of a building structure should, in most cases, be in line with the more or less uniform distribution of floor masses that occurs in buildings. This close relationship between the distribution of structural elements and masses will thus tend to eliminate large eccentricities.

The symmetrical or quasi-symmetrical distribution of the structural elements in plan is also a very positive feature for the seismic response of buildings because it decouples the vibration modes of the building in two independent horizontal directions, and thus its response to the seismic excitation is much simpler and less prone to torsional effects.

*Clause 4.2.1.2(2)*

On the other hand, uniformity of the building structure in elevation tends to eliminate the occurrence of large variations in the ratio between demand and resistance among the different vertical structural elements and thus avoids the appearance of sensitive zones where concentrations of stress or large ductility demands might prematurely cause collapse.

*Clause 4.2.1.2(5)*

Finally, the use of evenly distributed structural elements increases redundancy and allows a more favourable redistribution of action effects and widespread energy dissipation across the entire structure.

### 4.2.3. Bi-directional resistance and stiffness

*Clauses 4.2.1.3(1), 4.2.1.3(2)*

Seismic motion is a multi-directional phenomenon. In particular, its bi-directionality in the horizontal plan has to be considered in the conceptual design of the structure of a building. Accordingly, it is not surprising that Eurocode 8 requires that a building must be able to resist horizontal actions in any direction.

A very straightforward – and indeed the most common – way to achieve this is to arrange the structural elements in an orthogonal in-plan structural pattern. It is furthermore very desirable that such a pattern of structural elements ensures similar resistance and stiffness characteristics of the building as a whole in these two main orthogonal directions.

Provided that the building has resistance and stiffness in all horizontal directions, other structural arrangements in plan but not following an orthogonal pattern are naturally also acceptable, but normally they correspond to more complex seismic behaviours and require more sophisticated methods of analysis and dimensioning.

*Clause 4.2.1.3(3)*

The choice of the stiffness characteristics of the structure is also an important step in the conceptual design phase. In fact, the stiffness characteristics control the dynamic response of the building to future seismic events, and while it may be attempted to decrease the seismic forces by reducing the stiffness (i.e. by 'moving' the structure into the longer-period range where the spectral accelerations are smaller), their choice should also limit the development of excessive displacements that might lead to either instabilities due to second-order effects or excessive damage.

In this respect, it should be pointed out that in the conversion of Eurocode 8 from its previous pre-standard version (ENV 1998) into the full European standard (EN 1998-1) the influence of the lateral displacements of a building on its overall seismic response has been recognized. Thus, the emphasis that is given to an accurate evaluation of the displacements at the design level is reflected for instance in the deformation checks required for the verification of the damage limitation state and in the prescription that in reinforced concrete structures the structural analysis model should use the cracked stiffness of elements.

### 4.2.4. Torsional resistance and stiffness

*Clause 4.2.1.4(1)*

Torsional stiffness and resistance are characteristics of building structures which significantly influence their response to seismic actions. Responses in which translational motion is dominant are preferable to those in which torsional motion is significant because they tend to stress the different structural elements in a more uniform way.

To counteract the torsional response of buildings, the fundamental modes of vibration of the structure should be translational (or mainly translational in non-purely symmetrical buildings). To this end, the torsional stiffness of the structure must be sufficiently large to ensure that the first torsional vibration mode has a frequency higher than the translational modes.

In fact this is implicit in *condition 4.1b* of *clause 4.2.3.2(6)*, which establishes the criteria for in-plan regularity of a building. Such a condition corresponds to the objective that in regular buildings the first torsional mode has a frequency higher than the translational modes, thus ensuring that its importance in the global seismic response of the building is relatively minor.

It should be noted that this concern with the poorer behaviour of buildings with small torsional stiffness is also present in the classification of reinforced concrete buildings, for which a class of '*torsionally flexible systems*' is introduced (see *clause 5.1.2* of EN 1998-1). In line with this concern, these systems are given smaller values for their behaviour factor (see *clause 5.2.2.2* of EN 1998-1).

For the purpose of ensuring adequate torsional stiffness and resistance, the main elements resisting the seismic action should be well distributed in plan or, even better, they should be close to the periphery of the building and oriented along the two main directions. Buildings with their main lateral resisting elements located at the centre of the building in plan should be avoided because, even in the case of symmetrical structural arrangements, they may be prone to large uncontrolled torsional motions.

### 4.2.5. Diaphragmatic behaviour at the storey level

*Clause 4.2.1.5(1)*

In building structures the floors act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action.

The action of these diaphragms is especially relevant to complex and non-uniform layouts of the vertical structural systems because, in these cases, as indicated above, the inertia forces created in the distributed masses of the building have to be transmitted along more complex and longer paths within these diaphragms.

Diaphragmatic action at the floor levels is also important where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems), because in those situations the interaction between these different structural systems varies along the height of the building, and compatibility between them is ensured by the diaphragmatic action of the floors.

*Clause 4.2.1.5(2)*

Accordingly, floor systems (and the roof) should be considered as part of the overall structural system of the building, and provided with appropriate in-plane stiffness and resistance as well as with effective connection to the vertical structural elements.

Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, as these elements attract large forces which have to be transmitted effectively by the floor elements connected to those vertical elements.

*Clause 4.2.1.5(3)*

Diaphragms should have appropriate in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems, and, in many cases, at the conceptual design phase the choice of a rigid diaphragm approach is appropriate because it distributes the deformation in the vertical elements more uniformly. Furthermore, a building structure with rigid diaphragms allows for simplifying assumptions for its modelling and analysis (see *clause 4.3* of EN 1998-1).

The validity of the assumption of a rigid floor diaphragm depends on whether its deformation is or is not negligible in comparison with the deformation of the vertical elements. A note to *clause 4.3.1(4)* of EN 1998-1 indicates, as a general rule, that this assumption may be made if the horizontal displacements in the floor plane are not changed by more than 10% by the deformation of the floor itself. If this not the case, the flexibility of the floor diaphragm should be accounted for in the modelling of the structure.

Besides stiffness, resistance of the floor diaphragm and its connections should also be checked, either implicitly or explicitly. This matter is dealt with in general terms in *clause 4.4.2.5* of EN 1998-1, and more specific provisions for reinforced concrete and timber diaphragms are presented in *clauses 5.10* and *8.5.3*, respectively.

### 4.2.6. Adequate foundation

*Clauses 4.2.1.6(1), 4.2.1.6(2), 4.2.1.6(3)*

The choice of suitable foundation conditions is of paramount importance to ensure the good seismic response of a building structure. In fact, it should be stressed that a prerequisite for the survival of a structure of an earthquake event is that the bearing capacity of the main elements sustaining the gravity forces, among which the foundations are of prime importance, is retained throughout the duration of and after the event. Furthermore, even if the foundations do not collapse but instead suffer damage, their repair is extremely difficult and normally leads to a decision to demolish after the earthquake – i.e. total economic loss.

Accordingly, EN 1998-1 recommends that at the conceptual design stage the foundations and their connections to the superstructure should be developed in such a way as to ensure that the whole building is subjected to a uniform seismic excitation. Additionally, all foundation elements should be tied together and their stiffness should be appropriate to the stiffness of the vertical elements that they support (e.g. structural walls).

The conceptual design of foundation systems is dealt with in more detail in *Section 5* of EN 1998-5, where rules for the verification of tying elements are also provided (these rules should be taken into account in combination with the general rules set forth in *clause* 4.4.2.6 of EN 1998-1).

## 4.3. Structural regularity and its implications for design

### 4.3.1. Introduction

*Clauses 4.2.3.1(1), 4.2.3.1(2), 4.2.3.1(3)*

There is plenty of evidence from damage observation after earthquakes that regular buildings tend to behave much better than irregular ones. However, a precise definition of what is a regular structure in the context of the seismic response of buildings has eluded many attempts to achieve it. There are so many variables and structural characteristics that may (or should) be considered in such a definition that the classification of a building as 'regular' is, in the end, mostly intuitive. EN 1998-1 recognizes this difficulty, and does not attempt to establish very strict rules for the distinction between regular and non-regular buildings. Rather, it provides a relatively loose set of characteristics that a building structure should possess to be classified as regular. This classification serves the purpose of, essentially, establishing some distinctions regarding concerns relating to the more or less simplified structural model and the method of analysis to be used as well as in concerns relating to the value of the behaviour factor.

With this approach, EN 1998-1 does not forbid the design and construction of non-regular structures but, rather, attempts to encourage the choice of regular structures both by making it easier to design them and also by making them more economic (as a consequence of using in such cases higher values of the behaviour factor).

As in most other modern seismic design codes, the concept of building regularity in EN 1998-1 is presented with a separation between regularity in plan and regularity in elevation. Moreover, regularity in elevation is considered separately in the two orthogonal directions in which the horizontal components of the seismic action are applied, meaning that the structural system may be characterized as regular in one of these two horizontal directions but not in the other. However, a building assumes a single characterization for regularity in plan, independent of direction.

In order to reduce stresses due to the constraint of volumetric changes (thermal, or due to concrete shrinkage), buildings which are long in plan often have their structure divided, by means of vertical expansion joints, into parts that can be considered as separate above the level of the foundation. The same practice is recommended in buildings with a plan shape consisting of several (close to) rectangular parts (L-, C-, H-, I- or X-shaped plans), for reasons of clarity and predictability of their seismic response (see points 2 and 3 in Section 4.3.2.1). The parts into which the structure is divided through such joints are considered as 'dynamically independent'. Structural regularity is defined and checked at the level of each individual dynamically independent part of the building structure, regardless of whether these parts are analysed separately or together in a single model (which might be the case if they share a common foundation, or if the designer considers a single model as convenient for comparing the relative displacements of adjacent units to the width of the joint between them).

Unlike US codes (e.g. see Building Seismic Safety Council[39] and Structural Engineers Association of California[40]), which set quantitative – albeit arbitrary – criteria for regularity:

- in plan, on the basis of the planwise variation of floor displacements as computed from the analysis
- in elevation, based on the variation of mass, stiffness and strength from storey to storey.

Eurocode 8 introduces qualitative criteria, which can be checked easily at the preliminary design stage by inspection or through simple calculations, without doing an analysis. This makes sense, as the main purpose of the regularity classification is to determine what type of linear analysis may be used for the design: in three dimensions (3D), using a spatial model, or in two dimensions (2D), using two separate planar models, depending on the regularity in plan; and static, using equivalent lateral forces, or response spectrum analysis, depending on the regularity in elevation. So, it does not make sense to first do an analysis to find out what type of analysis is allowed to be used at the end. Moreover, the regularity in plan and in elevation affects the value of the behaviour factor $q$ that determines the design spectrum used in linear analysis.

## 4.3.2. Regularity in plan

Regularity in plan influences essentially the choice of the structural model. The reasoning behind the provisions of EN 1998-1 in this respect is that structures that are regular in plan tend to respond to seismic excitation along their main structural directions in an uncoupled manner. Accordingly, for the design of regular structures in plan it is acceptable to analyse them in a simplified way, using planar models in each main structural direction. *Clause 4.2.3.2*

### *4.3.2.1. Criteria for structural regularity in plan*

A building can be characterized as regular in plan if it meets all of the following numbered conditions, at all storey levels: *Clause 4.2.3.2(1)*

(1) The distribution in plan of the lateral stiffness and the mass is approximately symmetrical with respect to two orthogonal horizontal axes. Normally, the horizontal components of the seismic action are consequently applied along these two axes. As absolute symmetry is not required, it is up to the designer to judge whether this condition is met or not. *Clause 4.2.3.2(2)*

*Clauses 4.2.3.2(3), 4.2.3.2(4)*

(2) The outline of the structure in plan should have a compact configuration, delimited by a convex polygonal line. What counts in this respect is the structure, as defined in plan by its vertical elements, and not the floor (including balconies and any other cantilevering parts). Any single re-entrant corner or edge recess of the outline of the structure in plan should not leave an area between it and the convex polygonal line enveloping it which is more than 5% of the area inside the outline. For a rectangular plan with a single re-entrant corner or edge recess, this is equivalent to, for example, a recess of 20% of the parallel floor dimension in one direction and of 25% in the other; or, if there are four such re-entrant corners or edge recesses, to, for example, a recess of 25% of the parallel floor dimension in both directions. L-, C-, H-, I- or X-shaped plans should respect this condition, in order for the structure to be considered as regular in plan.

*Clause 4.2.3.2(4)*

(3) It should be possible to consider the floors as rigid diaphragms, in the sense that their in-plane stiffness is sufficiently large, so that the floor in-plan deformation due to the seismic action is negligible compared with the interstorey drifts and has a minor effect on the distribution of seismic shears among the vertical structural elements. Conventionally, a rigid diaphragm is defined as one in which, when it is modelled with its actual in-plane flexibility, its horizontal displacements due to the seismic action nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements. However, it is neither required nor expected that fulfilment of this latter definition is computationally checked. For instance, a solid reinforced concrete slab (or cast-in-place topping connected to a precast floor or roof through a clean, rough interface or shear connectors) may be considered as a rigid diaphragm, if its thickness and reinforcement (in both horizontal directions) are well above the minimum thickness of 70 mm and the minimum slab reinforcement of Eurocode 2 (which is a Nationally Determined Parameter (NDP) to be specified in the National Annex to Eurocode 2) required in *clause 5.10* of EN 1998-1 for concrete diaphragms (rigid or not). For a diaphragm to be considered rigid, it should also be free of large openings, especially in the vicinity of the main vertical structural elements. If the designer does not feel confident that the rigid diaphragm assumption will be met due to the large size of such openings and/or the small thickness of the concrete slab, then he or she may want to apply the above conventional definition to check the rigidity of the diaphragm.

*Clause 4.2.3.2(5)*

(4) The aspect ratio of the floor plan, $\lambda = L_{max}/L_{min}$, where $L_{max}$ and $L_{min}$ are respectively the larger and smaller in-plan dimensions of the floor measured in any two orthogonal directions, should be not more than 4. This limit is to avoid situations in which, despite the in-plane rigidity of the diaphragm, its deformation due to the seismic action as a deep beam on elastic supports affects the distribution of seismic shears among the vertical structural elements.

*Clause 4.2.3.2(6)*

(5) In each of the two orthogonal horizontal directions, $x$ and $y$, of near-symmetry according to condition 1 above, the 'static' eccentricity, $e$, between the floor centre of mass and the storey centre of lateral stiffness is not greater than 30% of the corresponding storey torsional radius, $r$:

$$e_x \le 0.3r_x \qquad e_y \le 0.3r_y \tag{D4.1}$$

The torsional radius $r_x$ in equation (D4.1) is defined as the square root of the ratio of (a) the torsional stiffness of the storey with respect to the centre of lateral stiffness to (b) the storey lateral stiffness in the (orthogonal to $x$) $y$ direction; for $r_y$, the storey lateral stiffness in the (orthogonal to $y$) $x$ direction is used in the denominator.

*Clause 4.2.3.2(6)*

(6) The torsional radius of the storey in each of the two orthogonal horizontal directions, $x$ and $y$, of near-symmetry according to condition 1 above is not greater than the radius of gyration of the floor mass:

$$r_x \ge l_s \qquad r_y \ge l_s \tag{D4.2}$$

The radius of gyration of the floor mass in plan, $l_s$, is defined as the square root of the ratio of (a) the polar moment of inertia of the floor mass in plan with respect to the centre of mass of the floor to (b) the floor mass. If the mass is uniformly distributed over a rectangular floor area with dimensions $l$ and $b$ (that include the floor area outside of the outline of the vertical elements of the structural system), $l_s$ is equal to $\sqrt{[(l^2 + b^2)/12]}$.

Condition 6 ensures that the period of the fundamental (primarily) translational mode in each of the two horizontal directions, $x$ and $y$, is not shorter than the lower (primarily) torsional mode about the vertical axis $z$, and prevents strong coupling of the torsional and translational response, which is considered uncontrollable and potentially very dangerous. In fact, as $l_s$ is defined with respect to the centre of mass of the floor in plan, the torsional radii $r_x$ and $r_y$ that should be used in equation (D4.2) for this ranking of the three above-mentioned modes to be ensured are those defined with respect to the storey centre of mass, $r_{mx}$ and $r_{my}$, which are related to the torsional radii $r_x$, $r_y$ defined with respect to the storey centre of lateral stiffness as $r_{mx} = \sqrt{(r_x^2 + e_x^2)}$ and $r_{my} = \sqrt{(r_y^2 + e_y^2)}$. The greater the 'static' eccentricities $e_x$, $e_y$ between the centres of mass and stiffness, the larger the margin provided by equation (D4.2) against a torsional mode becoming predominant. It is worth remembering that if the elements of the lateral-load-resisting system are distributed in plan as uniformly as the mass, then the condition of equation (D4.2) is satisfied (be it marginally) and does need to be checked explicitly, whereas if the main lateral-load-resisting elements, such as strong walls or bracings, are concentrated near the plan centre, this condition may not be met, and equation (D4.2) needs to be checked.

It is worth noting that, if the lower few eigenvalues are determined within the context of a modal response-spectrum analysis, they may be used directly to determine whether equation (D4.2) is satisfied for the building as a whole: if the period of a predominantly torsional mode of vibration is shorter than those of the primarily translational ones in the two horizontal directions $x$ and $y$, then equation (D4.2) may be considered as satisfied.

An exhaustive review of the available literature on the seismic response of torsionally unbalanced structures[41] has shown that conditions 5 and 6 provide a margin against excessive torsional response. In Fig. 4.1, solid black symbols represent good or satisfactory behaviour, while open and grey symbols correspond to poor behaviour, according to non-linear dynamic analyses of various degrees of sophistication and reliability. In Fig. 4.1 the regularity region of EN 1998-1 is that to the left of the right-most inclined line and above the horizontal line at $r/b = 0.35$ (the ratio $r/b$ ranges from $0.3r/l_s$ to $0.4r/l_s$, depending on the aspect ratio of the floor plan, $l/b$).

*Clause 4.2.3.2(8)*

The centre of lateral stiffness is defined as the point in plan with the following property: any set of horizontal forces applied at floor levels through that point produces only translation of the individual storeys, without any rotation with respect to the vertical axis (twist). Conversely, any set of storey torques (i.e. of moments with respect to the vertical axis, $z$) produces only rotation of the floors about the vertical axis that passes through the centre of lateral stiffness, without horizontal displacement of that point in $x$ and $y$ at any storey. If such a point exists, the torsional radius, $r$, defined as the square root of the ratio of torsional stiffness with respect to the centre of lateral stiffness to the lateral stiffness in one horizontal direction, is unique and well defined. Unfortunately, the centre of lateral stiffness, as defined above, and with it the torsional radius, $r$, are unique and independent of the lateral loading only in single-storey buildings. In buildings of two storeys or more, such a definition is not unique and depends on the distribution of lateral loading with height. This is especially so if the structural system consists of subsystems which develop different patterns of storey horizontal displacements under the same set of storey forces (e.g. moment frames exhibit a shear-beam type of horizontal displacement, while walls and frames with bracings – concentric or eccentric – behave more like vertical cantilevers). For the general case of such systems, *Section 4* of EN 1998-1 refers to the National Annex for an appropriate approximate definition of the centre of lateral stiffness and of the torsional radius, $r$.

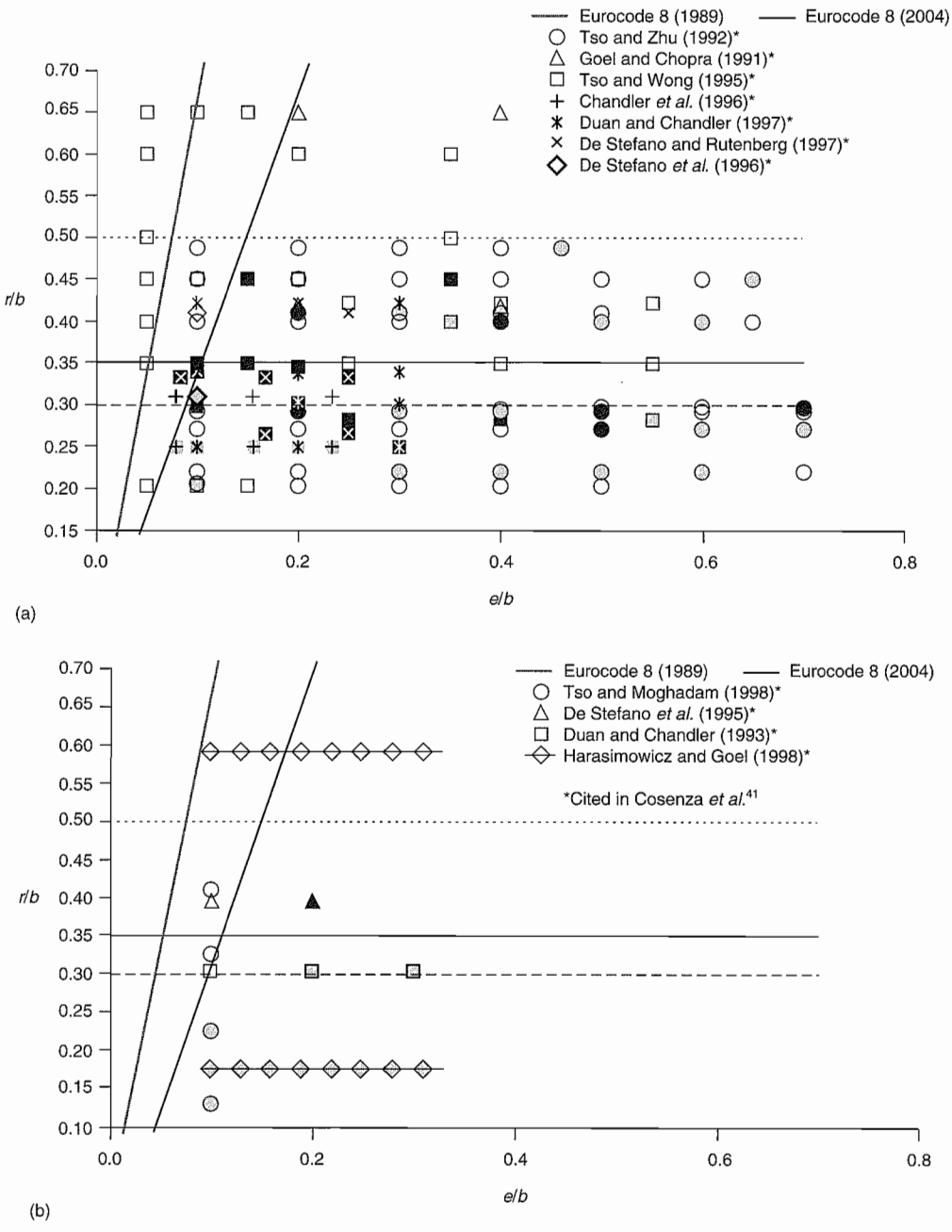



**Fig. 4.1.** Good or satisfactory (solid black symbols) versus poor performance (open and grey symbols) in the space of normalized static eccentricity, e, and torsional radius *r*, in (a) single-storey and (b) multi-storey systems[41]

*Clauses 4.2.3.2(7), 4.2.3.2(9)*

For single-storey buildings *Section 4* of EN 1998-1 allows the determination of the centre of lateral stiffness and the torsional radius on the basis of the moments of inertia of the cross-sections of the vertical elements, neglecting the effect of beams, as

$$x_{CS} = \frac{\sum(xEI_y)}{\sum(EI_y)} \qquad y_{CS} = \frac{\sum(yEI_x)}{\sum(EI_x)} \tag{D4.3}$$

$$r_x = \sqrt{\frac{\sum(x^2EI_y + y^2EI_x)}{\sum(EI_y)}} \qquad r_y = \sqrt{\frac{\sum(x^2EI_y + y^2EI_x)}{\sum(EI_x)}} \tag{D4.4}$$

In equations (D4.3) and (D4.4), $EI_x$ and $EI_y$ denote the section rigidities for bending within a vertical plane parallel to the horizontal directions $x$ or $y$, respectively (i.e. about an axis parallel to axis $y$ or $x$, respectively). Moreover, *Section 4* of EN 1998-1 allows the use of equations (D4.3) and (D4.4) to determine the centre of lateral stiffness and the torsional radius in multi-storey buildings also, provided that their structural system consists of subsystems which develop similar patterns of storey horizontal displacements under storey horizontal forces $F_i$ proportional to $m_i z_i$, namely only moment frames (exhibiting a shear-beam type of horizontal displacement pattern), or only walls (deflecting like vertical cantilevers). For wall systems, in which shear deformations are also significant in addition to the flexural ones, an equivalent rigidity of the section should be used in equations (D4.3) and (D4.4). It is noted that, unlike the general and more accurate but tedious method outlined above, which yields a single pair of radii $r_x$ and $r_y$ for the entire building, to be used to check if equations (D4.1) and (D4.2) are satisfied at all storeys, if the cross-section of vertical elements changes from one storey to another, the approximate procedure of equations (D4.3) and (D4.4) gives different pairs of $r_x$ and $r_y$, and possibly different locations of the centre of stiffness in different storeys (which affects, in turn, the static eccentricities $e_x$ and $e_y$).

#### *4.3.2.2. Design implications of regularity in plan*

**Implications for the analysis: the 2D (plane) versus 3D (spatial) structural model**

*Clauses 4.2.3.1(2), 4.2.3.1(3), 4.3.1(5)*

If a building is characterized as regular in plan, the analysis for the two horizontal components of the seismic action may use an independent 2D model in each of the two horizontal directions of (near-) symmetry, $x$ and $y$. In such a model, the structure is considered to consist of a number of plane frames (moment frames, or frames with concentric or eccentric bracings) and/or walls (some of which may actually belong in a plane frame together with co-planar beams and columns), all of them constrained to have the same horizontal displacement at floor levels.

Each 2D model will be analysed for the horizontal component of the seismic action parallel to it (possibly with consideration of the vertical component as well, if required), and will yield internal forces and other seismic action effects only within vertical planes parallel to that of the analysis. This means that the analysis will give no internal forces for beams, bracings or even walls which are in vertical planes orthogonal to the horizontal component of the seismic action considered. Bending in columns and walls will also be uniaxial, with axial force only due to the horizontal component of the seismic action which is parallel to the plane of the analysis.

Given the proliferation of commercial computer programs for linear elastic seismic response analysis – static or dynamic – in 3D, there is little sense today in pursuing analyses with two independent 2D models instead of a spatial 3D model. This is particularly so if the analysis is done for the purposes of seismic design, as in that case the software normally has capabilities to post-process the results, in order to serve the specific needs of design. Such post-processing is greatly facilitated if a single (3D) model is used for the entire structure. However, if two independent analyses are done using two different 2D models, the results of these analyses may have to be processed by a special post-processing module that reads and interprets topology data and internal force results from two different sources. Alternatively, the combination of the internal force results can be done manually. It should be noted that internal force results from the two different 2D analyses need to be combined primarily in columns, due to the requirement to consider that the two horizontal components of the seismic action act simultaneously and to combine their action effects (either via the 1:0.3 rule or through the square root of the sum of the squares (SRSS)). It is true that the facility provided in *Section 5* of EN 1998-1 for the biaxial bending of columns (namely to dimension the column for a uniaxial bending moment equal to that from the analysis divided by 0.7, neglecting the simultaneously acting orthogonal component of bending moment) is quite convenient in this respect. However, the need to combine the column axial forces due to the two horizontal components of the seismic action (via the 1:0.3 or the SRSS rule) remains,

even within the framework of uniaxial bending mentioned in the last sentence for concrete columns. A possible way out for such columns might be to: (1) dimension the vertical reinforcement of the two opposite sides of the cross-section considering uniaxial bending (with the 1/0.7 magnification on the moment) with axial force due to the horizontal component of the seismic action which is orthogonal to the two opposite sides considered; (2) repeat the exercise for the two other sides and the corresponding horizontal component of the action; and (3) add the resulting vertical reinforcement requirements on the section, neglecting any positive contribution of any one of them to the flexural resistance in the orthogonal direction of bending.

All things considered, it is not worthwhile using linear analysis with two independent 2D models in building structures which are regular in plan. In that regard, the characterization of a structure as regular or non-regular in plan is important only for the default value of the part of the behaviour factor $q$ which is due to the redundancy of the structural system, as explained below.

However, the facility of two independent 2D models is very important for non-linear analysis, static (pushover) or dynamic (time–history). Reliable, widely accepted and numerically stable non-linear constitutive models (including the associated failure criteria) are available only for members in uniaxial bending with (little-varying) axial force; their extension to biaxial bending for wide use in 3D analysis belongs to the future. So, for the use of non-linear analysis the characterization of a building structure as regular or non-regular in plan is very important.

*Clause 4.3.3.1(8)*

Within the framework of the lateral force procedure of analysis, two independent 2D models may also be used for buildings which have:

(1) a height less than 10 m, or 40% of the plan dimensions
(2) storey centres of mass and stiffness approximately on (two) vertical lines
(3) partitions and claddings well distributed vertically and horizontally, so that any potential interaction with the structural system does not affect its regularity
(4) torsional radii in the two horizontal directions at least equal to $r_x = \sqrt{(l_s^2 + e_x^2)}$ and $r_y = \sqrt{(l_s^2 + e_y^2)}$.

If conditions 1 to 3 are fulfilled, but not condition 4, then two separate 2D models may still be used, provided that all seismic action effects from the 2D analyses are increased by 25%.

The above relaxation of the regularity conditions for using two independent 2D models instead of a full 3D model is meant to make it easier for the designer (and hence the owner) of small buildings to apply Eurocode 8. For this reason, the extent of the application of this facility will be determined nationally, and a note in Eurocode 8 states, without giving any recommendations for the selection, that the importance class(es) to which this relaxation will apply should be listed in the National Annex.

**Implications for the behaviour factor $q$**

*Clauses 5.2.2.2(6), 6.3.2(4), 7.3.2(4)*

As we will see in more detail in Chapters 5 to 7 of this guide, in most types of structural systems system overstrength due to redundancy is explicitly factored into the value of $q$, as a ratio $\alpha_u/\alpha_1$. This is the ratio of the seismic action that causes development of a full plastic mechanism ($\alpha_u$) to the seismic action at the first plastification in the system ($\alpha_1$). The value of $\alpha_1$ may be computed as the lower value over all member ends in the structure of the ratio $(S_{Rd} - S_V)/S_E$, where $S_{Rd}$ is the design value of the action effect capacity at the location of first plastification, and $S_E$ and $S_V$ are the values of the action effect there from the elastic analysis for the design seismic action and for the gravity loads included in the load combination of the 'seismic design situation'. The value of $\alpha_u$ may be found as the ratio of the base shear on development of a full plastic mechanism according to a pushover analysis to the base shear due to the design seismic action (e.g. see Fig. 5.2). As the designer may not consider it worth performing iterations of pushover analyses and design based on elastic analysis just to compute the ratio $\alpha_u/\alpha_1$ for the determination of the $q$ factor, *Sections 5–7* of EN 1998-1 give default values for this ratio. For buildings which are regular in plan, the default values range

from $\alpha_u/\alpha_1 = 1.0$, in buildings with very little structural redundancy, to $\alpha_u/\alpha_1 = 1.3$ in multi-storey multi-bay frames, with a default value of $\alpha_u/\alpha_1 = 1.2$ used in the fairly common concrete dual systems (frame or wall equivalent), concrete coupled-wall systems and steel or composite frames with eccentric bracings.

In buildings which are not regular in plan, the default value of $\alpha_u/\alpha_1$ is the average of (1) 1.0 and (2) the default values given for buildings regular in plan. For the values of $\alpha_u/\alpha_1 = 1.2$–$1.3$ specified as the default for the most common structural systems in the case of regularity in plan, the reduction in the default $q$ factor is around 10%. If the designer considers such a reduction unacceptable, he or she may resort to iterations of pushover analyses and design based on elastic analysis, to quantify a possibly higher value of $\alpha_u/\alpha_1$ for the (non-regular) structural system.

Fulfilment or not of equation (D4.2) has very important implications for the value of the behaviour factor $q$ of concrete buildings. If at any floor, one or both conditions of equation (D4.2) are not met (i.e. if the radius of gyration of the floor mass exceeds the torsional radius in one or both of the two main directions of the building in plan), then the structural system is characterized as torsionally flexible, and the basic value of the behaviour factor $q$ (i.e. prior to any reduction due to potential non-regularity in elevation (see Section 4.3.3.1)) is reduced to a value of $q_o = 2$ for Ductility Class Medium (DCM) or $q_o = 3$ for Ductility Class High (DCH). As non-fulfilment of equation (D4.2) is most commonly due to the presence of stiff concrete elements, such as walls or cores, near the centre of the building in plan, *Section 6* of EN 1998-1 adopts the same reduction of the basic value of the $q$ factor in steel buildings which employ such walls or cores for (part of) their earthquake resistance.

*Clauses 5.2.2.2(2), 6.3.2(1)*

## 4.3.3. Regularity in elevation

### *4.3.3.1. Criteria for structural regularity in elevation*

A building is characterized as regular in elevation if it meets all the following conditions: *Clause 4.2.3.3(1)*

- Its lateral force-resisting systems (moment frames or frames with bracings, walls, etc.) should continue from the foundation to the top of the (relevant part of the) building. *Clause 4.2.3.3(2)*
- The storey mass and stiffness should be constant or decrease gradually and smoothly to the top. *Clause 4.2.3.3(3)*
- In frame buildings, there should be no abrupt variations of the overstrength of the individual storeys (including the contribution of masonry infills to storey shear strength) relative to the design storey shear. The storey shear force capacity can be computed as the sum over all vertical elements of the ratio of moment capacity at the storey bottom to the corresponding shear span (half of clear storey height in columns, or half of distance from the storey bottom to the top of the building in walls), plus the sum of shear strengths of infill walls (roughly equal to the minimum horizontal section area of the wall panel times the shear strength of bed joints). *Clause 4.2.3.3(4)*
- Individual setbacks of each side of the building should not exceed 10% of the parallel dimension of the underlying storey. *Clause 4.2.3.3(5)*
- The total setback of each side at the top with respect to the base, if not provided symmetrically on both sides of the building, should not exceed 30% of the parallel dimension at the base of the building.
- If there is a single setback within the bottom 15% of the total height of the building, $H$, this setback should not exceed 50% of the parallel dimension at the base of the building. In this particular case there should be no undue reliance on the enlargement of the structure at the base for transferring to the ground the seismic shears that develop in that part of the building above the enlargement. In other words, these shears should be transferred mainly through the vertical continuation of the upper part of the building to the ground, and the enlargement of the building at the base should mainly transfer to the ground its own seismic shear. The relevant clause of Eurocode 8 requires that the vertical continuation of the upper part of the building to the ground is designed for a seismic shear at least equal to 75% of the shear force that would develop in that zone in a similar building without the base enlargement. Strictly speaking, for this requirement to

be implemented, a structural model of a fictitious building without the base enlargement needs to be constructed and analysed, to compute its seismic shears within that part of the height corresponding to the enlargement and make sure that the corresponding part of the real building is not designed for less than 75% of these shears. Nonetheless, it serves the intended purpose to estimate these shears assuming that the fictitious building has similar dynamic characteristics to the real one and that a roughly linear first mode controls storey shears. Then, the storey shear that should be exceeded within the vertical continuation of the upper part of the building to the ground can be taken equal to the (base) shear of this upper part just above the base enlargement multiplied by $0.75[1 - (h_{n+1}/H)^2]/[1 - (h_i/H)^2]$, where $i$ indexes the floor level starting at the bottom, $n \geq i$ corresponds to the floor at the enlargement and $n$ to that immediately above, $h_i$ denotes the floor elevation from the ground and $H$ is the total height of the building.

#### *4.3.3.2. Design implications of regularity in elevation*

**Implications for the analysis: lateral force versus the modal response spectrum method**

*Clauses 4.2.3.1(2), 4.2.3.1(3), 4.3.3.2.1(2), 4.3.3.3.1(1)*

In the presence of structural non-regularity in elevation, it is unlikely that the first mode shape will be linear from the bottom to the top of the building. So, as a postulated linear mode shape underlies the lateral load pattern of the lateral force method of analysis, this method is not considered applicable to buildings which are not characterized as structurally regular in elevation. The modal response spectrum method has been found capable of capturing well the effects of structural non-regularity in elevation, not only in the linear elastic response, but, to a large extent, in the non-linear response as well. So its application is mandatory within the framework of force-based design of structures with non-regularity in elevation. This should not be considered as a penalty for such structures: a modal response spectrum analysis does not produce overall more conservative results than the lateral force method. It is simply an attempt to better approximate the peak dynamic response at the level of member internal forces and deformations.

**Implications for the behaviour factor $q$**

*Clauses 4.2.3.1(7), 5.2.2.2(3), 6.3.2(2), 7.3.2(2), 8.3(2), 9.3(5)*

In the presence of structural non-regularities in elevation, the uniform distribution of inelastic deformations throughout the height of the structure, pursued through

- capacity design in flexure of the columns of the moment frame, so that they are stronger than its beams
- promotion of concrete walls and their overdesign in flexure and shear above the base, so that they remain elastic there
- capacity design of all members in a steel or composite frame with bracings that are not intended for energy dissipation, so that they remain elastic, etc.,

may be in doubt. It is likely that there will be a local concentration of inelasticity at the elevation(s) where the irregularity takes place (e.g. at a large setback, or where a lateral force-resisting system is vertically discontinued, or where a storey has mass, lateral stiffness or overstrength higher than in the storey below) beyond the predictions of the modal response spectrum (elastic) analysis. Such a concentration will increase locally the deformation demands on dissipative regions, above the building-average value corresponding to the value of the $q$ factor used in the design. Instead of imposing more strict detailing on the regions likely to be affected by the structural non-regularity to enhance their ductility capacity so that they meet the locally increased ductility demands, the value of the $q$ factor used in the force-based design is reduced by 20%, without relaxing the detailing requirements anywhere in the structure. The resulting 25% increase in strength demands for the dimensioning is intended to reduce the locally increased ductility demands around the elevation(s) where the irregularity takes place to the level of their ductility capacity. No matter how closely that goal is met, the 25% increase in strength demands for the entire structure is certainly a major disincentive to adopting a structural system that is non-regular in elevation.

## 4.4. Combination of gravity loads and other actions with the design seismic action

### 4.4.1. Combination for local effects

*Clauses 3.2.4(1), 3.2.4(4), 4.2.4(1)*

At the local level, i.e. for the verification of members and sections, the design seismic action is combined with other actions as specified in EN 1990[3] for the seismic design situation. Symbolically, this combination is $\sum G_{k,j}$ '+' $A_{Ed}$ '+' $\sum \psi_{2,i} Q_{k,i}$, where $G_{k,j}$ is the nominal value of permanent action $j$ (normally the self weight and all other dead loads), $A_{Ed}$ is the design seismic action (corresponding to the 'reference return period' multiplied by the importance factor), $Q_{k,j}$ is the nominal value of variable action $i$ (live loads (in Eurocode terminology 'imposed loads'): wind, snow load, temperature, etc.) and $\psi_{2,i} Q_{k,i}$ is the quasi-permanent (i.e. the arbitrary-point-in-time) value of variable action $i$.

Coefficients $\psi_{2,i}$ are defined in Normative Annex A1 of EN 1990 as an NDP with the following recommended values:

- $\psi_{2,i} = 0$ for wind and temperature
- $\psi_{2,i} = 0$ for snow on the roof at altitudes below 1000 m above sea level in all CEN countries other than Iceland, Norway, Sweden and Finland, or $\psi_{2,i} = 0.2$ all over these four countries and at altitudes over 1000 m above sea level in all other CEN countries
- $\psi_{2,i} = 0$ for live loads on roofs
- $\psi_{2,i} = 0.3$ for live loads in residential or office buildings and for traffic loads from vehicles weighing between 30 and 160 kN
- $\psi_{2,i} = 0.6$ for live loads in areas used for public gatherings or shopping and for traffic loads from vehicles below 30 kN in weight
- $\psi_{2,i} = 0.8$ for live loads in storage areas.

Being quasi-permanent, the action effects of $\psi_{2,i} Q_{k,i}$ are taken into account always, regardless of whether they are locally favourable or unfavourable. If the same value of $\psi_{2,i}$ applies to all storeys, this is very convenient for the design, as it lends itself to a single analysis for the nominal value of the variable action, $Q_{k,i}$, for the whole building. The results of this analysis are multiplied by $\psi_{2,i}$ for superposition with those of the permanent and the seismic actions in the seismic design situation, or multiplied by the appropriate partial factor for variable actions, for superposition with those of the permanent actions in the persistent and transient design situations. If different values of $\psi_{2,i}$ are used in different storeys, separate analyses for live loads on groups of storeys with different $\psi_{2,i}$ values will be necessary.

### 4.4.2. Combination for global effects

*Clauses 3.2.4(2), 3.2.4(3), 4.2.4(2), 4.3.1(10)*

Eurocode 8 reduces the value of variable actions to be combined with the design seismic action beyond the level of a single member ('global' effects, such as the overall seismic shear or overturning moment in a storey, etc.) below that used locally for the verification of members and sections. This is to take into account the reduced likelihood that the live loads $\psi_{2,i} Q_{k,i}$ may not be present over the entire structure during the design earthquake. The reduction is effected in the calculation of masses, as these affect the inertia forces. This reduction of live loads may also account for a reduced participation of masses in the motion due to possibly non-rigid connection to the structure (in other words, some masses may not vibrate in full phase with their support, or at full amplitude).

The reduction factor to be applied on live loads $\psi_{2,i} Q_{k,i}$ for buildings is defined in *Section 4* of EN 1998-1 as an NDP. The recommended value is 0.5 for all storeys – other than the roof – used for residential or office purposes or for public gathering (except shopping areas) which are considered as independently occupied, or 0.8 in those storeys of the above uses which may be considered to have correlated occupancies. No reduction in live loads is recommended for any other use or on roofs.

If the same value of $\psi_{2,i}$ applies to all storeys, the facility of reducing live loads in some storeys below the value $\psi_{2,i} Q_{k,i}$ to be used for the verification of members and sections is inconvenient for the design, if masses are determined from the results of the analysis for live loads. There are two ways to implement this facility:

- The masses will be assigned without an analysis for live loads, which is convenient if masses are lumped at the mass centre of rigid diaphragms along with their rotational mass moment of inertia, but inconvenient if masses are to be assigned to nodes in proportion to their tributary area, to automatically account for rotational mass moment of inertia or when the diaphragm is not considered as rigid.
- The masses will be assigned to nodes on the basis of separate analyses for live loads on groups of storeys with different reductions of live loads; this option may be unavoidable if different $\psi_{2,i}$ values are used in different storeys, but depends on the options available in the analysis program used.

Given that at the storey level the resultant of the nominal value of live loads, $Q_k$, normally does not exceed 25–30% that of permanent loads, $G_k$, and that percentage is multiplied further by the value of the $\psi_{2,i}$ factor (0.3 usually, 0.6 or 0.8 rarely), the designer may have to consider whether the overall economy effected by the further reduction of live loads in some storeys is worth the additional design effort.

## 4.5. Methods of analysis

### 4.5.1. Overview of the menu of analysis methods

*Clauses 4.3.3.1(1), 4.3.3.1(2), 4.3.3.1(3), 4.3.3.1(4)*

*Section 4* of EN 1998-1 provides the following analysis options for the design of buildings and for the evaluation of their seismic performance:

- linear static analysis (termed the 'lateral force' method of analysis in EN 1998-1, but often in practice called 'equivalent static' analysis)
- modal response spectrum analysis (also termed in practice 'linear dynamic' analysis, with the risk of being confused with linear time–history analysis)
- non-linear static analysis (commonly known as 'pushover' analysis)
- non-linear dynamic analysis (time–history or response–history analysis).

Linear time–history analysis is not explicitly mentioned as an alternative to linear modal response spectrum analysis.

Unlike US codes, which consider the linear static analysis as the reference method for the seismic design of buildings, Eurocode 8 gives this status to the modal response spectrum method. This analysis procedure is applicable for the design of buildings without any limitations.

*Clauses 3.2.2.5(2), 3.2.2.5(3), 4.3.4(1), 4.3.4(2), 4.3.4(3)*

The linear methods of analysis use the design response spectrum, which is essentially the elastic response spectrum with 5% damping divided by the behaviour factor $q$. Internal forces due to the seismic action are taken to be equal to those estimated from the linear analysis; however, and consistent with the equal displacement rule and the concept and use of the behaviour factor $q$, displacements due to the seismic action are taken as equal to those derived from the linear analysis, multiplied by the behaviour factor $q$. In contrast, when a non-linear analysis method is used, both internal forces and displacements due to the seismic action are taken to be equal to those derived from the non-linear analysis.

The use of a linear method of analysis does not imply that the seismic response will be linear elastic; it is simply a device for the simplification of practical design within the framework of force-based seismic design with the elastic spectrum divided by the behaviour factor $q$.

### 4.5.2. The lateral force method of analysis

#### *4.5.2.1. Introduction: the lateral force method versus modal response spectrum analysis*

In the lateral force method a linear static analysis of the structure is performed under a set of lateral forces applied separately in two orthogonal horizontal directions, $X$ and $Y$. The intent is to simulate through these forces the peak inertia loads induced by the horizontal component of the seismic action in the two directions, $X$ or $Y$. Owing to the familiarity and experience of structural engineers with elastic analysis for static loads (due to gravity, wind

or other static actions), this method has long been – and still is – the workhorse for practical seismic design. The version of the method in Eurocode 8 has been tuned to give similar results for storey shears – considered as the fundamental seismic action effects – as those from modal response spectrum analysis (which is the reference method), at least for the type of structures to which the lateral force method is considered applicable.

For the type of structures where both the lateral force method and modal response spectrum analysis are applicable, the latter gives, on average, a slightly more even distribution of peak internal forces in different critical sections, such as the two ends of the same beam or column. These effects are translated to some savings in materials. Despite such savings, the overall inelastic performance of a structure is normally better if its members are dimensioned for the results of a modal response spectrum analysis, instead of the lateral force method. The better performance is attributed to closer agreement of the distribution of peak inelastic deformations in the non-linear response to the predictions of the elastic modal response spectrum analysis than to those of the lateral force approach.

As the use of modal response spectrum analysis is not subject to any constraints of applicability, it can be adopted by a designer who wishes to master the method as the single analysis tool for seismic design in 3D. In addition to this convenience, modal response spectrum analysis is more rigorous (e.g. unlike the lateral force method, it gives results independent of the choice of the two orthogonal directions, $X$ and $Y$, of application of the horizontal components of the seismic action), and offers a better overall balance of economy and safety. So, with today's availability of reliable and efficient computer programs for modal response spectrum analysis of structures in 3D, and with the gradual establishment of structural dynamics as a core subject in structural engineering curricula and continuing education programmes in seismic regions of the world, it is expected that modal response spectrum analysis will grow in application and prevail in the long run. Even then, though, the lateral force method of analysis will still be relevant, due to its intuitive appeal and conceptual simplicity.

#### *4.5.2.2. Applicability conditions*

The fundamental assumptions underlying the lateral force procedure are that: *Clause 4.3.3.2.1*

(1) the response is governed by the first translational mode in the horizontal direction in which the analysis is performed
(2) a simple approximation of the shape of that mode is possible, without any calculations.

*Section 4* of EN 1998-1 allows the use of the lateral force procedure only when both of the following conditions are met:

(a) The fundamental period of the building is shorter than 2 s and four times the transition period $T_C$ between the constant spectral acceleration and the constant spectral pseudo-velocity regions of the elastic response spectrum.
(b) The building structure is characterized as regular in elevation, according to the criteria set out in Section 4.3.3.1.

If the condition (a) is not met, the second and/or third modes may contribute significantly to the response in comparison to the fundamental one, despite their normally lower participation factors and participating masses: at periods longer than 2 s or $4T_C$, spectral values are low, while, when the fundamental period is that long, the second and/or third mode periods may fall within or close to the constant spectral acceleration plateau where spectral values are highest. Under these circumstances, accounting for higher modes through a modal response spectrum analysis is essential.

In structures that are not regular in elevation the effects of higher modes may be significant locally, i.e. near elevations of discontinuity or abrupt changes, although they may not be important for the global response, as this is determined by the base shear and overturning moment. A more important reason for this condition, though, is that the

common and simple approximation of the first mode shape may not be applicable when there are irregularities in elevation.

Only condition (a) above is explicitly required to be met in both horizontal directions for the lateral force procedure to be applicable. In principle, a structure that is characterized as regular in elevation in only one of the two directions may be subjected to lateral force analysis in that direction and to modal response spectrum analysis in the other, especially if the structure is analysed with a separate 2D model in each of these two directions. However, it is very unlikely that this is a practical design option. So, in practice, both conditions have to be met in both horizontal directions for the lateral force procedure to be applicable.

#### 4.5.2.3. Base shear

*Clause 4.3.3.2.2(1)*

The base shear is derived separately in the two horizontal directions in which the structure is analysed, on the basis of the first translational mode in that horizontal direction:

$$F_b = \lambda m S_d(T_1) \tag{D4.5}$$

where $S_d(T_1)$ is the value of the design spectrum at the fundamental period $T_1$ in the horizontal direction considered and $\lambda m$ is the effective modal mass of the first (fundamental) mode, expressed as a fraction $\lambda$ of the total mass, $m$, of the building above the foundation or above the top of a rigid basement. If the building has more than two storeys and a fundamental period $T_1$ shorter than $2T_C$ (with $T_C$ denoting again the transition period between the constant spectral acceleration and the constant spectral pseudo-velocity ranges), $\lambda = 0.85$. In buildings with just two storeys, practically the full mass participates in the first mode, and $\lambda = 1.0$; the same $\lambda$ value is used if $T_1 > 2T_C$, to account for the increased importance of the second (and of higher) modes. The aim of the introduction of the $\lambda$ factor is to emulate the modal response spectrum analysis method, at least as far as the global seismic action effects are concerned (base shear and overturning moment).

#### 4.5.2.4. Estimation of the fundamental period $T_1$

*Clause 4.3.3.2.2(2)*

Eurocode 8 encourages estimation of the fundamental period $T_1$ through methods based on mechanics. A fairly accurate such estimate of $T_1$ is provided by the Rayleigh quotient:

$$T_1 = 2\pi\sqrt{\frac{\sum_i m_i \delta_i^2}{\sum F_i \delta_i}} \tag{D4.6}$$

where $\delta_i$ denotes the lateral drift at degree of freedom $i$ from an elastic analysis of the structure under a set of lateral forces $F_i$ applied to the degrees of freedom of the system. Both $F_i$ and $\delta_i$ are taken in the horizontal direction, $X$ or $Y$, in which $T_1$ is sought. For a given pattern (i.e. distribution) of the forces $F_i$ over the degrees of freedom $i$, the drifts $\delta_i$ are proportional to $F_i$, and the outcome of equation (D4.6) is independent of the absolute magnitudes of $F_i$. As equation (D4.6) is also rather insensitive to the distribution of these forces to the degrees of freedom $i$, any reasonable distribution of $F_i$ may be used. It is both convenient and most accurate to use as $F_i$ the lateral forces corresponding to the distribution of the total base shear of equation (D4.5) to the degrees of freedom, $i$, postulated in the lateral force method of analysis (see Section 4.5.2.5 and equation (D4.7)). As at this stage the value of the design base shear is still unknown, the magnitude of lateral forces $F_i$ can be such that their resultant base shear is equal to the total weight of the structure, i.e. as if $\lambda S_d(T_1)$ is equal to $1.0g$. Then, a single linear static analysis for each horizontal direction, $X$ or $Y$, is used both for (1) the estimation of $T_1$ from equation (D4.6), and (2) for the calculation of the effects of the horizontal component of the seismic action in that direction. The seismic action effects from this analysis are multiplied by the value of $\lambda S_d(T_1)$ determined from the design spectrum for the now known natural period $T_1$ and used as the horizontal seismic action effects, $A_X$ or $A_Y$.

Eurocode 8 also allows the use in equation (D4.5) of values of $T_1$ estimated through empirical expressions – mostly adopted from the SEAOC '99 requirements.[40] For $T_1$ in seconds and all other dimensions in metres: *Clauses 4.3.3.2.2(3), 4.3.3.2.2(4)*

- $T_1 = 0.085H^{3/4}$, for steel moment frame buildings less than 40 m tall
- $T_1 = 0.075H^{3/4}$, for buildings less than 40 m tall with concrete frames or with steel frames with eccentric bracings
- $T_1 = 0.05H^{3/4}$ for buildings less than 40 m tall with any other type of structural system (including concrete wall buildings)
- $T_1 = 0.075/[\sum A_{wi}(0.2 + (l_{wi}/H))^2]^{1/2}$, in buildings with concrete or masonry walls

where $H$ denotes the total height of the building from the base or above the top of a rigid basement, and $A_{wi}$ and $l_{wi}$ denote the horizontal cross-sectional area and the length of wall $i$, with the summation extending over all ground storey walls $i$ parallel to the direction in which $T_1$ is estimated.

Such expressions represent lower (mean minus one standard deviation) bounds to values inferred from measurements on buildings in California in past earthquakes. As such measurements include the effects of non-structural elements on the response, these empirical expressions give lower estimates of the period than equation (D4.6). They are used because they give conservative estimates of $S_d(T_1)$ – usually in the constant spectral acceleration plateau – for force-based design. Being derived from a high-seismicity region, these expressions are even more conservative for use in moderate- or low-seismicity areas, where structures have lower required earthquake resistance and hence are less stiff. Moreover, as estimation of $T_1$ from equation (D4.6) is quite accurate and requires limited extra calculations (only application of equation (D4.6) to the results of the linear static analysis anyway performed for the lateral force analysis), there is no real reason to resort to the use of empirical expressions.

The use of a period calculated from mechanics, regardless of how its value compares to the empirical value, as well as the introduction of the $\lambda$ factor in equation (D4.5), show that Eurocode 8 tries to bring the results of the lateral force method closer to those of modal response spectrum analysis, and not the other way around as US codes do.

#### 4.5.2.5. *Lateral force pattern*

To translate the peak base shear from equation (D4.5) into a set of lateral inertia forces in the same direction (i.e. that of the horizontal component of the seismic action) applied to the degrees of freedom, $i$, of the structure, a distribution with height, $z$, of the peak lateral drifts in the same direction is assumed, $\Phi(z)$. Then, as in a single mode of vibration the peak lateral inertia force for the degree of freedom $i$ is proportional to $\Phi(z_i)m_i$, where $m_i$ is the mass associated with that degree of freedom, and the base shear from equation (D4.5), $F_b$, is distributed to the degrees of freedom as follows: *Clause 4.3.3.2.3*

$$F_i = F_b \frac{\Phi_i m_i}{\sum_j \Phi_j m_j} \tag{D4.7}$$

where the summation in the denominator extends over all degrees of freedom.

Within the field of application of the lateral force method (higher modes unimportant, structures regular in elevation) and in the spirit of the simplicity of the approach, the first-mode drift pattern is normally taken as proportional to elevation, $z$, from the base or above the top of a rigid basement, i.e. $\Phi_i = z_i$. Moreover, although the presentation above is general, for any arrangement of the masses and degrees of freedom in space, for buildings with floors acting as rigid diaphragms the discretization in equation (D4.7) refers to floors or storeys (index $i$, with $i = 1$ at the lowest floor and $i = \eta$ at the roof) and lateral forces $F_i$ are applied at the floor centres of mass.

The result of equation (D4.7) for $\Phi_i = z_i$ is commonly termed the 'inverted triangular' pattern of lateral forces (although in reality it is just the drifts that have an 'inverted triangular' distribution, and the pattern of forces depends also on the distribution of masses, $m_i$).

## 4.5.3. Modal response spectrum analysis

### *4.5.3.1. Modal analysis and its results*

*Clause 4.3.3.3*

Unlike linear static analysis, designers may not be so familiar with linear dynamic analysis of the modal response spectrum type. Moreover, some commercial computer programs with modal response spectrum analysis capability may not perform such an analysis in accordance with the relevant requirements of Eurocode 8. For instance, along the line of other seismic design codes (e.g. some US codes), a program may use the modal response spectrum method just to estimate peak inertia forces at storey levels, and to then apply these forces as static forces and calculate the static response to them as in the lateral force method. For these reasons, an overview is given below of how modal response spectrum analysis should be performed to fulfil the letter and spirit of EN 1998-1.

The first step in a modal response spectrum analysis is the determination of the 3D modal shapes and natural frequencies of vibration (eigenmodes and eigenvalues). Today, this task can be performed very reliably and efficiently by many computer programs dedicated to seismic response analysis for the purposes of earthquake-resistant design.

Even when the building qualifies for two separate 2D analyses in two orthogonal horizontal directions, $X$ and $Y$, it is preferable to do the modal response spectrum analysis on a full 3D structural model. Then, each mode shape, represented by vector $\boldsymbol{\Phi}_n$ for mode $n$, will in general have displacement and rotation components in all three directions, $X$, $Y$ and $Z$. In other words, vector $\boldsymbol{\Phi}_n$ will in general include all degrees of freedom of the structural model (unless the solution of the eigenvalue problem has been based on a few degrees of freedom, with the rest condensed out, statically or dynamically – see below).

If the origin of the global coordinate system, $X$–$Y$–$Z$, is far from the masses of the structure, the accuracy of an eigenmode–eigenvalue analysis in 3D might be adversely affected. Although most widely used computer programs take this into account, the designer should ideally choose the origin of the axes to be inside the volume of the structure.

The outcome of the eigenmode–eigenvalue analysis necessary for the subsequent estimation of the peak elastic response on the basis of the response spectra in the three directions, $X$, $Y$ and $Z$, comprises for each normal mode, $n$:

- The natural circular frequency, $\omega_n$, and the corresponding natural period, $T_n = 2\pi/\omega_n$.
- The mode shape, represented by vector $\boldsymbol{\Phi}_n$.
- The modal participation factors $\Gamma_{Xn}$, $\Gamma_{Yn}$, $\Gamma_{Zn}$ in response to the component of the seismic action in direction $X$, $Y$ or $Z$, computed as

$$\Gamma_{Xn} = \frac{\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{M}\boldsymbol{I}_X}{\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{M}\boldsymbol{\Phi}_n} = \frac{\sum_i \varphi_{Xi,n} m_{Xi}}{\sum_i (\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})}$$

  where $i$ denotes the nodes of the structure associated with dynamic degrees of freedom, $\mathbf{M}$ is the mass matrix, $\boldsymbol{I}_X$ is a vector with elements equal to 1 for the translational degrees of degrees of freedom parallel to direction $X$ and with all other elements equal to 0, $\varphi_{Xi,n}$ is the element of $\boldsymbol{\Phi}_n$ corresponding to the translational degree of freedom of node $i$ parallel to direction $X$ and $m_{Xi}$ is the associated element of the mass matrix (similarly for $\varphi_{Yi,n}$, $\varphi_{Zi,n}$, $m_{Yi}$ and $m_{Zi}$). If $\mathbf{M}$ contains rotational mass moments of inertia, $I_{\theta Xi}$, $I_{\theta iY}$, $I_{\theta Zi}$, the associated terms also appear in the sum of the denominator. $\Gamma_{Yn}$, $\Gamma_{Zn}$ are defined similarly.
- The effective modal masses in directions $X$, $Y$ and $Z$, $M_{Xn}$, $M_{Yn}$, and $M_{Zn}$, respectively, computed as

$$M_{Xn} = \frac{(\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{M}\boldsymbol{I}_X)^2}{\boldsymbol{\Phi}_n^{\mathrm{T}}\mathbf{M}\boldsymbol{\Phi}_n} = \frac{\left(\sum_i \varphi_{Xi,n} m_{Xi}\right)^2}{\sum_i (\varphi_{Xi,n}^2 m_{Xi} + \varphi_{Yi,n}^2 m_{Yi} + \varphi_{Zi,n}^2 m_{Zi})}$$

  and similarly for $M_{Yn}$, $M_{Zn}$. These are essentially base-shear-effective modal masses,

because the reaction force (base shear) in direction $X$, $Y$ or $Z$ due to mode $n$ are equal to $F_{bX,n} = S_a(T_n)M_{Xn}$, $F_{bY,n} = S_a(T_n)M_{Yn}$ and $F_{bZ,n} = S_a(T_n)M_{Zn}$, respectively. The sum of the effective modal masses in $X$, $Y$ or $Z$ over all modes of the structure (not just the $N$ modes taken into account) is equal to the total mass of the structure.

The first objective of a modal response spectrum analysis is to determine the peak value of any seismic action effect of interest (be it a global effect, such as the base shear, or local ones, such as member internal forces, or even intermediate ones, such as interstorey drifts) in every one of the $N$ modes considered due to the seismic action component in direction $X$, $Y$ or $Z$. This may be accomplished through different approaches in different computer programs. A simple and efficient approach is the following:

- For each normal mode $n$, the spectral displacement, $S_{dX}(T_n)$, is calculated from the design (pseudo-)acceleration spectrum of the seismic action component of interest, say $X$, as $(T_n/2\pi)^2 S_{aX}(T_n)$.
- The nodal displacement vector of the structure in mode $n$ due to the seismic action component of interest, say in direction $X$, $\boldsymbol{U}_{Xn}$, is computed as the product of the spectral displacement, $S_{dX}(T_n)$, the participation factor of mode $n$ to the response to the seismic action component of interest, $\Gamma_{Xn}$ for the component in direction $X$, and the eigenvector, $\boldsymbol{\Phi}_n$, of the mode: $\boldsymbol{U}_{Xn} = (T_n/2\pi)^2 S_{aX}(T_n)\Gamma_{Xn}\boldsymbol{\Phi}_n$.
- Peak modal values of the effects of the seismic action component of interest are computed from the modal displacement vector determined in the previous step: deformations of members (e.g. chord rotations) or of storeys (e.g. interstorey drifts) are computed directly from the nodal displacement vector of the mode $n$; member modal end forces are computed by multiplying the member modal deformations (e.g. chord rotations) by the member stiffness matrix, as in the back-substitution phase of the solution of a static analysis problem; modal storey shears or overturning moments, etc., are determined from modal member shears, moments, axial forces, etc., through equilibrium, etc.

The peak modal responses obtained as above are exact. However, they can only be combined approximately, as they occur at different instances of the response. Appropriate rules for the combination of peak modal responses are described in Section 4.5.3.3. Rules for taking into account, at different levels of approximation, the simultaneous occurrence of the seismic action components are given in Section 4.9.

For buildings with horizontal slabs considered to act as rigid diaphragms, and provided that the vertical component of the seismic action is not of interest or importance for the design, static and dynamic condensation techniques are sometimes applied to reduce the number of static degrees of freedom to just three dynamic degrees of freedom per floor (two horizontal translations and one rotation about the vertical axis). Dynamic condensation profits from the small inertia forces normally associated with vertical translations and nodal rotations about the horizontal axis due to the horizontal components of the seismic action. The reduced dynamic model in 3D has just $3n_{st}$ normal modes, where $n_{st}$ is the number of storeys. For each normal mode $n$, the response spectrum is entered with the natural period $T_n$ of the mode, to determine the corresponding spectral acceleration $S_a(T_n)$. Then, for each one of the two horizontal components of the seismic action, two horizontal forces and one torque component with respect to the vertical axis are computed for normal mode $n$ and at each floor level $i$: $F_{Xi,n}$, $F_{Yi,n}$ and $M_{i,n}$, where the indexes $X$ and $Y$ now denote the direction of the two forces and not that of the seismic action component (which may be either $X$ or $Y$). These forces and moments are computed as the product of:

- the participation factor of the normal mode $n$ to the response to the seismic action component of interest, say $\Gamma_{Xn}$ for the seismic action component in direction $X$
- the mass associated with the corresponding floor degrees of freedom – floor mass $m_{Xi} = m_{Yi}$ and floor rotational mass moment of inertia, $I_{\theta i}$
- the corresponding component of the modal eigenvector, $\varphi_{Xi,n}$, $\varphi_{Yi,n}$, $\varphi_{\theta i,n}$
- $S_a(T_n)$.

For each mode $n$ and separately for the two horizontal components of the seismic action, a static analysis of the full static model in 3D of the structure is then performed, under static forces and moments $F_{Xi,n}$, $F_{Yi,n}$ and $M_{i,n}$, applied to the corresponding dynamic degrees of freedom of each floor $i$. Peak modal response quantities, like nodal displacements, member internal forces, member deformations (chord rotations) or interstorey drifts, etc., are computed separately for each mode and combined for all modes according to the rules in Section 4.5.3.3 for each horizontal component $X$ or $Y$ of the seismic action.

The approach of the previous paragraph is not feasible for structures without rigid diaphragms at storey levels, and cannot be used when the vertical component of the seismic action is of interest. Moreover, with today's hardware the savings in computer time and memory are not worth the complexity in analysis software for the reduction of the static degrees of freedom into a much smaller number of dynamic degrees of freedom at floor levels.

In closing this relatively long account of modal analysis, it is noted that modal participation factors and effective modal masses are more than mathematical quantities internally used in the procedure: they convey a certain physical meaning, which is essential for the understanding of the nature and relative importance of each mode. For instance, the relative magnitude of the modal participation factors or of the effective modal masses determines the predominant direction of the mode: the inclination of this direction to the horizontal direction $X$ is equal to $\Gamma_{Xn}/\Gamma_{Yn}$, etc.; the predominant direction of the mode with the largest modal base shear, along with the orthogonal direction, is a good choice for the often ill-defined 'principal' or 'main' directions of the structure in plan, along which the horizontal components of the seismic action should be taken to act. Unfortunately, the presence of torsion in a mode cannot be appreciated on the basis of modal participation factors and effective modal masses defined along the three directions, $X$, $Y$ and $Z$: participation factors and modal masses for rotation about these axes would be necessary for that purpose, and such quantities are normally not reported in the output of computer programs. The importance of torsion in a mode may be judged, instead, on the basis of the modal reaction forces and moments. Last but not least, irrespective of the qualitative criteria for regularity in plan, a good measure of such regularity is the lack of significant rotation about the vertical axis (and global reaction torque with respect to that axis) in the (few) lower modes.

#### *4.5.3.2. Minimum number of modes to be taken into account*

*Clauses 4.3.3.3.1(2), 4.3.3.3.1(3), 4.3.3.3.1(4)*

All modes of vibration that contribute significantly to the response quantities of interest should normally be taken into account. However, as the number of modes to be considered should be specified as input to the eigenvalue analysis, a generally applicable and simple criterion should be adopted. Such a criterion can only be based on global response quantities. The most commonly used criterion, adopted by Eurocode 8, requires that the $N$ modes taken into account provide together a total effective modal mass along any one of the seismic action components, $X$, $Y$ or even $Z$, considered in design, of at least equal to 90% of the total mass of the structure.

As an alternative, in case the criterion above turns out to be difficult to satisfy, the eigenvalue analysis should take into account all modes with effective modal mass along any individual seismic action component, $X$, $Y$ or $Z$, considered in design, of greater than 5% of the total. It is obvious, though, that this criterion is hard to apply, as it refers to modes that have not been captured so far by the eigenvalue analysis.

*Clause 4.3.3.3.1(5)*

As a third alternative for very difficult cases (e.g. in buildings with a significant contribution from torsional modes, or when the seismic action components in the vertical direction, $Z$, should be considered in the design), the minimum number $N$ of modes to be taken into account should be at least equal to $3\sqrt{n_{st}}$ (where $n_{st}$ is the number of storeys above the foundation or the top of a rigid basement) and should be such that the shortest natural period captured does not exceed 0.2 s. It is clear from the wording of the code that recourse to the third alternative can be made only if it is demonstrated that it is not feasible to meet any of the two criteria above.

The most commonly used criterion, requiring a sum of effective modal masses along each individual seismic action component, $X$, $Y$ or $Z$, considered in design, of at least 90% of the total mass, addresses only the magnitude of the base shear captured by the modes taken into account, and even that only partly: modal shears are equal to the product of the effective modal mass and the spectral acceleration at the natural period of the mode; so, if the fundamental period is well down the tail of the (pseudo-)acceleration spectrum and higher mode periods are in the constant (pseudo-)acceleration plateau, the effective modal mass alone underplays the importance of higher modes for the base shear. Other global response quantities, such as the overturning moment at the base and the top displacement, are even less sensitive to the number of modes than the base shear. However, estimation of global response quantities is less sensitive to the number of modes considered than that of local measures, such as the interstorey drift, the shear at an upper storey, or the member internal forces. As important steps of the seismic design process, such as member dimensioning for the ultimate limit state are based on seismic action effects from the analysis at the local (i.e. member) level, the modes considered in the eigenvalue analysis should preferably account for much more than 90% of the total mass (close to 100%), to approximate with sufficient accuracy the peak dynamic response at that level.

There exist techniques to approximately account for the missing mass due to truncation of higher modes (e.g. by adding static response). Unlike some other codes, including EN 1998-2 (on bridges), EN 1998-1 does not require such measures.

#### *4.5.3.3. Combination of modal responses*

Within the response spectrum method of analysis, the elastic responses to two different vibration modes are often taken as independent of each other. The magnitude of the correlation between modes $i$ and $j$ is expressed through the correlation coefficient of these two modes, $r_{ij}$:[42,43] *Clause 4.3.3.3.2(1)*

$$r_{ij} = \frac{8\sqrt{\xi_i \xi_j}(\xi_i + \rho\xi_j)\rho^{3/2}}{(1-\rho^2)^2 + 4\xi_i\xi_j\rho(1+\rho^2) + 4(\xi_i^2 + \xi_j^2)\rho^2} \tag{D4.8}$$

where $\rho = T_i/T_j$, and $\xi_i$ and $\xi_j$ are the viscous damping ratios assigned to modes $i$ and $j$, respectively. If two vibration modes have closely spaced natural periods (i.e. if $\rho$ is close to unity), the value of the correlation coefficient is also close to unity, and the responses in these two modes cannot be taken as independent of each other. For buildings, EN 1998-1 considers that two modes $i$ and $j$ cannot be taken as being independent of each other if the ratio of the minimum to the maximum of their periods, $\rho$, is between 0.9 and 1/0.9; for the two extreme values of this range of $\rho$ and $\xi_i = \xi_j = 0.05$, equation (D4.8) gives $r_{ij} = 0.47$. (EN 1998-2 on bridges is more restrictive, considering that two modes $i$ and $j$ are not independent if the value of the ratio $\rho$ of their periods is between $1 + 10(\xi_i\xi_j)^{1/2}$ and $0.1/[0.1 + (\xi_i\xi_j)^{1/2}]$; for $\xi_i = \xi_j = 0.05$ and $\rho$ equal to these limit values, equation (D4.8) gives $r_{ij} = 0.05$.) It is noted that in buildings with similar structural configuration and earthquake resistance in two horizontal directions, $X$ and $Y$, pairs of natural modes with very similar natural periods at about 90° in plan (often not in the two horizontal directions, $X$ and $Y$) are quite common; the two modes in each pair are not independent but closely correlated.

If all relevant modal responses may be regarded as independent of each other, then the most likely maximum value $E_E$ of a seismic action effect may be taken equal to the square root of the sum of squares of the modal responses (SRSS rule[44]): *Clause 4.3.3.3.2(2)*

$$E_E = \sqrt{\sum_N E_{Ei}^2} \tag{D4.9}$$

where the summation extends over the $N$ modes taken into account and $E_{Ei}$ is the peak value of this seismic action effect due to vibration mode $i$.

If the response in any two vibration modes $i$ and $j$ cannot be taken as independent of each other, Eurocode 8 requires that more accurate procedures for the combination of modal maximum responses are used, giving the complete quadratic combination (CQC rule[42]) as an *Clause 4.3.3.3.2(3)*

example. According to this rule, the most likely maximum value $E_E$ of a seismic action effect may be taken as equal to

$$E_E = \sqrt{\sum_{i=1}^{N}\sum_{j=1}^{N} r_{ij} E_{Ei} E_{Ej}} \qquad \text{(D4.10)}$$

where $r_{ij}$ is the correlation coefficient of modes $i$ and $j$ given by equation (D4.8) and $E_{Ei}$ and $E_{Ej}$ are the peak values of the seismic action effect due to vibration modes $i$ or $j$, respectively. Comparison with the results of response–history analyses has demonstrated the accuracy of the CQC rule, in cases where the SRSS rule has been found to be unconservative due to mode correlation.

The SRSS rule, equation (D4.9), is a special case of equation (D4.10) for $r_{ij} = 0$ if $i \neq j$ (obviously $r_{ij} = 1$ for $i = j$). As in computer programs with capabilities of eigenvalue and response spectrum analysis the additional complexity of equation (D4.10) is not an issue, there is no reason to implement in such a program the simpler equation (D4.9) instead of the more general and always accurate and acceptable one, equation (D4.10).

### 4.5.4. Linear analysis for the vertical component of the seismic action

*Clause 4.3.3.5.2(1)*

In buildings the vertical component of the seismic action may in general be neglected, because:

- its effects are normally covered by the design for the persistent and transient design situation, which involves the permanent actions (dead loads) and the imposed ones (live loads), both multiplied by partial factors for actions, which are normally significantly greater than 1.0
- except when a building has beams with long span and significant mass along the span, the fundamental period of vibration in the vertical direction is controlled by the axial stiffness of vertical members and is short, therefore spectral amplification of the vertical ground motion is small.

Eurocode 8 requires taking into account the vertical component of the seismic action only when its effects are likely to be significant, in view of the two arguments above against this likelihood. This is considered to be the case when both of the following conditions are met:

(1) The design peak vertical acceleration of the ground, $a_{vg}$, exceeds $0.25g$.
(2) The building or the structural member falls in one of the following categories:
   (a) the building is base-isolated
   (b) the structural member being designed is (nearly) horizontal (i.e. a beam, a girder or a slab) and
      - spans at least 20 m or
      - cantilevers over more than 5 m or
      - consists of prestressed concrete or
      - supports one or more columns at intermediate points along its span.

*Clauses 4.3.3.5.2(2), 4.3.3.5.2(3)*

In the cases listed in condition 2(b), the dynamic response to the vertical component is often of local nature, e.g. it involves the horizontal elements for which the vertical component needs to be taken into account, as well as their immediately adjacent or supporting elements, but not the structure as a whole. For this reason, Eurocode 8 permits analysis on a partial structural model that captures the important aspects of the response in the vertical direction without irrelevant and unimportant influences that confuse and obscure the important results. The partial model will include fully the elements on which the vertical component is considered to act (those listed above) and their directly associated supporting elements or substructures, while all other adjacent elements (e.g. adjacent spans) may be included only with their stiffness.

## 4.5.5. Non-linear methods of analysis

### *4.5.5.1. Introduction: field of application*

The primary use of non-linear methods of analysis within the framework of Eurocode 8 is to evaluate the seismic performance of new designs, or to assess existing or retrofitted buildings. In fact, in EN 1998-3 (on the assessment and retrofitting of buildings) the reference analysis methods are the non-linear ones. In the context of EN 1998-1, non-linear methods are limited to:

*Clauses 4.3.3.1(4), 4.3.3.4.2.1(1)*

- the detailed evaluation of the seismic performance of a new building design (including confirmation of the intended plastic mechanisms and of the distribution and extent of damage)
- the design of buildings with seismic isolation, for which application of linear analysis methods is allowed under fairly restrictive conditions, and non-linear methods are the reference for the analysis.

Specifically, for the non-linear static (pushover) method of analysis, EN 1998-1 defines two additional uses:

- To verify or revise the value of the factor $\alpha_u/\alpha_1$ incorporated in the basic or reference value $q_0$ of the behaviour factor of concrete, steel or composite buildings, to account for overstrength due to redundancy of the structural system (cf. Section 5.5 and Fig. 5.2).
- To design buildings on the basis of a non-linear static analysis and deformation-based verification of its ductile members, instead of force-based design with linear elastic analysis and the design spectrum that incorporates the behaviour factor $q$. In this case, the seismic action is defined in terms of the target displacement – derived from the elastic spectrum with 5% damping as described in Section 4.5.5.2 – instead of the design spectrum.

The introduction of 'pushover' analysis for the direct codified design of buildings is a novelty of Eurocode 8. As there is no precedent in the world, and available design experience is not sufficient to judge the implications of this bold step, countries are allowed to restrict, or even forbid, through their National Annex, the use of non-linear analysis methods for purposes other than the design of buildings with seismic isolation.

### *4.5.5.2. Non-linear static ('pushover') analysis*

Unlike (a) linear elastic analysis of the lateral force or modal response spectrum type, which has long been the basis for codified seismic design of new structures, and (b) non-linear dynamic (response time–history) analysis, which has been extensively used since the 1970s for research, code calibration or other special purposes, non-linear static ('pushover') analysis was not a widely known or used method until important guidance documents emerged in the USA[45,46] in response to the pressing need for practical and cost-efficient procedures for the seismic assessment and retrofitting of existing buildings. Since then, due to its appealing simplicity and intuitiveness and the wide availability of the necessary computer programs, pushover analysis has become the analysis method of choice in the everyday seismic assessment practice of existing buildings.

*Clause 4.3.3.4.2*

Pushover analysis is non-linear static approach carried out under constant gravity loads and monotonically increasing lateral forces, applied at the location of the masses in the structural model to simulate the inertia forces induced by a single horizontal component of the seismic action. As the applied lateral forces are not fixed but increase monotonically, the method can describe the evolution of the expected plastic mechanism(s) and of structural damage, as a function of the magnitude of the imposed lateral loads and of the associated horizontal displacements.

*Clauses 4.3.3.4.2.1(1), 4.3.3.4.2.2(2), 4.3.3.5.2(5)*

The method is essentially the extension of the lateral force method of linear analysis into the non-linear regime. As such, it addresses only the horizontal component(s) of the seismic action and cannot treat the vertical component at all.

**Lateral force patterns**

*Clause 4.3.3.4.2.2(1)*

Pushover analysis was developed initially for 2D analyses, and this is how it is still mainly applied today. Even in applications to 3D structural models, the applied lateral forces simulate the inertia due to a single horizontal component of the seismic action: the forces $F_i$ applied to masses $m_i$ in the course of the pushover analysis are taken to remain proportional to a certain pattern of horizontal displacements $\Phi_i$:

$$F_i = \alpha m_i \Phi_i \qquad \text{(D4.11)}$$

According to Eurocode 8, pushover analyses should be performed using both of the following lateral load patterns:

(1) A 'uniform pattern', corresponding to uniform unidirectional lateral accelerations, i.e. to $\Phi_i = 1$ in equation (D4.11).
(2) A 'modal pattern', which depends on the type of linear analysis applicable to the particular structure:
   - If the building satisfies the conditions for the application of lateral force analysis method, an 'inverted triangular' unidirectional force pattern, similar to the one used in that method (i.e. $\Phi_i = z_i$ in equation (D4.11)).
   - If the building does not meet the conditions for the application of lateral force analysis, a pattern simulating the peak inertia forces of the fundamental mode in the horizontal direction in which the analysis is performed. Although Eurocode 8 is not very specific in this respect, the meaning is that $\Phi_i$ in equation (D4.11) should follow the fundamental mode shape as determined from a modal analysis. If this mode is not purely translational, the pattern of $\Phi_i$ and of the lateral forces $F_i$ will not be unidirectional anymore: it may have horizontal components orthogonal to that of the considered seismic action component.

*Clauses 4.3.3.4.2.4, 4.3.3.4.1(7)*

The most unfavourable result of the pushover analyses using the two standard lateral force patterns (the 'uniform' and the 'modal' pattern) should be adopted. Moreover, unless there is perfect symmetry with respect to an axis orthogonal to that of the seismic action component considered, each lateral force pattern should be applied in both the positive and the negative directions (sense), and the result to be used should be the most unfavourable one from the two analyses.

**Capacity curve**

*Clause 4.3.3.4.2.3*

A key outcome of the pushover analysis is the 'capacity curve', i.e. the relation between the base shear force, $F_b$, and a representative lateral displacement of the structure, $d_n$. That displacement is often taken at a certain node $n$ of the structural model, termed the 'control node'. The control node is normally at the roof level, usually at the centre of mass there. The pushover analysis has to extend at least up to the point on the capacity curve with a displacement equal to 1.5 times the 'target displacement', which defines the demand due to the seismic action component of interest. The inelastic deformations and forces in the structure from the pushover analysis at the time the target displacement is attained are taken as the demands at the local level due to the horizontal component of the design seismic action in the direction in which the pushover analysis is performed.

Although it is physically appealing to express the capacity curve in terms of the base shear force and of the roof displacement, a mathematically better choice that relates very well to the definition of the seismic demand in terms of spectral quantities is to present the capacity curve in terms of the lateral force and displacement of an equivalent single-degree-of-freedom (SDOF) system. The equivalent SDOF system, which is essential for the determination of seismic demand, is introduced below.

**Equivalent SDOF system for a postulated displacement pattern**

This section relates to informative *Annex B.2* of EN 1998-1.

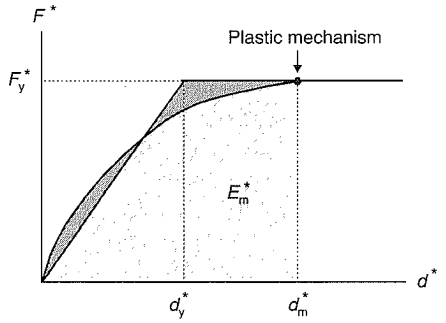



**Fig. 4.2.** Elastic–perfectly plastic idealization of the capacity curve of an equivalent SDOF system in pushover analysis[47]

The equivalent SDOF for pushover analysis is derived via the N2 procedure in Fajfar,[47] given in informative *Annex B* of EN 1998-1.

The horizontal displacements $\Phi_i$ in equation (D4.11) are considered to be normalized, so that at the control node, $\Phi_n = 1$. The mass of the equivalent SDOF system $m^*$ is

$$m^* = \sum m_i \Phi_i \tag{D4.12}$$

and the force $F^*$ and displacement $d^*$ of the equivalent SDOF system are

$$F^* = \frac{F_b}{\Gamma} \tag{D4.13}$$

$$d^* = \frac{d_n}{\Gamma} \tag{D4.14}$$

where

$$\Gamma = \frac{m^*}{\sum m_i \Phi_i^2} \tag{D4.15}$$

It is clear from equations (D4.11) to (D4.15) that if $\Phi_i$ emulates the shape of a normal mode, then the transformation factor is the participation factor of that mode in the direction of application of the lateral forces.

**Elastic–perfectly plastic idealization of the capacity curve**

This section relates to informative *Annex B.3* of EN 1998-1.

For the determination of the seismic demand in terms of the 'target displacement', an estimate of the period $T^*$ of the equivalent SDOF system is necessary. According to Fajfar's N2 procedure, this period is determined on the basis of the elastic stiffness of an elastic–perfectly plastic curve fitting the capacity curve of the SDOF system. The yield force, $F_y^*$, of the elastic–perfectly plastic curve, taken also as the ultimate strength of the SDOF system, is equal to the value of the force $F^*$ at formation of a completely plastic mechanism. The elastic stiffness of the elastic–perfectly plastic curve is determined in such a way that the areas under the actual capacity curve and its elastic–perfectly plastic idealization up to formation of the plastic mechanism are equal (Fig. 4.2). This condition gives the following value for the yield displacement of the elastic–perfectly plastic SDOF system, $d_y^*$:

$$d_y^* = 2\left(d_m^* - \frac{E_m^*}{F_y^*}\right) \tag{D4.16}$$

where $d_m^*$ is the displacement of the equivalent SDOF system at formation of the plastic mechanism and $E_m{}^*$ the deformation energy under the actual capacity curve up to that point.

The only use of the values of the yield force, $F_y^*$, and of the yield displacement of the SDOF system, $d_y^*$, is for the estimation of the elastic stiffness as $F_y^*/d_y^*$. It is not essential to identify formation of the plastic mechanism on the capacity curve to determine the values of these two parameters; if a complete plastic mechanism does not develop between the target displacement and the terminal point of the capacity curve, $F_y^*$, $d_m^*$ and $E_m^*$ may be determined on the basis of that latter point.

**Period of the equivalent SDOF system**

This section relates to informative *Annex B.4* of EN 1998-1.

The period $T^*$ of the equivalent SDOF system is estimated as

$$T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}} = 2\pi\sqrt{\frac{m^* d_{ny}}{F_{by}}} \qquad \text{(D4.17)}$$

where $F_{by}$ and $d_{ny}$ are repectively the base shear and the control node displacement at the 'yield point' of the elastic–perfectly plastic SDOF system.

If the structure is indeed linear-elastic up to the yield point of the elastic–perfectly plastic SDOF system, the period obtained from equation (D4.17) is identical to the value computed through the Rayleigh quotient, equation (D4.6), on the basis of the results of a linear analysis for the same pattern of lateral forces used for the construction of the capacity curve. In other words, the fundamental period is invariant during the transformation of the 3D structure into an equivalent SDOF system.

**Target displacement**

*Clause 4.3.3.4.2.6*

This section relates to *clause 4.3.3.4.2.6* and informative *Annex B.5* of EN 1998-1.

Unlike linear elastic analysis of the lateral force or modal response spectrum type, or non-linear dynamic (response time–history) analysis, both of which readily yield the (maximum) value of the response quantities to a given earthquake (i.e. the seismic demands), pushover analysis yields only the capacity curve *per se*. The demand needs to be estimated separately. This is normally done in terms of the maximum displacement induced by the earthquake, either to the equivalent SDOF system or to the control node of the full structure; the displacement demand on either one of these is termed 'target displacement'.

The procedure adopted in Eurocode 8 for the estimation of the target displacement is that of the N2 method in Fajfar.[47] It is based on the equal displacement rule, appropriately modified for short-period structures. In this approach, the target displacement of the equivalent SDOF system with period $T^*$ determined from equation (D4.17) at the yield point of the elastic–perfectly plastic approximation to the capacity curve is determined directly from the 5%-damped elastic acceleration spectrum, $S_e(T)$, at period $T^*$, if $T^*$ is longer than the corner period, $T_C$, between the constant pseudo-acceleration and the constant pseudo-velocity parts of the elastic spectrum:

$$d_{et}^* = S_e(T^*)\left(\frac{T^*}{2\pi}\right)^2 \qquad \text{if } T \geq T_C \qquad \text{(D4.18)}$$

If $T^*$ is less than $T_C$ the target displacement is corrected on the basis of the $q$–$\mu$–$T$ relation proposed in Vidic *et al.*[4] and given by equations (D2.1) and (D2.2). Equation (D2.2) gives:

$$d_t^* = \frac{d_{et}^*}{q_u}\left(1+(q_u-1)\frac{T_C}{T^*}\right) \geq d_{et}^* \qquad \text{if } T < T_C \qquad \text{(D4.19)}$$

where $q_u$ is the ratio of $m^* S_e(T^*)$ to the yield strength $F_y^*$ in the elastic–perfectly plastic approximation to the capacity curve. Figure 4.3 depicts graphically how equations (D4.18) and (D4.19) work.

The displacement at the control $n$ that corresponds to the target displacement of the SDOF system is obtained by inverting equation (D4.14).

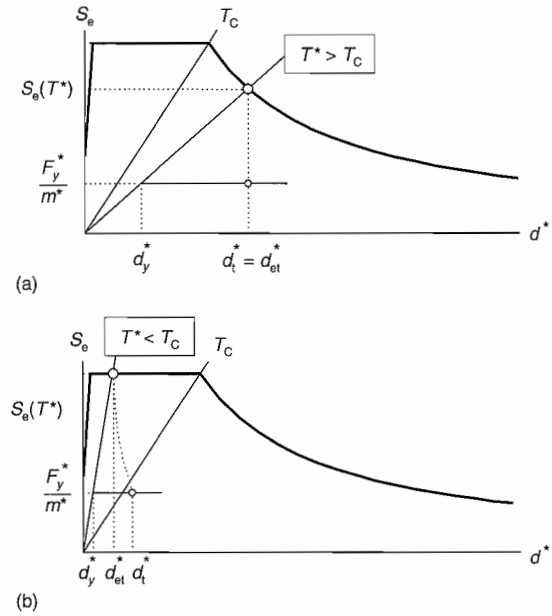



**Fig. 4.3.** Determination of the target displacement of an equivalent SDOF system in pushover analysis:[47] (a) long- and intermediate-period range; (b) short-period range

**Torsional effects in pushover analysis**

*Clauses 4.3.3.4.2.7(1), 4.3.3.4.2.7(2)*

As noted already, pushover analysis, as well as Fajfar's N2 procedure[47] adopted in EN 1998-1, have been developed for 2D analyses under a single component of the seismic action. It is clear from the above that the standard pushover analysis can capture the expected plastic mechanism(s) and the distribution and extent of damage only if, during the response, lateral inertial forces (represented by $F_i$) indeed follow the postulated pattern of horizontal displacements $\Phi_i$ according to equation (D4.11), as if the structure responds in a single normal mode described by equation (D4.11). The question may arise, then, to what extent the standard pushover analysis may be applied, if the response may be significantly affected by torsion in 3D and/or by higher-mode effects, and what corrections may be appropriate in such cases.

If the fundamental mode in, or close to, each one of the two orthogonal horizontal directions in which the pushover analysis is performed includes a torsional component, then the effects of this component on the response will most likely be captured if lateral forces $F_i$ are applied to nodes and the corresponding displacement pattern $\Phi_i$ in equation (D4.11) follows the modal shape of the corresponding fundamental mode. However, it has been found that if the first mode or the second mode in one of the two orthogonal horizontal directions is predominantly torsional, then standard pushover analysis may overestimate deflections on the flexible/weak side in plan (i.e. the one that develops larger horizontal displacements than the opposite side under static lateral forces parallel to it) and underestimate them on the opposite, stiff/strong, side. The difference in the prediction on the flexible/weak side is usually on the safe (conservative) side, and may be ignored. However, on the stiff/strong side the difference in the prediction may be on the unsafe side; according to Eurocode 8, it should be taken into account.

More specifically, this provision may be implemented as follows:[48,49]

(1) The standard pushover analysis is performed on the 3D structural model, with the unidirectional pattern of lateral forces, 'uniform' or 'modal', applied to the centres of mass of the floors.
(2) The equivalent SDOF system is established, along with its elastic–perfectly plastic approximation to its capacity curve; the target displacement of the equivalent SDOF

system is determined from the elastic response spectrum with 5% damping and is transformed into a displacement at the control $n$ at the centre of mass of the roof by inverting equation (D4.14).

(3) A modal response spectrum analysis of the same 3D structural model is performed. The displacement in the horizontal direction in which the pushover analysis has been performed is computed at all nodes of the roof (including the control node at the centre of mass there) through the SRSS (equation (D4.9)) or the CQC rule (equation (D4.10)), as appropriate, and divided by the corresponding value at the control node at the centre of mass, to give an 'amplification factor' that reflects the effect of torsion on the roof displacements.

(4) Wherever the amplification factor derived as in point 3 above is greater than 1.0, it is used to multiply the displacements of all nodes along the same vertical line, as these are obtained from the standard pushover analysis in points (1) and (2) above. The outcome is assumed to reflect, on one hand, the evolution of the global inelastic behaviour and its heightwise distribution as captured by the standard pushover analysis, and, on the other hand, the effect of global torsion on the planwise distribution of inelasticity. The restriction of the amplification factor being greater than 1.0 implies that de-amplification due to torsion is neglected, as non-linear response–history analyses have shown that the larger the extent and the magnitude of inelasticity, the smaller the effects of torsion on local response.

**Higher mode effects in pushover analysis**

As noted above, pushover analysis with a force pattern according to equation (D4.11) captures only the effects of a single normal mode, and then only to the extent that the modal shape is fairly well approximated by the displacement pattern used in equation (D4.11). Modal pushover analysis has been proposed[50,51] to capture the effects of higher modes. Its application to flexible multi-storey steel frames, symmetric as well as mass-unsymmetric ones, has shown that three normal modes may suffice for agreement with the predictions of non-linear response–history analysis.

EN 1998-3[52] limits the use of pushover analysis with the two standard lateral force patterns (the 'uniform' and the 'modal' pattern) to buildings that meet condition (a) in Section 4.5.2.2 for the applicability of the lateral force analysis method (fundamental period shorter than 2 s and four times the transition period $T_C$ between the constant spectral acceleration and the constant spectral pseudo-velocity regions of the spectrum). For buildings not meeting this condition, reference is made to the use of either non-linear dynamic (response–history) or modal pushover analysis.

#### *4.5.5.3. Non-linear dynamic (time–history) analysis*

*Clauses 3.2.3.1.1(2), 3.2.3.1.2(4)(a), 4.3.3.4.3(1), 4.3.3.4.3(3)*

The non-linear dynamic (time–history or response–history) analysis method was developed in the 1970s for research, code calibration or other special purposes. Since then, and owing to the wide availability of several reliable and numerically stable computer programs with non-linear dynamic analysis capabilities, the method has gained a place in engineering practice for the evaluation of structural designs previously achieved through other approaches (e.g. through conventional force-based design that uses the $q$ factor and linear analysis) or through cycles of analysis and design evaluation. Its practical application is greatest in structures (buildings or bridges) with seismic isolation, as there the response is governed by a few elements (the isolation devices) with force–deformation behaviour which is strongly non-linear and does not follow a standard pattern (i.e. it depends on the specific device used).

Unlike the static version, the dynamic version of non-linear analysis does not require an *a priori* and approximate determination of the global non-linear seismic demand (cf. the target displacement in pushover analysis). Global displacement demands are determined in the course of the analysis of the response. Moreover, unlike modal response spectrum analysis, which provides only best estimates of the peak response (through statistical means, such as

the SRSS and the CQC rules), peak response quantities determined by non-linear dynamic analysis are exact, within the framework of the reliability and representativeness of the non-linear modelling of the structure. The only drawbacks of the approach are its sophistication and the relative sensitivity of its outcome to the choice of input ground motions.

For a non-linear dynamic analysis the seismic action should be represented in the form of time–histories of the ground motion, conforming, on average, to the 5% damping elastic response spectrum defining the seismic action. At least three artificial, recorded or simulated records should be used as input (or pairs or triplets of different records, for analysis under two or three simultaneous components of the action). If the response is obtained from at least seven non-linear time–history analyses with (triplets or pairs of) ground motions conforming, on average, to the 5% damping elastic response spectra, the average of the response quantities from all these analyses may be used as the action effect in the relevant verifications. Otherwise, the most unfavourable value of the response quantity among the analyses should be used.

## 4.6. Modelling of buildings for linear analysis

### 4.6.1. Introduction: the level of discretization

In constructing the structural model of a building for the purposes of its earthquake-resistant design, the designer should keep in mind that his or her objective is the design of an earthquake-resistant structure and not the analysis *per se*. This ultimate objective is pursued through a long process, an intermediate stage of which is normally a linear elastic analysis of a mathematical model of the structure, as conceived. A subsequent, and at least equally important phase, is that of the detailed design of members, which comprises dimensioning of regions for the internal force results of the analysis and member detailing for the ductility demands of the design seismic action. The only purpose of modelling and analysis is to provide the data for this penultimate phase of detailed design. Rules for practical dimensioning and detailing of members against cyclic inelastic deformations are sufficiently developed mainly – if not only – for prismatic members. Corresponding rules for 2D members are available only for special cases with a specific structural role, e.g. low-shear-ratio coupling concrete beams in antisymmetric bending, seismic link regions in steel frames with eccentric bracings, or interior or exterior beam–column joint panel zones. So, the structural model should employ primarily 3D beam elements.

According to *Section 4* of EN 1998-1, the model of the building structure for linear elastic analysis should represent well the distribution of stiffness in structural elements and of the mass throughout the building. This may not be enough for the purposes of design. As emphasized in the above, the idealization and discretization of the structure should correspond closely to its geometric configuration in 3D, so that it is fit for the main purpose of the analysis, i.e. to provide the seismic action effects for the dimensioning and detailing of members and sections. This means, for instance, that a stick-type model, with all members of a storey combined into a single mathematical element connecting adjacent floors and only three degrees of freedom per storey (for analysis in 3D) is not sufficient for the purposes of seismic design. At the other extreme, a very detailed finite-element discretization, providing very 'accurate' predictions of elastic displacements and stresses on a point-by-point basis, is practically useless, as reliable and almost equally accurate predictions of the 'average' seismic action effects which are necessary for member dimensioning, i.e. the stress resultants, can be directly obtained through an appropriate space frame idealization of the structure. Moreover, some fine effects captured by detailed finite-element analyses, such as those of non-planar distributions of strains in the cross-section of deep members, or shear lag in members with composite cross-section, lose their relevance under inelastic response conditions, such as those encountered under the design seismic action and used as the basis of ultimate limit state calculations and member verification. It should also be recalled that

*Clause 4.3.2(1)*

the connection between (1) a 2D element or region modelled using 2D finite-element and (2) 3D beam elements in the same plane requires special treatment, as in shell finite-elements the rotation degrees of freedoms about the normal to the shell surface do not have any stiffness and hence cannot be directly connected to 3D beam elements. For all these reasons, the type of structural model appropriate for an analysis for seismic design is a member-by-member type of model, in which every beam, column or bracing and every part of a wall between successive floors is represented as a 3D beam element, with the three translations and the three rotations at each joint of these elements considered as degrees of freedom. Masses may also be lumped at these nodal points and associated in general with all six degrees of freedom there. If the analysis also considers the vertical component of the seismic action, lumped masses at intermediate points of long-span girders or at the ends of cantilevers should also be included. This requires nodes with six degrees of freedom at these points, regardless of whether other elements frame into them there, or not.

### 4.6.2. Modelling of beams, columns and bracings

Beams, columns and bracings are normally modelled as prismatic 3D beam elements, characterized by their cross-sectional area, $A$, moments of inertia, $I_y$ and $I_z$, with respect to the principal axes $y$ and $z$ of the cross-section, shear areas $A_y$ and $A_z$ along these local axes (for shear flexibility, which is important in members with low length-to-cross-sectional-depth ratio) and torsional moment of inertia, $C$ or $I_x$ for St Venant torsion about the member centroidal axis $x$.

Members with a cross-section consisting of more than one rectangular part (e.g. L-, T- and C-shaped sections) are always dimensioned for internal forces (moments and shears) parallel to the sides of the cross-section. So, the analysis should provide action effects referring to centroidal axes parallel to the sides. In columns, walls or bracings with non-symmetrical cross-section (e.g. L- and T-shaped sections, etc.), these axes normally deviate from the principal axes of the cross-section. When this deviation is large and the difference in flexural rigidity between the two actual principal directions of bending is significant (e.g. in L-shaped sections), and if it is considered important that the bending moments from the analysis reflect this difference (e.g. for consistency with the different flexural capacities in these two directions), then, along with the easily computed moments of inertia with respect to centroidal axes $y$ and $z$ parallel to the sides of the cross-section, its product of inertia $I_{yz}$ should also be specified (alternatively, the orientation of the principal axes $y$ and $z$ with respect to the global coordinate system, and principal moments of inertia should be given). For the same type of section, shear areas in the two directions parallel to the sides may be taken as equal to the full area of the rectangle(s) with the long sides parallel to the direction of interest and projected on the principal centroidal axes, to find the shear areas $A_y$ and $A_z$ in these directions.

Concrete or composite beams connected with a concrete slab are considered to have a T, L, etc., cross-section, with the effective flange width considered constant throughout the span. The effective slab width, taken for convenience to be the same as for gravity loads, is specified in the material Eurocodes as a fraction of the distance between successive points of inflection of the beam. In long girders supporting at intermediate points secondary joist beams or even vertically interrupted ('cut-off') columns and modelled as a series of sub-beams, the effective flange width of all these sub-beams should be taken to be the same, and established on the basis of the overall span of the girder between supports on vertical elements. In contrast, the effective flange width of secondary joist beams will depend on their shorter spans between girders.

At variance with the statement in the second paragraph of the present subsection on columns, walls or bracings with an L, T or other non-symmetrical cross-section, beams with a concrete flange connected to a floor slab should be assigned local $y$ and $z$ axes normal and parallel to the plane of the slab, respectively, even when their webs are not normal to the plane of the slab (e.g. horizontal beams supporting an inclined roof). The moment of inertia $I_z$ is computed for the T or L section on the basis of the effective flange width and the shear

area $A_y$ is that of the beam web alone. If the slab to which the beam is connected is considered as a rigid diaphragm, the values of $A$, $I_y$ and $A_z$ are immaterial; if this is not the case, these properties may have to be determined to model the flexibility of the diaphragm.

According to *Section 4* of EN 1998-1 the structural model should also account for the contribution of joint regions (e.g. end zones in beams or columns of frames) to the deformability of the structure. To this end, the length of the 3D beam element which falls within the physical region of a joint with another member is often considered as rigid. If this is done for all members framing into a joint, the overall structural stiffness is overestimated, as significant shear deformation takes place in the joint panel zone (there is also slippage and partial pull-out of longitudinal bars from concrete joints). It is recommended, therefore, that only the part within the physical joint of the less bulky and stiff elements framing into it, e.g. normally of the beams, is considered to be rigid. There are two ways of modelling the end region(s) of a member as rigid: *Clause 4.3.2(2)*

(1) to consider the clear length of the element, say of a beam, as its real 'elastic' length and use a $(6 \times 6)$ transfer matrix to express the rigid-body-motion kinematic constraint between the degrees of freedom at the real end of the member at the face of the column and those of the mathematical node, where the mathematical elements are interconnected
(2) to insert fictitious, nearly infinitely rigid, short elements between the real ends of the 'elastic' member and the corresponding mathematical nodes.

Apart from the increased computational burden due to the additional elements and nodes, approach 1 may produce ill-conditioning, due to the very large difference in stiffness between the connected elements, real and fictitious. If this approach is used due to lack of computational capability for approach 2, the sensitivity of the results to the stiffness of the fictitious members should be checked, e.g. by ensuring that they remain almost the same when the stiffness of the fictitious elements changes by an order of magnitude.

If the end regions of a member, e.g. of a beam, within the joints are modelled as rigid, member stress resultants at member ends, routinely given in the output of the analysis, can be used directly for dimensioning the member end sections at the column faces. If no such rigid ends are specified, as recommended above for columns, then either the stress resultants at the top and the soffit of the beam will be separately calculated on the basis of the beam depth, etc., or dimensioning of the column will be conservatively performed on the basis of the stress resultants at the mathematical nodes.

If the centroidal axes of connected members do not intersect, the mathematical node should be placed on the centroidal axis of one of the connected members, typically a vertical one, and the ends of the other members should be connected to that node at an eccentricity. The eccentricity of the connection will be readily incorporated in the modelling of the beam end regions within the joint as rigid: the rigid end will not be collinear with the beam axis but at an angle.

Distributed loads specified on a member with rigid ends are often considered by the analysis program to act only on the 'elastic' part of the member between the rigid ends. The part of the load which is unaccounted for as falling outside the 'elastic' member length should be specified separately as concentrated forces at the nodes.

### 4.6.3. Special modelling considerations for walls

The marked preference of Section 4.6.1 in favour of member-type modelling, representing every individual structural member between connections to others as a single 3D beam element, applies also to concrete, masonry or even composite (steel–concrete) walls, or at least to parts of such walls between successive floors and/or substantial openings. Such modelling of walls is often called 'wide-column analogy'. Supporting this position is the requirement of *Section 5* of EN 1998-1 that concrete walls with a section consisting of connected or intersecting rectangular segments (L, T, U, I or similar), should be dimensioned in bending with axial force and in shear as a single integral unit, consisting of one or more webs (approximately) parallel to the shear force and one or more flanges *Clause 5.4.3.4.1(4)*

(about) normal to it, regardless of how they are modelled for the analysis. Moreover, the rules for calculating the confinement reinforcement in such walls are also given considering a single integral section. So, it is most convenient for the subsequent phases of dimensioning and detailing to model walls with any section as a single storey-tall 3D beam element having the cross-sectional properties of the entire section. The only questions on the approach may refer to the modelling of torsion in walls with section other than (nearly) rectangular, as detailed after the next paragraph.

An alternative to the single-element modelling of a wall with a section consisting of connected or intersecting rectangular segments is to use a separate 3D beam element at the centroidal axis of each rectangular segment of the section. To dimension and detail the entire cross-section in bending with axial force, as required for concrete walls by *Section 5* of EN 1998-1, computed bending moments and axial forces of the individual 3D beam elements need to be composed into a single $M_y$, a single $M_z$ and a single $N$ for the entire section. If these elements are connected at floor levels to a common mathematical node (e.g. through absolutely rigid horizontal segments or equivalent kinematic constraints), the model is completely equivalent to a single 3D beam element along the centroidal axis of the entire section.

With the possible exception of walls with a semi-closed channel section, compatibility torsion is not an important component of the seismic resistance of walls. So, accurate estimation of torsion-induced shear for the purposes of the design of the wall itself is unimportant. The relevant issue is whether potentially unrealistic modelling of the torsional stiffness and response of a wall with a section other than (nearly) rectangular significantly affects the predicted seismic action effects in other structural members. If storey-tall 3D beam elements with the cross-sectional properties of the entire section are used, then a step to improve the accuracy of the prediction of seismic action effects in other members is to place the axis of the 3D beam element modelling the wall through the shear centre of its cross-section, instead of the centroidal axis. For L- or T-shaped sections this is very convenient, as the shear centre is at the intersection of the longitudinal axes of the two rectangular parts of the cross-section, which typically coincide with the axes of the webs of beams framing into the wall. Placing the axis of the wall element at the shear centre of the cross-section rather than at its centroid introduces an error in the calculation of the vertical displacement induced at the end of a beam connected to the corresponding node of the wall through a horizontal rigid arm by the flexural rotation of the wall. Another issue is that the estimation of the torsional rigidity, $GC$, of the cross-section assuming pure St Venant torsion (i.e. as $G\sum(l_w b_w^3/3)$, with $l_w$ and $b_w$ denoting the length and thickness of each rectangular part of the section) does not account for the resistance to torsion-induced warping of the section. In considering these problems, though, the designer should bear in mind the large uncertainty regarding the reduction of torsional rigidity due to concrete cracking, as described in Section 4.6.4 (last paragraph).

Beams framing into the wall at floor levels, etc., should be connected to the mathematical node at the axis of the wall. Any eccentricity between this node and the real end of the beam should be modelled as a rigid connection. If eccentrically framing beams are at right angles to the plane of the wall (i.e. in its weak direction), it is more accurate to include some flexibility of the connection, if this is computationally feasible: the very stiff or rigid connecting element between the end of the beam and the node at the wall centreline may be considered to have a finite torsional rigidity, $GC = Gh_{st}b_w^3/3$, where $h_{st}$ is the storey height and $b_w$ is the thickness of the web of the wall.

### 4.6.4. Cracked stiffness in concrete and masonry

*Clauses 4.3.1(6), 4.3.1(7)*

A fundamental assumption underlying the provisions of Eurocode 8 for design for energy dissipation and ductility is that the global inelastic response of a structure to monotonic lateral forces is bilinear, close to elastic–perfectly-plastic. The elastic stiffness used in analysis should correspond to the stiffness of the elastic branch of such a bilinear global force–deformation response. This means that the use of the full elastic stiffness of uncracked

concrete or masonry in the analysis is completely inappropriate. For this reason, *Section 4* of EN 1998-1 requires that the analysis of concrete, composite steel–concrete or masonry buildings should be based on member stiffness, taking into account the effect of cracking. Moreover, to reflect the requirement that the elastic stiffness corresponds to the stiffness of the elastic branch of a bi-linear global force–deformation response, *Section 4* of EN 1998-1 also requires that the stiffness of concrete members corresponds to the initiation of yielding of the reinforcement. Unless a more accurate modelling of the cracked member is performed, it is permitted to take that stiffness as equal to 50% of the corresponding stiffness of the uncracked member, neglecting the presence of the reinforcement. This default value is quite conservative: the experimentally measured secant stiffness of typical reinforced concrete members at incipient yield, including the effect of bar slippage and yield penetration in joints, is on average about 25% or less of that of the uncracked gross concrete section.[53] The experimental values are in good agreement with the effective stiffness specified in Eurocode 2 for the calculation of second-order effects in concrete structures:

- a fraction of the stiffness $E_c I_c$ of the uncracked gross concrete section equal to 20% or to 0.3 times the axial load ratio $\nu_d = N/A_c f_{cd}$, whichever is smaller, plus the stiffness $E_s I_s$ of the reinforcement with respect to the centroid of the section, or
- if the reinforcement ratio exceeds 0.01 (but its exact value may not be known yet), 30% of the stiffness $E_c I_c$ of the uncracked gross concrete section.

When an estimate of the effective stiffness on the low side is used in the analysis, second-order effects increase, which is safe-sided in the context of Eurocode 2. In contrast, within the force- and strength-based seismic design of Eurocode 8 it is more conservative to use a high estimate of the effective stiffness, as this reduces the period(s) and increases the corresponding spectral acceleration(s) for which the structure has to be designed. The use of 50% of the uncracked section stiffness serves exactly that purpose. However, lateral drifts and $P$–$\Delta$ effects computed on the basis of overly high stiffness values may be seriously underestimated.

Torsion in beams, columns or bracings is almost immaterial for their earthquake resistance. In concrete buildings the reduction of torsional rigidity when the member cracks diagonally is much larger than that of shear or flexural rigidity upon cracking. The effective torsional rigidity, $GC_{ef}$, of concrete members should be assigned a very small value (close to zero), because torsional moments due to deformation compatibility drop also with torsional rigidity upon cracking, and their overestimation may be at the expense of member bending moments and shears, which are more important for earthquake resistance. The reduction of member torsional rigidity should not be effected through reduction of the concrete $G$ value, as this may also reduce the effective shear stiffness $GA_{sh}$, and unduly increase member shear deformations.

### 4.6.5. Accounting for second-order ($P$–$\Delta$) effects

*Section 4* of EN 1998-1 requires taking into account second-order ($P$–$\Delta$) effects in buildings, when for the vertical members of the storey, these exceed 10% of the first-order effects in the aggregate. The criterion is the value of the interstorey drift sensitivity coefficient, $\theta$, defined for storey $i$ as the ratio of the total second-order moment in storey $i$ to the change in the first-order overturning moment in that storey:

*Clauses 4.4.2.2(2), 4.4.2.2(3)*

$$\theta_i = \frac{N_{tot,i}\Delta d_i}{V_{tot,i}h_i} \qquad \text{(D4.20)}$$

where:

- $N_{tot,i}$ is the total gravity load at and above storey $i$ in the seismic design situation, i.e. as determined according to Section 4.4.2.
- $\Delta d_i$ is the interstorey drift at storey $i$, i.e. the difference of the average lateral displacements at the top and bottom of the storey, $d_i$ and $d_{i-1}$; if linear elastic analysis is

used on the basis of the design response spectrum, (i.e. the elastic spectrum for 5% damping divided by the behaviour factor $q$), then the values of the displacements to be used for $d_i$ and $d_{i-1}$ are those from the analysis multiplied by the behaviour factor $q$; the value of the interstorey drift is determined at the centre of mass of the storey (at the master node, if one is used).

- $V_{\text{tot},i}$ is the total seismic shear at storey $i$.
- $h_i$ is the height of storey $i$.

Second-order effects may be neglected, provided that the value of $\theta_i$ does not exceed 0.1 in any storey; however, they should be taken into account for the entire structure, if at any storey the value of $\theta_i$ exceeds 0.1. If the value of $\theta_i$ does not exceed 0.2 at any storey, *Section 4* of EN 1998-1 allows $P$–$\Delta$ effects to be taken into account approximately without a second-order analysis, by multiplying all first-order action effects due to the horizontal component of the seismic action by $1/(1-\theta_i)$. Although it is the value $\theta_i$ of the individual storeys that can be used in this amplification, the use for the entire structure of the maximum value of $\theta_i$ in any storey is safe-sided and maintains force equilibrium in the framework of first-order analysis. In the rather unlikely case that a value of $\theta_i$ exceeds 0.2 in any one of the storeys, an exact second-order analysis is required. This analysis may be performed with the modelling described in the next paragraph for buildings without rigid diaphrams.

If the vertical members connect floors considered as rigid diaphragms, $P$–$\Delta$ effects can be accounted for sufficiently according to the previous paragraph. If there are no such floors, or if floors cannot be taken as rigid diaphragms, then $P$–$\Delta$ effects may be considered on an individual column basis, by subtracting from the column elastic stiffness matrix its linearized geometric stiffness matrix. If the analysis is elastic on the basis of the design response spectrum, the linearized geometric stiffness matrix of each column should be multiplied by the behaviour factor $q$, to account for the fact that $P$–$\Delta$ effects should be computed for the full inelastic deformations of the structure and not for the elastic ones which incorporate division by the behaviour factor $q$. Within the framework of elastic analysis, column axial forces in the geometric stiffness matrix may be considered as constant and equal to the value due to the gravity loads included in the seismic design situation according to Section 4.4.2.

## 4.7. Modelling of buildings for non-linear analysis

### 4.7.1. General requirements for non-linear modelling

*Clauses 4.3.3.4.1(1), 4.3.3.4.1(2)*

Modelling for the purposes of non-linear analysis should be an extension of that used for linear methods, to include the post-elastic behaviour of members beyond their yield strength. Put differently, as a non-linear analysis degenerates into a linear one if member yield strength is not attained during the seismic response, in the linear range of behaviour, modelling for non-linear analysis should be consistent with that used for linear analysis. Consistency does not imply that the level of discretization and the modelling of elastic stiffness needs to be identical to that used in linear analysis: as non-linear analysis is done mainly for the purposes of evaluation of a design, its modelling is not bound by the fact that present-day seismic dimensioning and detailing rules address the member as a whole and hence point in the direction of member-by-member modelling in linear analysis. However, all things considered – including the consistency with linear analysis and the computational and modelling effort required for non-linear finite-element modelling – the member-by-member type of modelling, with every beam, column, bracing or part of a wall between successive floors modelled as a non-linear 3D beam element, is the most appropriate option for non-linear analysis.

In principle, only the stiffness properties of members are of interest for linear elastic analysis. As emphasized in Section 4.6.4, to reflect the requirement that the elastic global stiffness corresponds to the stiffness of the elastic branch of a bi-linear global force–deformation response in monotonic loading, the elastic stiffness of a bilinear monotonic force–deformation relation in a member model should be the secant stiffness to the yield

point. Member models to be used in non-linear analysis should also include the yield strength of the member, as this is governed by the most critical (i.e. weakest) mechanism of force transfer in the member, and the post-yield branch in monotonic loading thereafter.

The bilinear force–deformation relationship advocated here for the monotonic force–deformation relation in a non-linear member model is a minimum requirement according to the relevant clause of Eurocode 8. For concrete and masonry, the elastic stiffness of such a bilinear force–deformation relation should be that of the cracked concrete section according to Section 4.6.4. If it is taken equal to the default value of 50% of the uncracked gross section stiffness for consistency with the linear analysis, storey drifts and member deformation demands are seriously underestimated. In case the response is evaluated by comparing member deformation demands to (realistic) deformation capacities, such as those given in Annex A of Part 3 of Eurocode 8,[52] then demands should also be realistically estimated by using as the effective elastic stiffness a representative value of the member secant stiffness to incipient yielding (also given in Annex A of EN 1998-3,[52] after Biskinis and Fardis[54]).

If the monotonic behaviour exhibits strain hardening after yielding (as in concrete members in bending and in steel or composite members in bending or shear, or in tension) a constant hardening ratio (e.g. 5%) may be considered for the post-yield stiffness. Alternatively, positive strain hardening may be neglected and a zero post-yield stiffness may be conservatively adopted. However, elements exhibiting post-elastic strength degradation, e.g. (unreinforced) masonry walls in shear or steel braces in compression, should be modelled with a negative slope of their post-elastic monotonic force–deformation relationship. It should be pointed out that the ductile mechanisms of force transfer also exhibit significant strength degradation when they approach their ultimate deformation. However, as in new designs the deformation demands in ductile members due to the design seismic action stay well below their ultimate deformation, there is no need to introduce a negative slope anywhere along their monotonic force–deformation relationship.

*Clause 4.3.3.4.1(3)*

Gravity loads included in the seismic design situation according to Section 4.4.2 should be taken to act on the relevant elements of the model as in linear analysis. Eurocode 8 requires taking into account the value of the axial force due to these gravity loads, when determining the force–deformation relations for structural elements. This means that the effect of the fluctuation of axial load during the seismic response may be neglected. In fact, this fluctuation is significant only in vertical elements on the perimeter of the building and in the individual walls of coupled wall systems. Most element models can take into account – be it only approximately – the effect of the fluctuation of axial load on the force–deformation relations of vertical elements. Examples are the fibre models, as well as any simple lumped inelasticity (point hinge) model with parameters (e.g. yield strength and effective elastic stiffness) which are explicitly given in terms of the current value of the axial load.

For simplicity, Eurocode 8 allows the bending moments in vertical members due to gravity loads to be neglected, unless they are significant with respect to the flexural capacity of the member.

*Clauses 4.3.3.4.1(5), 4.3.3.4.1(6)*

Non-linear models should be based on mean values of material strengths, which are higher than the corresponding nominal values. For an existing building the mean strength of a specific material is the one inferred from *in situ* measurements, laboratory tests of samples and other relevant sources of information. For the mean strength of materials to be incorporated in the future in a new building, Eurocode 8 makes reference to the material Eurocodes. However, only the mean strength of concrete is given there: Eurocode 2 gives the mean strength as 8 MPa greater than the characteristic strength, $f_{ck}$. Statistics drawn from all over Europe suggest a mean value of the yield strength of steel about 15% higher than the characteristic or nominal value, $f_{yk}$. Locally applicable data should be used for the reinforcing steel, if known. Similarly for structural steel, for which the relatively small number of manufacturers serving most parts of Europe points towards the most likely supplier of the steel to be used as the source of relevant statistics.

*Clause 4.3.3.4.1(4)*

Other than the use of the mean value of material strengths instead of the design values, member strengths (resistances) to be used in the non-linear member models may be computed as for the relevant force-based verifications.

It is noteworthy that the use of mean material properties is not specific to non-linear analysis: linear analysis is based on mean values of elastic moduli, which are the only material properties used for the calculation of (the effective) elastic stiffness.

### 4.7.2. Special modelling requirements for non-linear dynamic analysis

*Clause 4.3.3.4.3(2)*

In order to be used in non-linear response–history analysis, member force-displacement models need only be supplemented with hysteresis rules describing the behaviour in post-elastic unloading–reloading cycles. The only requirement posed by Eurocode 8 for the hysteresis rules is to reflect realistically energy dissipation within the range of displacement amplitudes induced in the member by the seismic action used as input to the analysis. Given that the predictions of non-linear dynamic analysis – especially those for the peak response – are not very sensitive to the exact shape and other details of the hysteresis loops produced by member models, a far more important attribute of the model used for the hysteresis is the numerical robustness under any conceivable circumstance. This is crucial, as it is almost certain that potential numerical weakness of the model will show up in an analysis involving possibly hundreds of non-linear members, thousands of time-steps and, possibly, a few iteration cycles within each step. In some cases, local numerical problems may develop into lack of convergence and global instability of the response. Inertia forces and other stabilizing influences may sometimes prevent local numerical problems from causing global instability; due to the numerical problems, though, local or even global predictions of the response may be in error and – what is worse – it takes a lot of experience and judgement to recognize that predictions are wrong. In general, simple and clear hysteresis models that use just a few rules to describe the response under any cycle of unloading and reloading, small or large, complete or partial, are less likely to lead to numerical problems than elaborate, complex and often obscure models.

Given that within the framework of EN 1998-1 non-linear dynamic analysis is meant to be applied for the evaluation of new buildings designed for a minimum of ductility and dissipation capacity according to this part of Eurocode 8, the non-linear response will be limited to ductile and stable mechanisms of cyclic force transfer and will be prevented in brittle or degrading ones. This facilitates the choice of hysteretic rules, as degradation of stiffness and strength with cycling can be ignored as insignificant. Therefore, the best balance of accuracy, simplicity and reliability is provided by the following types of models for members with a ductile-dominant mechanism of cyclic force transfer:

- For steel or composite (steel–concrete) beams, columns or seismic links in unidirectional cyclic bending and shear with axial force, and for steel or composite (steel–concrete) bracings in tension: an elastic, linearly strain-hardening (bilinear) force–deformation model for monotonic loading and a bilinear cyclic model with kinematic hardening and unloading and reloading branches parallel to those of the monotonic response.
- For concrete beams, columns or walls in unidirectional cyclic bending with axial force (shear in concrete is a brittle mechanism of force transfer and it is designed for sufficient overstrength with respect to flexure so that it is kept in the elastic range): an elastic, linearly strain-hardening (bilinear) force–deformation model for monotonic loading; linear unloading up to zero-force and linearly reloading thereafter towards the most extreme point reached previously on the monotonic loading curve in the opposite direction. In other words, a model with 'stiffness degradation' but without 'strength degradation' or 'pinching' (e.g. a modified Takeda model,[55] according to Otani[56]).
- For steel or composite (steel–concrete) bracings in alternating tension and compression: an elastic, linearly strain-hardening (bilinear) force–deformation model for monotonic loading in tension, linearly unloading up to the buckling load in compression; shedding

load linearly or non-linearly with shortening after buckling; linearly reloading from compression to tension towards the most extreme point reached previously on the monotonic curve in tension.

Non-linear dynamic analysis is considered to excel over its static counterpart (pushover analysis) mainly in its ability to capture the effects of modes of vibration higher than the fundamental mode. For this to be done correctly, member non-linear models should provide a realistic representation of the stiffness of all members up to their yield point. This is far more important than for non-linear static (pushover) analysis because higher modes, when they are important, often involve post-yield excursions in members which stay in the elastic range under the fundamental mode alone. Moreover, in pushover analysis it is primarily (if not only) the determination of the target displacement that is affected by the effective stiffness to yielding. In fact, the target displacement depends only on the global elastic stiffness which is fitted to the capacity curve and is possibly sensitive to the elastic stiffness of certain members which may be crucial for global yielding but are not known before the analysis.

If the response is fully elastic, the peak response predicted through non-linear time–history analysis should be consistent with the elastic response spectrum of the input motion (exactly in the extreme case of a single-degree of freedom system, or in good approximation for a multi-degree of freedom system subjected to modal response spectrum analysis with the CQC modal combination rule). Such conformity is difficult to achieve when using a trilinear monotonic force–deformation relationship for members that takes into account the difference in pre- and post-cracking stiffness of concrete and masonry (e.g. see Takeda[55]), as allowed by Eurocode 8. Under cyclic loading such models produce hysteretic damping in the pre-yielding stage of the member, which increases with displacement amplitude from zero at cracking to a maximum value at yielding. Similarly to the equivalent viscous damping ratio, in that range of elastic response the elastic stiffness of the trilinear model is not uniquely valued. This ambiguity does not allow direct comparisons with the elastic response spectrum predictions, let alone conformity. For this reason, it is preferable in non-linear dynamic analysis to use member models with a force–deformation relationship which is (practically) bilinear in monotonic loading. After all, it is expected that, at the time it is subjected to a strong ground motion, a concrete or masonry structure will already be extensively cracked due to gravity loads, thermal strains and shrinkage, or even previous shocks. Last but not least, steel (or even composite steel–concrete) members have a (practically) bilinear force–deformation curve under monotonic loading, and it is convenient for computer programs to use the same type of monotonic force–deformation model for all structural materials.

It should be pointed out that in non-linear static (pushover) analysis the effect of using a trilinear monotonic force–deformation relationship for members will be limited to the initial part of the capacity curve and will not give rise to the problems and ambiguities mentioned above in connection with the application of non-linear dynamic analysis.

If non-linear dynamic analysis employs for members a bilinear force–deformation model under monotonic loading, as advocated above, it should also account for the 5% viscous damping ratio considered to characterize the elastic (in this case pre-yield) response. Unless the computer program used for the non-linear dynamic analysis provides the facility of user-specified viscous damping for all modes of practical importance, Rayleigh damping should be used. To ensure a damping ratio not far from 5% for elastic response in all these modes, it may be specified as equal to 5% at:

(1) the natural period of the mode with the highest modal base shear, for analysis under a single component of the seismic action, or at the average of the natural periods of the two modes with the highest modal base shears in two nearly orthogonal horizontal directions, for simultaneous application of the two horizontal components
(2) twice the value of the period in point 1.

### 4.7.3. The inadequacy of member models in 3D as a limitation of non-linear modelling

It is natural to expect that a sophisticated method (in this case non-linear seismic response analysis) will be at least as good at tackling general design situations in their full complexity as simplified approaches (in the present case, linear seismic response analysis). However, as already noted, the non-linear static (pushover) analysis method has been developed for analysis of seismic response in 2D (regardless of whether a 3D structural model is used), and its application in cases of truly 3D response (due to torsional effects) still raises certain questions. Although it has also been developed primarily for 2D analysis, the non-linear dynamic method can, in principle, be applied equally well to seismic response analysis in 3D. It is presumed that, for such an extension to 3D, appropriate models of the behaviour of members under 3D loading are available. However, the lack of reliable yet simple models for the (monotonic or cyclic) post-elastic behaviour of vertical members in two orthogonal transverse directions (in biaxial bending and shear with axial load) is currently the single most important challenge to the achievement of full-fledged non-linear seismic response analysis, static or dynamic, in 3D.

Fibre models of members can, in principle, represent well the (monotonic or cyclic) post-elastic flexural behaviour of prismatic members in the two orthogonal directions of bending. However, due to the requirements of such models in computer time and memory and the exponential increase of the risk of numerical problems with the amount of calculations, fibre models cannot be used practically for the non-linear seismic response analysis in 3D of full-sized buildings. Moreover, fibre models need careful tuning of their input properties and parameters, in order to reproduce the intended behaviour pattern of a member, including its connections – be it a pattern consistent with the fundamental assumptions and rules specified in Eurocode 8 for member modelling, or experimental behaviour: such tuning requires specialized knowledge and experience, which is far beyond the current capabilities of design professionals. Lumped inelasticity (point hinge) models are not capable of representing well the (monotonic or cyclic) post-elastic behaviour of members in two orthogonal transverse directions, without sacrificing their simplicity, flexibility and – most importantly – their reliability and numerical stability, i.e. all the attributes that made them the workhorse of member modelling for non-linear analysis in 2D. Currently, non-linear seismic response analysis in 3D often uses one independent model of this type in each one of the two orthogonal directions of bending. Coupling of the response between these two directions is normally ignored, or taken into account only as far as the value of the yield moment and the failure criteria in terms of plastic hinge rotations in the two orthogonal directions of bending. Such an approximation is usually acceptable if the non-linear response is primarily in one of the two directions of bending, as is often the case in fairly symmetric buildings subjected to a single horizontal component of the seismic action. It may be insufficient – and certainly in the unconservative direction – for simultaneous application of the two horizontal components of the seismic action and/or when the building develops a strongly torsional response due to irregularity in plan.

## 4.8. Analysis for accidental torsional effects

### 4.8.1. Accidental eccentricity

*Clauses 4.3.2(1), 4.3.3.2.4(2), 4.3.6.3.1(2), 4.3.6.3.1(4)*

When the distributions of stiffness and/or mass in plan are unsymmetric, the response to the horizontal components of the seismic action has certain torsional–translational features. These features are sufficiently taken into account in an analysis in 3D for the horizontal components, especially when a modal response spectrum analysis or a non-linear dynamic one is performed. Unlike some other seismic design codes, amplification or de-amplification of the 'natural' eccentricities between the centres of mass and stiffness is not required. This is convenient, because normally the storey stiffness centre cannot be uniquely defined (see Section 4.3.2.1). Moreover, determination of the position of a conventionally defined

storey stiffness centre at a level of accuracy and sophistication consistent with the dynamic amplification of natural eccentricities, requires tedious additional analyses.

For buildings with full symmetry of stiffness and nominal masses in plan, the analysis for the horizontal components of the seismic action gives no torsional response at all. Effects which cannot be captured by conventional seismic response analysis according to Eurocode 8, such as variations in the stiffness and mass distributions from the nominal ones considered in the analysis, or a possible torsional component of the ground motion about a vertical axis, may produce a torsional response even in nominally fully symmetric buildings. To ensure a minimum of torsional resistance and stiffness and limit the consequences of unforeseen torsional response, EN 1998-1 introduces accidental torsional effects by displacing the masses with respect to their nominal positions adopted in modelling. This displacement is assumed to take place in the positive and in the negative sense along any horizontal direction (in practice, along the two orthogonal directions of the horizontal seismic action components). It is more conservative for the global seismic action effects to consider that all the masses of the structure are displaced along the same horizontal direction and in the same sense (positive or negative) at a time.

It is completely impractical to study the effect of displacing the masses through dynamic analysis: the dynamic characteristics of the system will change with the location of the masses. So, Eurocode 8 allows replacing the 'accidental eccentricity' of the masses from their nominal positions, by 'accidental eccentricity' of the horizontal seismic components with respect to the nominal position of the masses. All accidental eccentricities are considered at a time along the same horizontal direction and in the same sense (positive or negative). The effects of this accidental eccentricity are determined through static approaches.

The accidental eccentricity of a horizontal seismic action component is specified as a fraction of the dimension of the storey in plan orthogonal to this horizontal component. The fraction of the storey plan dimension is normally 5%; it is doubled to 10% if the effects of accidental eccentricity are taken into account in the simplified way described in Section 4.8.3 and, in addition, instead of a full structural model in 3D for each horizontal component of the seismic action, a separate 2D model is analysed (which is allowed in structures regular in plan, but entails neglecting any small static eccentricity that may exist between the floor centres of stiffness and mass). Moreover, if there are masonry infills with a moderately irregular and unsymmetric distribution in plan (this excludes strongly irregular arrangements, such as infills mainly along two adjacent faces of the building), the effects of the accidental eccentricity are doubled further (i.e. as if the accidental eccentricity is 10% of the orthogonal dimension of the storey in the reference case, or 20% for simplified evaluation of accidental torsional effects when using two separate 2D models).

### 4.8.2. Estimation of the effects of accidental eccentricity through static analysis

*Clause 4.3.3.3.3(1)*

Even when the modal response spectrum method is used for the analysis of the response to the two horizontal components of the seismic action, Eurocode 8 allows a static analysis for the effects of the accidental eccentricities of these components. In this analysis, a 3D structural model is subjected to storey torques about the vertical axis, which have all the same sign and are equivalent to the storey lateral loads due to the horizontal component considered multiplied by its accidental eccentricity at the storey. The lateral loads are those calculated for the considered horizontal component of the seismic action according to the lateral force method of analysis (equations (D4.5) and (D4.7) with $\Phi_i = z_i$), even though this method may not be applicable for the particular structure. In fact, this static approach of taking into account the effects of the accidental eccentricity is essentially the implementation of the displacement of masses by the accidental eccentricity with respect to their nominal positions within the lateral force method of analysis. In the context of the modal response spectrum method it would be more meaningful and closer to the concept of displacing masses to apply the static approach with storey torques computed as the storey accidental eccentricity multiplied by the floor mass and by the floor response acceleration in

the direction of the considered horizontal component of the seismic action, computed from the modal contributions to such a floor response acceleration through the SRSS or CQC combination rule, as appropriate. The approach advocated by Eurocode 8 is computationally simpler, especially if in both horizontal directions the storey accidental eccentricity is constant at all levels (which implies constant dimensions of the building in plan at all floors). Then, it is sufficient to perform a single static analysis for storey torques proportional to the storey lateral loads from equation (D4.7) (with $\Phi_i = z_i$) for a base shear $F_b$ of unity. The effects of the 'accidental eccentricity' of each horizontal seismic action component can then be obtained by multiplying the results of this single analysis by the product of the base shear $F_b$ from equation (D4.5) that corresponds to the fundamental period of vibration in the horizontal direction of interest, multiplied by the (constant at all levels) eccentricity of this component of the seismic action.

Application of the total storey torque to a single floor node of the storey (the 'master node') according to the previous paragraph, implies the floors must act as rigid diaphragms. If the floor cannot be considered as rigid and its in-plane flexibility is taken into account in the 3D structural model, it is more meaningful to apply, instead of a storey torque, nodal torques at each node $i$ where there is a mass $m_i$, equal to the product of the accidental eccentricity and the lateral force determined from equation (D4.7) (with $\Phi_i = z_i$) for that mass.

Coming from a static analysis, the action effects of the accidental eccentricities have signs. As the sign of the accidental eccentricity should be taken such that the most unfavourable result is produced for the seismic action effect of interest, the action effect of the accidental eccentricity $e_X$ of the horizontal component $X$ of the seismic action is superimposed on that of the horizontal component $X$ itself, with the same sign as the latter. The outcome is the total seismic action effect of horizontal component $X$, $E_X$. It is these latter total first-order action effects that should be multiplied by $1/(1-\theta_i)$ to take into account *a posteriori* $P$–$\Delta$ effects. If an exact second-order analysis is performed, this has to be done both in the analysis for the horizontal component $X$ itself and in that for its accidental eccentricity.

### 4.8.3. Simplified estimation of the effects of accidental eccentricity

*Clause 4.3.3.2.4(1)*

The approach outlined in the previous section can also be applied when the lateral force method is used for the analysis of the response to the two horizontal components of the seismic action. As already pointed out, in the context of the lateral force method this approach is indeed fully consistent with the concept of displacing the masses by the accidental eccentricity with respect to their nominal position. Within the spirit of simplicity normally associated with the lateral force method of analysis, Eurocode 8 allows in that case the effects of accidental eccentricities to be accounted for in a much simpler way: by multiplying by $1 + 0.6x/L$ the results of the lateral force analysis for each horizontal component of the seismic action, where $x$ denotes the distance of the element of interest from the centre in plan and $L$ the plan dimension, both normal to the horizontal component of the seismic action. This factor is derived assuming that:

- the torsional effects are fully taken up by the stiffness and resistance of the structural elements in the direction of the horizontal component considered, without assistance from the stiffness and resistance of these and other structural elements in the orthogonal horizontal direction
- the stiffness and resistance of the structural elements taking up the torsional effects are uniformly distributed in plan.

In fact, the term $0.6/L$ is (1) the total storey torsional moment due to the accidental eccentricity of $0.05L$, namely $0.05L$ times the storey seismic shear, $V$, (2) divided by $k_B BL^3/12$, which is the moment of inertia of a uniform lateral stiffness, $k_B$, per unit floor area parallel to side $B$ in plan, and (3) further divided by the normalized storey shear, $V/k_B BL$. Normally there is also lateral stiffness, $k_L \approx k_B$, per unit floor area parallel to side $L$ in plan, which

contributes with $k_L LB^3/12$ to the polar moment of inertia to be used in point 2. As the contribution of $k_L$ is neglected, the term $0.6x/L$ is conservative by a factor of 2, on average. If the designer considers this additional conservatism is too high a price to pay for the simplicity, then he or she may choose to use with the lateral force method of analysis the general approach outlined in the previous section.

The general approach of Section 4.8.2 can only be applied to a full 3D structural model. For buildings that meet the conditions of Section 4.3.2.1 or 4.3.2.2, the designer may opt for analysis – of the lateral force or the modal response spectrum type – with a separate 2D model for each horizontal component of the seismic action. As the general approach of Section 4.8.2 cannot be applied in that case, the effects of the accidental eccentricity can only be estimated through the simplified approach of the present section. In that case the second term in the amplification factor becomes $1.2x/L$, to account also for the otherwise unaccounted for effects of any static eccentricity between the storey centres of mass and stiffness.

*Clauses 4.3.3.2.4(2), 4.3.3.3.3(3)*

## 4.9. Combination of the effects of the components of the seismic action

The two horizontal components of the seismic action and the vertical one (when it is taken into account) are considered to act simultaneously on the structure. Simultaneous occurrence of more than one component can be handled only by a time–history analysis of the response (which in Eurocode 8 is meant to be non-linear). All other analysis methods give only estimates of the peak values of seismic action effects during the response to a single component. These are denoted here as $E_X$ and $E_Y$ for the two horizontal components (considered to also include the effect of the associated accidental eccentricities) and $E_Z$ for the vertical. The peak value of the seismic action effects do not occur simultaneously, so a combination rule of the type $E = E_X + E_Y + E_Z$ is overly conservative. More representative combination rules, with a probabilistic basis, have been adopted in Eurocode 8 for the estimation of the expected value of the peak seismic action effect, $E$, under simultaneous action of the three components.

*Clauses 3.2.3.1.1(2), 4.3.3.5.1(1), 4.3.3.5.2(2), 4.3.3.5.1(7)*

The reference combination rule of the peak values of seismic action effects, $E_X$, $E_Y$ and $E_Z$, due to separate action of the individual components is the SRSS combination:[57]

$$E = \sqrt{E_X^2 + E_Y^2 + E_Z^2} \tag{D4.21}$$

*Clauses 4.3.3.5.1(2), 4.3.3.5.2(4), 4.3.3.5.1(6), 4.3.3.1(11)*

Equation (D4.21) always gives a positive result, regardless of whether $E_X$, $E_Y$ and $E_Z$ have been computed through the lateral force or the modal response spectrum method of analysis. If $E_X$, $E_Y$ and $E_Z$ are computed through the modal response spectrum method by combining modal contributions to each one of them via the CQC rule, equation (D4.10), and the seismic action components in the three directions $X$, $Y$ and $Z$ are statistically independent, in an elastic structure the outcome of equation (D4.21) is indeed the expected value of the maximum seismic action effect, $E$, under simultaneous seismic action components. Under these conditions, the outcome of equation (D4.21) is also invariant to the choice of the horizontal directions $X$ and $Y$. In other words, on the basis of a single modal response spectrum analysis that covers the three components, $X$, $Y$ and $Z$, at the same time and uses the CQC rule to combine modal contributions for each one of them, equation (D4.21) provides the expected value of the maximum elastic seismic action effect, $E$, for all members of the structure, irrespective of the choice of directions $X$ and $Y$. In this simple way, equation (D4.21) automatically fulfils an – at first sight – onerous requirement of Eurocode 8 for buildings with resisting elements not in two perpendicular directions and hence without an obvious choice of the two directions $X$ and $Y$ as the main or principal ones: namely, to apply the two horizontal components along all relevant horizontal directions, $X$, and the orthogonal direction, $Y$.

*Clauses 4.3.3.5.1(3), 4.3.3.5.2(4), 4.3.3.5.1(6)*

Eurocode 8 has adopted the combination rule of equation (D4.21) as the reference, not only under the conditions for which the rule has been developed and shown to be exact, namely the application of modal response spectrum analysis and of the CQC rule for combining modal contributions, but also for all other types of analysis: linear static analysis (the lateral force method for the horizontal components and the method outlined in Section 4.5.4.3 for the vertical component, if the latter is considered), modal response spectrum analysis with combination of modal contributions via the SRSS rule, or even non-linear static (pushover) analysis. However, Eurocode 8 also accepts as an alternative the linear combination rule:

$$E = E_X + \lambda E_Y + \lambda E_Z \quad \text{(D4.22a)}$$

$$E = \lambda E_X + E_Y + \lambda E_Z \quad \text{(D4.22b)}$$

$$E = \lambda E_X + \lambda E_Y + E_Z \quad \text{(D4.22c)}$$

where the meaning of '+' is superposition. With the three terms in each of the three alternatives of equation (D4.22) taken to have the same sign, a value $\lambda \approx 0.275$ provides the best average agreement with the result of equation (D4.21) within the entire range of possible values of $E_X$, $E_Y$ and $E_Z$. In Eurocode 8 this optimal $\lambda$ value has been rounded up to $\lambda = 0.3$, which may underestimate the result of equation (D4.21) by at most 9% (when $E_X$, $E_Y$ and $E_Z$ are about equal) and may overestimate it by not more than 8% (when two of these three seismic action effects are an order of magnitude less than the third).

If dimensioning is based on a single, one-component stress resultant, such as for beams in bending or shear, the outcome of equation (D4.21), or the maximum value among the three alternatives in equation (D4.22) (with the three terms in each alternative taken positive), should be added to, or subtracted from, the action effect of the gravity loads considered to act in the seismic design situation together with the design seismic action according to Section 4.4.1. Then, equations (D4.21) and (D4.22) give approximately the same design.

*Clause 4.3.3.5.1(8)*

In buildings which are regular in plan and have completely independent lateral-force-resisting systems in two orthogonal horizontal directions, the seismic action component in each one of these directions does not produce (significant) seismic action effects in the lateral-force-resisting systems of the orthogonal direction. For this reason, for buildings regular in plan with completely independent lateral-force-resisting systems in two orthogonal horizontal directions consisting solely of walls or bracing systems, *Section 4* of EN 1998-1 does not require combining the effects of the two horizontal components of the seismic action.

## 4.10. 'Primary' versus 'secondary' seismic elements

### 4.10.1. Definition and role of 'primary' and 'secondary' seismic elements

*Clauses 4.2.2(1), 4.2.2(3)*

EN 1998-1 recognizes that a certain number of structural elements which are not essential parts of the seismic-resisting structural system of the building may be considered as '*secondary seismic*', as far as their role and contribution to earthquake resistance of the building is concerned. The main objective of this distinction is to allow for some simplification of the seismic design by not considering such elements in the structural model used for the seismic analysis of the building. Accordingly, only the remaining elements, which are termed '*primary seismic members*', should be modelled in the structural analysis and designed and detailed for earthquake resistance in full accordance with the rules of *Sections 5–9* in EN 1998-1.

The differentiation between primary and secondary elements is essentially equivalent to the traditional distinction in US seismic design codes for new buildings between members which belong to the lateral-force-resisting system and those that do not. The terminology of primary and secondary elements has also been adopted by the US prestandard for seismic retrofitting of existing buildings.[45,46] In EN 1998-1, the term 'seismic' has been added to make it clear that the characterization applies only to the seismic action.

The building structure is taken in design to rely for its earthquake resistance only on its primary seismic elements. Cyclic degradation of the strength and/or stiffness of primary seismic elements is disregarded, provided that their dimensioning and detailing fully follows the rules and requirements given in *Sections 5–9* of EN 1998-1 for elements designed for energy dissipation and ductility.

The strength and stiffness of secondary seismic elements against lateral loads is to be neglected in the analysis for the seismic action. However, their contribution in resisting other actions (mainly gravity loads) should be fully accounted for.

The contribution of all secondary seismic elements to the lateral stiffness should be not more than 15% of the lateral stiffness of the system of primary seismic elements. For this requirement to be met, the full structural system, consisting of both primary and secondary seismic elements, should develop lateral drifts less than 1.15 times those developed by the system of primary seismic elements alone. Drifts should be computed for the same system of horizontal forces, acting separately along the two main horizontal axes of the building and having the heightwise distribution of *clause 4.3.3.2.3* (for the lateral force method of analysis), and should be compared at least at roof level, but preferably at all storeys.

*Clause 4.2.2(4)*

### 4.10.2. Special requirements for the design of secondary seismic elements

Secondary seismic elements do not need to conform to the rules and requirements given in *Sections 5–9* of EN 1998-1 for the design and detailing of structural elements for earthquake resistance based on energy dissipation and ductility; they only need to satisfy the rules of the other Eurocodes (2 to 6), plus the special requirement of Eurocode 8 that they maintain support of gravity loads when subjected to the most adverse displacements and deformations induced in them in the seismic design situation. These deformations are determined according to the equal displacement rule, i.e. they may be taken as equal to those computed from the elastic analysis for the design seismic action (neglecting, of course, the contribution of secondary seismic elements to lateral stiffness) multiplied by the behaviour factor, $q$. They should account for second-order ($P$–$\Delta$) effects, by dividing the first-order values by $(1 - \theta)$ if the value of the sensitivity ratio $\theta$ (see equation (D4.20)) exceeds 0.1. *Section 4* of EN 1998-1 refers to the material-specific sections for more detailed application rules.

*Clauses 4.2.2(1), 5.7(1), 5.7(2)*

Such rules are given, though, only in *Section 5* of EN 1998-1 for concrete buildings. However, these rules are general enough to be applicable to all other materials. According to them, internal forces (bending moments and shears) calculated for secondary seismic elements on the basis of the deformations above and their (cracked) flexural and shear stiffness should not exceed the design value of their flexural and shear resistance, $M_{Rd}$ and $V_{Rd}$, respectively, determined according to the material Eurocode (Eurocode 2 in the case of concrete buildings). The implications are twofold:

*Clause 5.7(3)*

- The computer software used for the analysis should have the capability of activating the stiffness matrix of secondary seismic elements, with their full (cracked) stiffness and normal connectivity to their end nodes (regardless of any hinges introduced there in the modelling), to compute their internal forces from the displacements of their real end nodes. This may require a special module for secondary seismic elements, which during the assembly of the global stiffness matrix neglects their flexural and shear stiffness terms and/or considers the member end releases due to fictitious hinges there, but in the phase of back-substitution for the computation of internal forces from end displacements it considers the actual flexural and shear stiffness and/or removes the artificial member end releases.
- Secondary seismic elements are severely penalized by being required to remain elastic in the seismic design situation. This amounts to an overstrength factor of $q$ in these elements, relative to the primary seismic ones, if their strength is controlled by the seismic design situation. Dimensioning of secondary seismic elements for these requirements may not be feasible, unless (1) the global stiffness of the system of primary seismic elements and its connectivity to the secondary seismic ones is such that seismic

deformations imposed on the latter are low; and/or (2) the real flexural and shear stiffness of secondary seismic elements is very low.

## 4.11. Verification

The verification provisions in *Section 4* of EN 1998-1 for buildings elaborate the compliance criteria set out in Sections 2.2.1 for the damage limitation and in Sections 2.2.2.1 and 2.2.2.2 for the no-(local)-collapse requirement. These provisions are presented here in that order, as this is the order normally followed in the design process.

### 4.11.1. Verification for damage limitation

*Clauses 4.4.3.1(1), 4.4.3.2(1)*

The damage limitation requirement for buildings is simply an upper limit on the interstorey drift ratio demand under the frequent (serviceability) seismic action. The limit on the interstorey drift ratio is set equal to:

(1) 0.5%, if there are brittle non-structural elements attached to the structure so that they are forced to follow structural deformations (normally partitions)
(2) 0.75%, if non-structural elements (partitions) attached to the structure as above are ductile
(3) 1%, if no non-structural elements are attached to the structure.

The interstorey drift ratio demand for storey $i$ is determined at the most adverse relevant point in plan, as the ratio of the difference of the lateral displacements at the top and bottom of the vertical element there, $d_i$ and $d_{i-1}$, $\Delta d_i$, divided by the height, $h_i$, of storey $i$. The most adverse relevant point is the one where the interstorey drift ratio attains its maximum value over the part of the plan where the same limit value applies (e.g. if there are no partitions over the part of the plan where interstorey drifts are at a maximum, taking into account the effects of natural and accidental torsion, but ductile partitions are attached to the structure over the rest of the plan, then the maximum interstorey drifts over these two parts of the plan should be separately checked against the corresponding limits).

The interstorey drift ratio demand should be determined under the frequent (serviceability) seismic action, which is defined by multiplying the entire elastic response spectrum of the design seismic action for 5% damping by the same factor $\nu$ that reflects the effect of the mean return periods of these two seismic actions. If the analysis for the design seismic action is linear-elastic based on the design response spectrum (i.e. the elastic spectrum with 5% damping divided by the behaviour factor $q$), then the values of the displacements to be used for $d_i$ and $d_{i-1}$ are those from that analysis multiplied by the behaviour factor $q$. If the analysis is non-linear, the interstorey drift ratio should be determined for a seismic action (acceleration time–history for time–history analysis, acceleration–displacement composite spectrum for pushover analysis) derived from the elastic spectrum (with 5% damping) of the design seismic action times $\nu$. The rules of Section 4.9 should be applied to take into account the effect of the two simultaneous horizontal components of the seismic action on drifts. Interstorey drift demands to be checked against drift limits 1 or 2 listed at the beginning of the present section should be computed within the plane of the relevant partitions attached to the structure. Interstorey drift demands to be checked against drift limit 3 should be computed within the plane of lateral-force-resisting systems, which are normally parallel to the directions of the horizontal components considered.

If there are (brittle or ductile) non-structural elements attached to the structure, and unless most of the lateral force resistance is provided by – concrete, composite or masonry – walls or heavy (steel) concentric bracings, member sizes will be controlled by the limit on interstorey drift ratio. For this reason, compliance with the damage limitation requirement should be established, before proceeding with dimensioning and detailing of members to satisfy the no-collapse requirement. Given the criticality of the damage limitation requirement for member sizing, there is a strong incentive for the designer of concrete

buildings to use the default (high) stiffness of 50% of the uncracked gross section stiffness, instead of pursuing more accurate and representative alternatives that may be less conservative for the force-based dimensioning of members to satisfy the no-collapse requirement, but may make the damage limitation requirement more difficult to meet.

Given the large global stiffness necessary to meet interstorey drift limits, the limits on the sensitivity coefficient $\theta$ for $P$–$\Delta$ effects (an upper limit of 0.3, and geometrically non-linear analysis required if $\theta > 0.2$) are normally not critical for buildings. In fact, equation (D4.20) shows that the value of $\theta$ is equal to the interstorey drift ratio at the storey centre of mass divided by the storey shear coefficient (ratio of storey shear to weight of overlying storeys), both under the design seismic action. So, $P$–$\Delta$ effects may be important at the base (where the storey shear coefficient is minimum), but mainly in moderate-seismicity regions, where the seismic action is relatively low, but not low enough for a 'low-dissipative' design to be used with a low value of $q$.

### 4.11.2. Verification for the no-(local)-collapse requirement

What was said in Section 2.2.2.1 concerning seismic design for energy dissipation (normally through ductility) with a $q$ factor greater than 1.5, and in Section 2.2.2.2 on design without energy dissipation or ductility and with a $q$ factor not greater than 1.5 for overstrength, applies to buildings. The specific rules for the fulfilment of the no-(local)-collapse requirement within the framework of design for energy dissipation and ductility are elaborated further here.

#### *4.11.2.1. Verification in force-based dissipative design with linear analysis*

In the standard case of force-based seismic design based on linear analysis with a $q$ factor value greater than 1.5, the following verifications are performed:

- Dissipative zones are dimensioned so that the design resistance of the ductile mechanism(s) of force transfer, $R_d$, and the design value of the corresponding action effect due to the seismic design situation, $E_d$, from the analysis satisfy equation (D2.3). *Clause 4.4.2.2(1)*
- Regions of the structure outside the dissipative zones and non-ductile mechanisms of force transfer within or outside the dissipative zones are dimensioned to remain elastic until and beyond yielding of the ductile mechanism(s) of the dissipative zones. This is pursued through overdesign of the regions not considered as dissipative zones and of the non-ductile mechanisms of force transfer relative to the corresponding action effect due to the seismic design situation, $E_d$, from the analysis. Normally this overdesign is accomplished through 'capacity design'. In capacity design, the ductile mechanisms of force transfer in dissipative zones are assumed to develop overstrength capacities, $\gamma_{Rd}R_d$, and equilibrium of forces is employed to provide the action effect in the regions not considered as dissipative zones and in the non-ductile mechanisms of force transfer. Capacity design is also used to spread the inelastic deformation demands over the whole structure and to prevent their concentration in a limited part of it. In frames, this is achieved according to the rules and procedures outlined in Section 4.11.2.2 *Clauses 4.4.2.2(2), 4.4.2.2(3), 4.4.2.2(7)*
- Dissipative zones are detailed to provide the deformation and ductility capacity that is consistent with the demands placed on them by the design of the structure for the chosen $q$ factor value.
- The foundation is also capacity designed on the basis of the overstrength of ductile mechanisms of force transfer in dissipative zones of the superstructure. Foundation elements are either capacity designed to remain elastic beyond yielding in dissipative zones of the superstructure or are dimensioned and detailed for energy dissipation and ductility, like the superstructure. *Clauses 4.4.2.6(1), 4.4.2.6(2)*

#### *4.11.2.2. Design strategy for spreading inelastic deformation demands throughout the structure*

According to Section 2.2.1 and equations (D2.1) and (D2.2), buildings designed on the basis of $q$ values higher than 1.5 should be capable of sustaining ductility demands corresponding to a value of the global displacement ductility factor, $\mu_\delta$, about equal to $q$. In a multi-storey *Clause 4.4.2.3(3)*

building, the global displacement ductility factor is defined on the basis of the horizontal displacement of the building (drift) either at the roof or – preferably – at the height of application of the resultant lateral force. The global displacement ductility demand, in terms of $\mu_\delta$, should be spread as uniformly as possible to all storeys of the building. In other words, a storey-sway (or soft-storey) mechanism should be avoided and a beam-sway mechanism should be promoted instead. As shown in Fig. 4.4a, if a soft-storey mechanism develops, the entire inelastic deformation demand will be concentrated there: chord rotations at the ends of the ground storey columns will be equal to $\theta_{st} = \delta/H_{st}$, where $\delta$ is the top displacement (the magnitude of which is essentially determined from the properties of the elastic structure and the elastic response spectrum of the seismic action, irrespective of the inelastic response) and $H_{st}$ is the height of the ground storey; for buildings of more than two storeys, inability of the ground-storey columns to sustain such chord rotation demands will most likely lead to local failures and global collapse. In contrast, in a beam-sway mechanism the global displacement demand is uniformly spread to all storeys, and inelastic deformations and energy dissipation takes place at all beam ends; the kinematics of the mechanism require that vertical elements – which are not only more important for global stability but also inherently less ductile than beams – develop plastic hinging only at the base (Figs 4.4b and 4.4d). Even that hinging may be replaced by rotation of the column footing (Figs 4.4c and 4.4e). In the beam-sway mechanisms of Figs 4.4b to 4.4e the chord rotation at the ends of members where plastic hinges form will be equal to $\theta = \delta/H_{tot}$, where the top displacement $\delta$ is essentially the same as in the soft-storey mechanism of Fig. 4.4a if the properties of the elastic structure and the elastic spectrum of the seismic action are the same, and $H_{tot}$ is the full height of the building.

Eurocode 8 pursues the development of beam-sway mechanisms in multi-storey buildings by providing a stiff and strong vertical spine to them that remains elastic above the base during the response. This is pursued through:

- choices in the structural configuration
- rules for the dimensioning of vertical members so that they form a stiff and strong vertical spine above the base.

More specifically:

(1) In concrete buildings, wall systems (or wall-equivalent dual systems) are promoted, and their walls are (capacity-)designed to ensure that they remain elastic above the base, both in flexure and in shear. In steel and composite (steel–concrete) buildings, frames with concentric or eccentric bracings are promoted, and all members except the few intended for energy dissipation (i.e. except the tension diagonals in frames with concentric bracings or the 'seismic links' in those with eccentric bracings) are designed to remain elastic above the base during the response. These systems are indirectly promoted through the strict interstorey drift limits for the damage limitation seismic action (see Section 4.11.1), which are difficult to meet with frames alone – especially in concrete frames, where the cracked stiffness of members is used in the analysis.

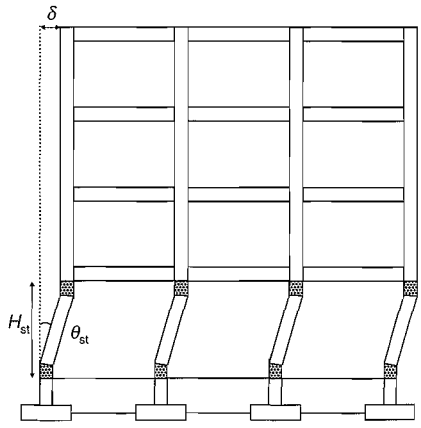
(2) In moment-resisting frame systems (and frame-equivalent dual concrete frames) strong columns are promoted, indirectly through the interstorey drift limits mentioned above, and directly through the capacity design of columns in flexure described in Section 4.11.2.3, so that formation of plastic hinges in columns before beam hinging is prevented.

#### *4.11.2.3. Capacity design of frames against plastic hinging in columns*

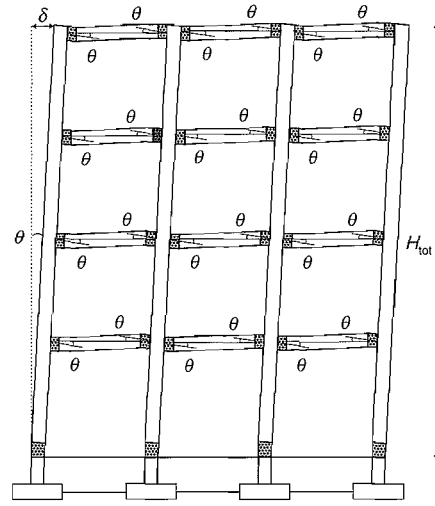
*Clauses 4.4.2.3(4), 4.4.2.3(5), 4.4.2.3(6)*

The objective of the Eurocode 8 rules for the design of (concrete, steel or composite) moment-resisting frames is to force plastic hinges out of the columns and into the beams, so that a beam-sway mechanism develops and a soft storey is prevented. To this end, at their joints with beams, primary seismic columns are (capacity) designed to be stronger than the beams, with an overstrength factor of 1.3 on beam design flexural capacities:

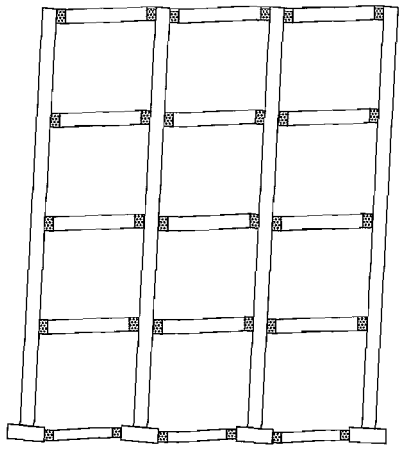
$$\sum M_{Rd,c} \geq 1.3 \sum M_{Rd,b} \qquad \text{(D4.23)}$$



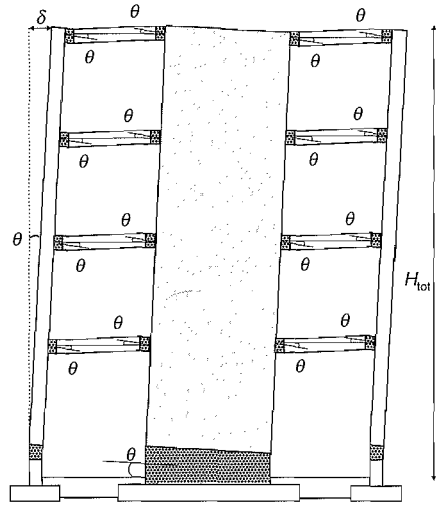

(a)



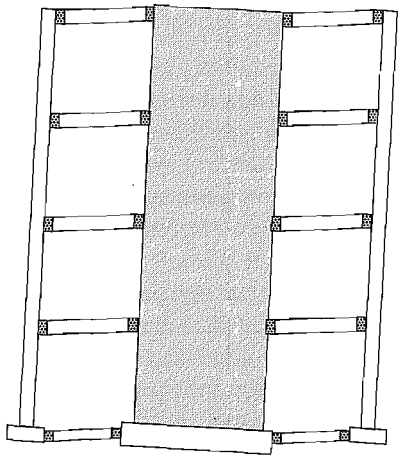

(b)



(c)




(d)



(e)

**Fig. 4.4.** Plastic mechanisms in frame and wall systems: (a) soft-storey mechanism in a weak column/strong beam frame; (b, c) beam-sway mechanisms in a strong column/weak beam frame; (d, e) beam-sway mechanisms in a wall system

where $M_{Rd,c}$ and $M_{Rd,b}$ denote the design value of the flexural capacity of columns and beams, respectively. The summation on the left-hand side extends over the column sections above and below the joint; the summation on the right-hand side extends over all beam ends framing into the joint, regardless of whether they are primary or secondary seismic beams.

Equation (D4.23) has to be verified in each of the two main horizontal directions of the building in plan, or at least in the direction in which the structural type has been characterized as a frame or a frame-equivalent dual system. In each horizontal direction in which equation (D4.23) should be fulfilled, it has to do so first with the column flexural capacities in the positive (clockwise) sense about the normal to the horizontal direction of the frame (or frame-equivalent dual) system and then in the negative (anticlockwise) sense, with the beam flexural capacities always taken to act on the joint in the opposite sense with respect to the column capacities.

If a beam framing into a joint is at an angle $\theta$ to the horizontal direction in which equation (D4.23) is checked, the value of $M_{Rd,b}$ enters into equation (D4.23) multiplied by $\cos\theta$. On the other hand, if the two cross-sectional axes in which the flexural capacities of the column, $M_{Rd,c}$, are expressed are at angles $\theta_1$ and $\theta_2 = 90 + \theta_1$ with respect to the horizontal direction in which equation (D4.23) is checked, these capacities should enter equation (D4.23) multiplied by $\sin\theta_1$ and $\sin\theta_2$, respectively.

Fulfilment of equation (D4.23) is not required at the joints of the top floor. In fact, it does not make any difference to the plastic mechanism whether the plastic hinge will form at the top of the top storey column or at the ends of the top floor beams. After all, it is difficult to satisfy equation (D4.23) there, as only one column enters in the summation of the left-hand side.

#### 4.11.2.4. *Verification of the foundation and design and detailing of foundation elements*

*Clauses 4.4.2.6(1), 4.4.2.6(2), 4.4.2.6(3)*

Due to the importance of the foundation for the integrity of the whole building structure, and the difficulty to access, inspect and repair damaged foundation systems, the verification of the foundation of buildings designed for energy dissipation is based on seismic action effects derived from capacity design, on the basis of the overstrength capacity of the yielding elements of the superstructure. This always applies to the verification of the foundation soil and, in general, for the dimensioning of the foundation elements. This is in the opposite direction to US codes,[39,40] which allow reduction of overturning moment at the base due to uplift by 25% for linear static analysis or by 10% for a response spectrum analysis.

Wherever the seismic action effects determined for the foundation or its elements according to capacity design exceed the corresponding value from the analysis for the design seismic action without reduction by the behaviour factor $q$, then this latter – smaller – value may be used as seismic demand in the verifications. This applies to individual parts of the foundation and individual foundation elements. Moreover, the option is given to calculate the seismic action effects for the entire foundation system from the analysis for the design seismic action using $q = 1.5$ and completely neglecting capacity design. This option is consistent with the way seismic action effects are calculated in buildings which are designed as 'low-dissipative' according to Section 2.2.2.2. This is not a viable alternative, though, in high-seismicity regions, especially for medium- or high-rise buildings, as the seismic action effects resulting from the application of $q = 1.5$ in the entire foundation system may be so high that verification of some parts of the foundation system may be unfeasible.

*Clause 4.4.2.6(4)*

For the foundation of individually founded vertical elements (essentially for individual footings) the seismic action effects determined through capacity design are calculated assuming that seismic action effects from the elastic analysis increase proportionally until the dissipative zone or element that controls the seismic action effect of interest reaches the design value of its force capacity, $R_{di}$, and is, indeed, increased by an overstrength factor $\gamma_{Rd}$, which is taken equal to $\gamma_{Rd} = 1.2$ if the value of the $q$ factor used in the design of the superstructure exceeds 3. This is achieved by multiplying all seismic action effects from the analysis by the value $\gamma_{Rd}\Omega = \gamma_{Rd}(R_{di}/E_{di}) \leq q$, where $E_{di}$ is the seismic action effect from the elastic analysis in the dissipative zone or element controlling the seismic action effect of interest.

In individual footings of walls or of columns of moment-resisting frames, $\Omega$ is taken as the minimum value of the ratio $M_{Rd}/M_{Ed}$ in the two orthogonal principal directions at the lowest cross-section of the vertical element where a plastic hinge can form in the seismic design situation, as it is in that direction that the element will first develop its force capacity. The value of $M_{Rd}$ should be determined assuming that the axial force in that section of the vertical element is equal to the value from the analysis for that particular seismic design situation. In individual footings of columns of steel or composite braced frames, $\Omega$ is taken as the minimum value of the force capacity to the corresponding value from the analysis in the seismic design situation, among all intended dissipative zones in the braced frame. If it is a concentric braced frame, $\Omega$ is the minimum value of the ratio $N_{pl,Rd}/N_{Ed}$ over all diagonals of the entire braced frame which are in tension for that particular seismic design situation, as only the tensile diagonals are intended for energy dissipation in such frames. If the braced frame is eccentric, $\Omega$ is the minimum value of the ratio $V_{pl,Rd}/V_{Ed}$ over all plastic shear zones and of $M_{pl,Rd}/M_{Ed}$ over all plastic hinge zones in this particular braced frame, where $V_{pl,Rd}$ and $M_{pl,Rd}$ denote the design value of the plastic shear or moment resistance, respectively, of seismic links in the eccentric frame, as these may depend on the axial load in the seismic link from the analysis for the particular seismic design situation. Implicit in such calculations of $\Omega$ is the assumption that the action effect of gravity loads present in the seismic design situation is negligible in comparison to $R_{di}$ and $E_{di}$. In connecting beams between individual footings, seismic action effects from the analysis should also be multiplied by the value of $\gamma_{Rd}\Omega$ derived from the nearest individual footing for that particular seismic design situation.

*Clauses 4.4.2.6(5), 4.4.2.6(6), 4.4.2.6(7)*

For common foundations of more than one vertical element (e.g. in rafts, foundation beams and strip footings) the value of $\Omega$ derives from the vertical element that develops the largest seismic shear in the seismic design situation. Alternatively, the value of $\gamma_{Rd}\Omega$ may be taken equal to 1.4, meaning that the seismic action effects from the analysis are magnified by 1.4, without any capacity design calculations.

*Clause 4.4.2.6(8)*

All seismic action effects in the foundation system or element of interest are multiplied by the value of $\gamma_{Rd}\Omega$ applicable to that particular design situation. For an individual footing this includes the seismic action effects transmitted from the vertical element and any tie beams to the footing and all components of the reaction from the ground. The implication is that if the vertical seismic reaction is tensile, the eccentricity of the total vertical reaction due to the combination of gravity loads and the vertical seismic reaction multiplied by $\gamma_{Rd}\Omega$ may be large.

#### *4.11.2.5. Verification in displacement-based dissipative design on the basis of non-linear analysis*

EN 1998-1 allows design on the basis of non-linear analysis (mainly of the pushover type) without the use of the behaviour factor $q$. In that case, verification for the no-(local-)collapse requirement comprises the following:

*Clauses 4.4.2.2(5), 4.3.3.1(4)*

(1) Brittle elements or mechanisms of force transfer are verified via equation (D2.3) expressed in terms of forces, with design action effects, $E_d$, as obtained from the non-linear analysis for the seismic design situation (taking into account second-order effects, as appropriate), and design resistances, $R_d$, determined as for linear analysis, including the same partial factors for the materials.

(2) Dissipative zones, which are designed and detailed for ductility, are verified via equation (D2.3) expressed in terms of member deformations (e.g. plastic hinge or chord rotations), taking as design action effects, $E_d$, the deformations obtained from the non-linear analysis for the seismic design situation (including second-order effects, as appropriate), and as design resistances, $R_d$, the design values of member deformation capacities (including appropriate partial factors on deformation capacities).

(3) All the material-specific rules given in *Sections 5–9* of EN 1998-1 for dissipative seismic design should be verified. These rules include the minimum requirements for materials, member geometry and detailing, etc. for DCM, as well as fulfilment of equation (D4.23)

*Clause 4.3.3.1(6)*

at the joints of moment resisting frames (or frame-equivalent dual concrete systems). They also include the magnification of shear forces in concrete walls of DCM, but do not include the determination of design shears in concrete beams or columns by capacity design, as this is explicitly covered by point 1 above. They do not include, either, the determination of confinement reinforcement in the plastic hinge or other dissipative zones of concrete walls or columns as a function of the curvature ductility factor, as this is determined from the behaviour factor $q$, because this factor is not relevant in this case. The deformation-based verification of dissipative zones according to point 2 covers this requirement in a more direct way.

*Clauses 4.4.2.3(8), 4.4.2.3(3)*

(4) The plastic mechanism predicted to develop in the seismic design situation is satisfactory, in the sense that soft-storey plastic mechanisms or similar concentrations of inelastic deformations are avoided.

Fulfilment of the requirements and verification according to point 3 above may appear as superfluous or even onerous, in view of the fulfilment of all the other conditions. However, this requirement has been introduced to ensure that the final design will possess the global ductility and deformation capacity which is implicitly required as a safeguard against global collapse under a seismic action much stronger than the design earthquake. With the accumulation of experience of design on the basis of non-linear analysis without the $q$ factor, these minimum requirements may be refined, revised or even abolished.

*Clause 4.3.3.1(4)*

By allowing design on the basis of non-linear analysis (mainly of the pushover type) without the use of the behaviour factor $q$, EN 1998-1 is taking the bold step of introducing displacement-based design for new buildings. However, this step is incomplete, as specific information on capacities in terms of deformations is not given and the task is delegated to National Annexes, in which individual countries are requested to specify (through reference to relevant sources of information) these capacities, along with the associated partial factors on deformation capacities. Fortunately, in the meantime, Part 3 of Eurocode 8[52] has filled this gap. Being fully displacement based, that part of Eurocode 8 gives in informative annexes the ultimate deformation capacities of concrete, steel (and composite) and masonry elements, as well as partial factors on these capacities for the 'significant damage' limit state, which is defined (in a note in the normative part of EN 1998-3[52]) as equivalent to the ultimate limit state for which the no-(local-)collapse requirement should be verified in new buildings according to EN 1998-1. The information in these annexes may provide guidance for the National Annexes to EN 1998-1, or even be directly adopted by them for the deformation capacities of members and the associated partial factors.

#### *4.11.2.6. Verification of seismic joint with adjacent structures or between structurally independent units of the same building*

*Clauses 4.4.2.7(1), 4.4.2.7(2)*

Buildings are designed as separate structural units, independent from adjacent ones. To make sure that the structural model adopted for the analysis applies and to prevent any unforeseen consequences of dynamic interaction of the response with that of adjacent structures, EN 1998-1 requires securing a minimum spacing from such structures. The space is meant to be provided between the structures and may be filled, locally or fully, by a non-structural material which offers little resistance to compression in the event of an earthquake.

If the building being designed and that adjacent to it belong to the same property, or is a structurally independent unit of the same building, then the designer has full access to the information necessary for the construction of a full structural model of both buildings or structurally independent units and their analysis for the design seismic action. Then, he or she may compute the maximum horizontal displacements of both buildings or units normal to the vertical plane of the joint between them under the design seismic action. If the analysis for the design seismic action is linear, based on the design response spectrum (i.e. the elastic spectrum with 5% damping divided by the behaviour factor $q$), then the value of the floor displacement under the design seismic action is that from the analysis multiplied by the behaviour factor $q$ adopted in the horizontal direction normal to the vertical plane of the

seismic joint. If the analysis is non-linear, the floor displacements are determined directly from the analysis for the design seismic action. The rules of Section 4.9 should be applied to take into account the effect of the two simultaneous horizontal components of the seismic action on floor drifts. Unless the analysis is of the response time–history type, it only provides the peak value(s) of floor drifts during the response. To account for the fact that these peak values do not take place simultaneously, the width of the seismic joint is taken as the SRSS of the peak horizontal displacements of the two buildings or units at the corresponding level normal to the vertical plane of the joint.

If the building being designed and that adjacent to it do not belong to the same property, the owner and the designer normally do not have the information necessary for the calculation of the peak horizontal displacement of the other building or unit normal to the vertical plane of the joint. Even if they have access to such information, they normally have no control over future developments on the other side of the property line. So, EN 1998-1 simply requires the designer to provide a distance from the property line to the potential points of impact at least equal to the peak horizontal displacement of his or her building at the corresponding level, calculated according to the previous paragraph. This ends his or her responsibility, even when the structure of the adjacent building or unit has been built up to the property line.

*Clause 4.4.2.7(3)*

Apart from the uncertainty created about the validity of the structural model and of the predictions of the seismic response analysis, dynamic interaction with adjacent buildings normally does not have catastrophic effects. On the contrary, given that it is only a few buildings that collapse even under very strong earthquakes, weak or flexible buildings may be spared by being in contact with adjacent strong and stiff buildings on both sides. For this reason, EN 1998-1 allows reducing the width of the seismic joint calculated according to the previous two paragraphs by 30%, provided that there is no danger of the floors of one building or independent units ramming vertical elements of the other within their clear height. So, if the floors of the two adjacent buildings or units overlap in elevation, just 70% of the width of the seismic joint calculated according to the previous two paragraphs needs to be provided.

## 4.12. Special rules for frame systems with masonry infills

### 4.12.1. Introduction and scope

*Clause 2.2.2(6)*

Field experience and analytical and experimental research have demonstrated the overall beneficial effect of masonry infills attached to the structural frame on the seismic performance of buildings, especially when the building structure has little engineered earthquake resistance. If they are effectively confined by the surrounding frame, infill panels reduce, through their in-plane shear stiffness, storey drift demands, increase, through their in-plane shear strength, the storey lateral force resistance and contribute, through their hysteresis, to the global energy dissipation capacity.[58] In buildings designed for earthquake resistance, non-structural masonry infills normally constitute a second line of defence and a source of significant overstrength. EN 1998-1 adopts this attitude, and does not encourage the designer to reduce the earthquake resistance of the structure to account for the beneficial effects of masonry infills.

If the contribution of masonry infills to the lateral strength and stiffness of the building is large relative to that of the structure itself, the infills may override the seismic design of the structure and invalidate both the efforts of the designer and the intention of Eurocode 8 to control the inelastic response by spreading the inelastic deformation demands throughout the structure and the building. For instance, loss of integrity of ground storey infills will produce a soft storey there, and may trigger collapse of the structural frame itself. Concentration of inelastic deformation demands in a small part of the building is much more likely if the infills are not uniformly distributed in plan or – more importantly – in elevation. This situation may also have serious adverse effects on seismic performance and safety. Last

but not least, the infills may have adverse local effects on the structural frame, possibly causing pre-emptive brittle failures. It is against such local or global adverse effects that EN 1998-1 strives to provide safeguards, in the form of guidance to the designer or even mandatory rules.

*Clauses 4.3.6.1(1), 4.3.6.1(2), 4.3.6.1(4)*

The rules of EN 1998-1 for buildings with masonry infills are mandatory when the structure itself is designed for relatively low lateral force stiffness and strength but for high ductility and deformation capacity. This is the case for unbraced moment frame systems (in concrete, also of frame-equivalent dual systems) designed for DCH, i.e. for high ductility and a high value of the $q$ factor. Structural systems of lower ductility class (DCL or DCM) are considered as designed for lateral strength which is sufficient to overshadow that of infill walls. Steel or composite frames with concentric or eccentric bracings and concrete wall (or wall-equivalent dual) systems are also considered as stiff enough not to be affected by the presence of masonry infills. For these two categories of structural systems, the safeguards specified by Eurocode 8 against the negative effects of infill walls are not mandatory; however, the designer is advised to consider them as guidance for good practice.

*Clause 4.3.6.1(5)*

If structural connection is provided between the masonry and the surrounding frame members (through shear connectors, or other ties, belts or posts), then the structure should be considered and designed as a confined masonry building, rather than as a concrete, steel or composite frame with masonry infills.

### 4.12.2. Design against the adverse effects of planwise irregular infills

*Clauses 4.3.6.2(1), 4.3.6.3.1(1), 4.3.6.3.1(4)*

An unsymmetric distribution of the infills in plan may cause torsional response to the translational horizontal components of the seismic action. Obviously, due to the torsional component of the response, structural members on the side of the plan which has fewer infills (termed the 'flexible' side in torsionally unbalanced structures) will be subjected to larger deformation demands than those on the opposite, heavier infilled side. Analytical and experimental research[59,60] has shown that the increase in lateral strength and stiffness due to the infills compensates for the uneven distribution of interstorey drift demands over the plan. In other words, the maximum member deformation demands in the presence of planwise irregular infilling normally do not exceed (at least by much) the peak demands anywhere in plan in a similar structure without the infills. Nevertheless, as local deformation demands might exceed those estimated from an analysis that neglects the infills, EN 1998-1 requires doubling of the accidental eccentricity of Section 4.8 in the analysis of the structural system that neglects planwise irregular infills. This does not unduly penalize either the design procedure or the structural system; it is also quite effective, especially when the structural system is almost fully symmetric and planwise regular and its analysis without accidental eccentricity predicts a response without any torsional features.

*Clauses 4.3.6.3.1(2), 4.3.6.3.1(3)*

*Section 4* of EN 1998-1 distinguishes the case of severe irregularities in plan due to the unsymmetrical arrangement of the infills. As an example, it mentions infills concentrated along two consecutive sides of the perimeter of the building, as may be the case at the corners of blocks of buildings which are practically in contact with each other. In fact, there are – not fully substantiated – claims that such buildings have a larger incidence of severe damage or collapse, although such claims often attribute the difference to pounding. Anyway, EN 1998-1 does not consider the doubling of the accidental eccentricity as sufficient for such cases. It requires instead analysis of a 3D structural model that explicitly includes the infills; moreover, given the uncertainty about the properties, the modelling and even the future configuration of the infills (including the presence and size of windows), it also requires a sensitivity analysis of the effect of the stiffness and the position of the infills. It mentions disregarding one out of three or four infill panels per planar frame, especially on the more flexible sides, as (a main) part of this sensitivity analysis. Unfortunately, other than stating that infill panels with more than one significant opening or perforation (doors, windows, etc.) should not be included in the model, Eurocode 8 itself does not provide any guidance on modelling infill panels. For cases where the National Annex does not provide reference to

literature on mechanical models for the (masonry) infill panels, relevant guidance for the designer is given in the following paragraph.

A solid infill panel can be conveniently modelled as a diagonal strut along its compressed diagonal. *Section 5* of EN 1998-1 addressing concrete buildings alludes to application of the beam-on-elastic-foundation model[61] for the estimation of the strut width. As an alternative, *Section 5* of EN 1998-1 allows taking the strut width as a fixed fraction of the length of the panel diagonal. A value of the order of 15% of this diagonal is quite representative. *Clause 5.9(4)*

Within the framework of linear analysis the strut may be considered as elastic, with a cross-sectional area equal to the wall thickness $t_w$ times the strut width, and modulus $E$ that of the infill masonry. The strength of the infill – for non-linear analysis, or verification of the infill, or calculation of its local effects on the surrounding frame members – may be taken as equal to the horizontal shear strength of the panel (shear strength of bed joints times the horizontal cross-sectional area of the panel) divided by the cosine of the angle, $\theta$, between the diagonal and the horizontal.

Eurocode 8 draws the attention of the designer to the verification of structural elements furthest away from the side where the infills are concentrated (the 'flexible side') for the effects of torsional response due to the infills. For severe irregularity in plan due to concentration of stiff and strong infills along two consecutive sides of the perimeter of the building, the response due to the translational horizontal components of the seismic action is nearly torsional about the corner where these two sides meet. It turns out that in the vertical elements at or close to that corner the peak deformation and internal force demands computed for separate action of these two components on the system without the infills take place simultaneously.[59,60] So, regardless of whether the infills are taken into account or not in a 3D structural model, the seismic action effects (bending moments and axial forces) due to the two horizontal components in these vertical structural elements would be better taken to occur simultaneously, instead of combined in accordance with Section 4.9. *Clause 4.3.6.3.1(2)*

### 4.12.3. Design against the adverse effects of heightwise irregular infills

A soft and weak storey may develop wherever the infills are reduced relative to the other storeys (notably the overlying storey). The consequences for the global seismic performance are most critical in buildings with an (almost) open ground storey, which, unfortunately, seems to be the most common case of infill irregularity in elevation. *Clause 4.3.6.2(2)*

A reduction of the infills in a storey relative to adjacent storeys increases the inelastic deformation demands on the columns of the storey with the reduced infills, owing to:

- the concentration of the global lateral drift demands to that particular storey (soft/weak-storey effect)
- the near-fixity conditions of the columns of that storey at floor levels, due to the restraint of drift in the neighbouring storeys by the infill panels.

Unlike columns, floor beams above and below that storey are protected from excessive damage owing to the low magnitude of their chord rotation demands. Moreover, the columns of storeys with reduced infills cannot be effectively protected from plastic hinging through application of equation (D4.23). The reason is as follows.[62] As the storeys above and below that with the reduced infills develop low interstorey drift ratio(s), the chord rotations at the ends of the columns of these storeys will also be very low. In fact, if the infills of these storeys are very stiff and strong, column chord rotations there may have a sign opposite that of the beams, so that their algebraic sum indeed gives a low interstorey drift ratio. As the magnitude of moments at column ends is directly related to that of chord rotations there, the end sections of columns in the storeys with reduced infills will get very little aid from the other column section across the joint in resisting the sum of beam flexural capacities, $\sum M_{Rb}$, around the joint, without yielding.[59,62] The end result is that, despite fulfilment of equation (D4.23) at the joints of the frame, plastic hinges may develop at both the top and bottom of the columns of the storey with the reduced infills; moreover, chord rotation demands at these plastic hinges may be large enough to exhaust the corresponding capacities. The outcome may be storey collapse.

*Clauses 4.3.6.3.2(1), 4.3.6.3.2(2), 4.3.6.3.2(3)*

To safeguard against the possibility that the columns of a storey where infills are reduced relative to the overlying storey will develop pre-emptive plastic hinging that may lead to failure, EN 1998-1 calls for these columns to be designed to remain elastic until the infills in the storey above attain their ultimate force resistance. To achieve this, the deficit in infill shear strength in a storey should be compensated for by an increase in resistance of the frame (vertical) members there. More specifically, the seismic internal forces in the columns (bending moments, axial forces, shear forces) calculated from the analysis for the design seismic action are multiplied by the factor $\eta$:

$$\eta = 1 + \frac{\Delta V_{\mathrm{Rw}}}{\sum V_{\mathrm{Ed}}} \leq q \qquad \text{(D4.24)}$$

where $\Delta V_{\mathrm{Rw}}$ is the total reduction of the resistance of masonry walls in the storey concerned, compared with the storey above, and $\sum V_{\mathrm{Ed}}$ is the sum of seismic shear forces on all vertical primary seismic members of the storey (storey design shear force). If the value of the factor $\eta$ turns out to be lower than 1.1, the magnification of seismic action effects may be omitted.

CHAPTER 5

# Design and detailing rules for concrete buildings

## 5.1. Scope

This chapter covers the design of concrete buildings for earthquake resistance according to the provisions of *Section 5* of EN 1998-1. It summarizes the important points of *Section 5* without repeating them, and provides comments and explanations for their application, as well as background information. *Clause 5.1.1*

The scope of *Section 5* in EN 1998-1 covers buildings made of cast-in-place or precast concrete. It is stated clearly in *Section 5* that its provisions do not fully cover buildings in which 'flat slab frames' (i.e. frames of columns connected through flat slabs, instead of beams) are used as primary seismic elements. In such frames, strips of the flat slab between columns act and behave like beams in the event of an earthquake. The effective width of such strips increases with the magnitude of the seismic demands, as measured in this case by interstorey drift; nonetheless, it is very uncertain. There is also large uncertainty about the behaviour of these strips under inelastic cyclic loading, and especially of the regions around the columns. Irrespective of this uncertainty, the stiffness and flexural capacity of these strips is relatively low compared with the columns, conducive to a beam mechanism with column plastic hinging only at the base, as in a strong-column–weak-beam design. However, due to the flexibility of the strips of the flat slab that act like beams, such frames may develop large second-order ($P$–$\Delta$) effects.

Although not explicitly excluded from the scope of *Section 5*, the use of prestressing in primary seismic elements is not fully covered in EN 1998-1. In buildings, prestressing could conceivably be used to advantage in long-span primary seismic beams. However, it is mainly at the ends of beams that plastic hinges are expected to form in the event of an earthquake, and *Section 5* indeed gives rules for the design and detailing of the end regions of primary seismic beams for ductility and energy dissipation. These rules are limited to reinforced concrete beams, hence the implicit exclusion of the use of prestressing in primary seismic elements.

Concrete buildings designed according to *Section 5* for energy dissipation may include flat slabs or prestressed concrete beams, provided that these elements as well as the columns connected to them are considered and designed as secondary seismic elements. As an alternative, concrete buildings with flat slabs or prestressed concrete beams may be designed considering all elements as primary seismic ones, but for almost fully elastic response under the design seismic action, i.e. for Ductility Class Low (DCL) and a value of the behaviour factor $q$ of not more than 1.5. It should be recalled, though, that this alternative is recommended in EN 1998-1 only for low-seismicity regions.

## 5.2. Types of concrete elements – definition of 'critical regions'

*Clause 5.1.2*

*Section 5* categorizes primary seismic concrete elements into beams, columns and walls, in order to prescribe distinctly different sets of design and detailing rules for each of these element types.

### 5.2.1. Beams and columns

A beam is defined as a generally horizontal element which is subjected mainly to transverse loading **and** does not develop significant axial compression in the 'design seismic situation' (a limit of 0.1 is prescribed for the normalized axial compression $\nu_d = N_{Ed}/A_c f_{cd}$ of a beam in the design seismic situation). In contrast, a column is defined as a generally vertical element which supports gravity loads by axial compression **or** develops a non-negligible axial compression in the design seismic situation ($\nu_d$ greater than the above limit of 0.1). This definition will not (re-)classify as a 'beam' any column which is lightly loaded, for example at the top storey(s) of a building, even though it may also carry transverse loads. It will classify, though, as a 'column' any element with significant axial compression, vertical, horizontal or inclined, with or without transverse loading.

### 5.2.2. Walls

Elements which are normally vertical and support other elements are classified as walls, if their cross-section has an aspect ratio (ratio of the two sides) above 4. Obviously, if the cross-section consists of rectangular parts, one of which has an aspect ratio greater than 4, the element is also classified as a wall. With this definition, on the basis of the shape of the cross-section alone, a wall differs from a column in that it resists lateral forces primarily in one horizontal direction, namely that of the long side of the cross-section, and, furthermore, that it can be designed for such a unidirectional resistance by assigning flexural resistance to the opposite ends of the section ('flanges', or 'tension and compression chords') and shear resistance to the 'web' in-between, as in a beam. Concentration of longitudinal (i.e. vertical) reinforcement and concrete confinement is needed only at the two ends of the section providing the flexural capacity. If the cross-section is not elongated, the vertical element develops significant lateral force resistance in both horizontal directions; it is then meaningless to distinguish between flanges, where longitudinal reinforcement is concentrated and concrete is confined, and webs, where the aforementioned do not occur

The above definition of walls is consistent with that in EN 1992-1-1 (clause 9.6.1(1)), and may be appropriate as far as dimensioning and detailing at the level of the cross-section is concerned. It is not very meaningful, though, in view of the intended role of walls in the structural system and of their design, dimensioning and detailing as an entire element, and not just at the cross-sectional level. In fact, if at least 50% of the seismic base shear in a horizontal direction is resisted by concrete walls (see the definition of wall-equivalent dual systems below), then EN 1998-1 relies on these walls alone for the prevention of a storey mechanism in that direction, without any additional verification: the check that plastic hinges will form in beams rather than in primary seismic columns, equation (D4.23), is waived. However, walls can meet the objective of enforcing a beam-sway mechanism only if they act as vertical cantilevers (i.e. if their bending moment diagram does not change sign within at least the lower storeys, see Fig. 5.1) and develop plastic hinging only at the base (at their connection to the foundation). The assumption that walls, as defined above, will indeed act as vertical cantilevers and form a plastic hinge only at the base, underlies all the rules in *Section 5* for the design and detailing of 'ductile walls'. However, whether this assumption corresponds or not to the real behaviour of the wall depends not so much on the aspect ratio of its section but primarily on how stiff and strong the wall is, compared with the beams it is connected to at storey levels. For concrete walls to play the role intended for them by EN 1998-1 and fulfil its tacit assumptions, the length dimension of their cross-section, $l_w$, should be large, not just relative to its thickness, $b_w$, but in absolute terms. To this end, and

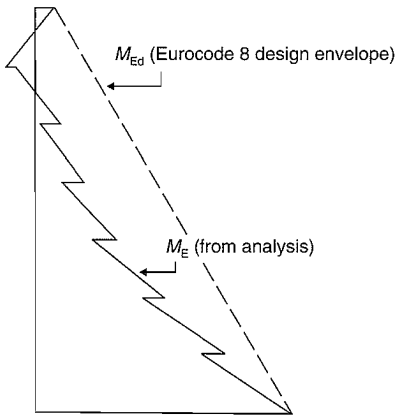



**Fig. 5.1.** Typical bending moment diagram in a concrete wall from the analysis and linear envelope according to Eurocode 8

for the beam sizes commonly found in buildings, a value of at least 1.5 m for low-rise buildings or 2 m for medium- or high-rise ones is recommended here for $l_w$.

A distinction is made in *Section 5* between 'ductile walls' and 'large lightly reinforced walls'. Ductile walls are further classified as 'coupled' or 'uncoupled'.

### 5.2.3. Ductile walls: coupled and uncoupled

The main type of wall according to *Section 5* is the ductile wall, designed and detailed to dissipate energy in a flexural plastic hinge only at the base and to remain elastic throughout the rest of its height, in order to promote – or even force – a beam-sway plastic mechanism: for a flexural plastic hinge with high ductility and dissipation capacity to develop at the base, the ductile wall should be fixed there so that relative rotation of its base with respect to the rest of the structural system is prevented. Moreover, the zone just above the base of the ductile wall should be free of openings or large perforations that might jeopardize the ductility of the plastic hinge.

Two or more individual ductile walls connected through – more or less – regularly spaced beams meeting special ductility conditions ('coupling beams') may be considered as a single element termed a 'coupled wall', provided that their connection through the coupling beams reduces by at least 25% the sum of bending moments at the base of the individual walls, compared with the case when they are working separately. It is noted that the total bending moment at the base of a coupled wall is equal to the sum of the base moments of the individual walls plus the couple moment of the axial forces that develop in the individual walls due to the coupling beams. (The shear forces in the string of coupling beams above the base accumulate into axial forces in the individual walls connected by them, positive in one of the walls, negative in the other; the couple moment of these axial forces is the contribution of the coupling beams to the total bending moment of the coupled wall.) Strictly speaking, to check whether an ensemble of walls meets the criteria of a coupled wall, the analysis of the structural system for the horizontal design seismic action should be repeated, with the coupling beams removed from the model. Moreover, if there are several candidate coupled walls in the building, this exercise has to be performed separately for each of them. Conclusions are not expected to change if the characterization of the walls as coupled or not is based on a single analysis of the structural system including the coupling beams and a comparison of the sum of the resulting bending moments at the base of the individual walls to 75% of the total bending moment at the base of the candidate coupled wall (sum of the base moments of the individual walls plus the moment of their axial forces with respect to the centroid of the section of the candidate coupled wall). After all, and notwithstanding the significant enhancement of wall ductility brought about by the coupling, the characterization of the walls as coupled has minor impact on the design. The only practical consequence is

that the $q$ factor of structural systems in which more than 65% of the seismic base shear is resisted by walls (the wall system, see Section 5.3) is reduced by 10–20%, if more than 50% of the wall resistance is provided by uncoupled rather than by coupled walls.

Because in coupled walls more energy is normally dissipated in the coupling beams than in the plastic hinges at the base of the individual walls, the coupling beams are equally important as these walls, and *Section 5* has special dimensioning and detailing provisions for them. (In fact, the couple moment of the axial forces in the individual walls, on which a lower limit of 25% of the total bending moment of the individual walls is placed for the wall to be considered as coupled, is simply the sum of bending moments at the two ends of all coupling beams, transferred from the face of the individual walls to their axes.) No special rules are given for the individual walls, though. Despite their action as a system, these walls are dimensioned in bending and shear as if they were separate. However, the values of the bending moment and the axial force for which the vertical reinforcement is dimensioned do of course reflect the coupling, at least as far as this is captured by the elastic analysis. It should be noted that, because the axial force in the individual walls from the analysis for the design seismic action is large, there is often a large difference between the absolutely maximum and minimum axial forces in the individual walls in the seismic design situation (including the axial force due to gravity loads). As the vertical reinforcement at the base of each individual wall is controlled by the case in which the bending moment from the analysis, $M_{Edo}$, is combined with the minimum axial compression (or maximum axial tension), the flexural capacity when the maximum axial compression is considered at the base, $M_{Rdo}$, is much larger than $M_{Edo}$. This has serious repercussions on the design of walls of Ductility Class High (DCH) in shear, as in these walls the capacity design magnification factor $\varepsilon$ applied to shear forces from the analysis, $V_{Ed}$, depends on the ratio $M_{Rdo}/M_{Edo}$ (see equations (D5.17) and (D5.18)). In some cases the value of $\varepsilon$ may become so high that the verification of the individual walls in shear (especially against failure due to diagonal compression) may be unfeasible. The (up to 30%) redistribution of bending moments $M_{Edo}$ from the individual wall with the low axial compression (or net axial tension) to the one with the high axial compression, as recommended for coupled walls in *clause 5.4.2.4(2)* of EN 1998-1, may be used to advantage; however, the advantage is limited by the need to redistribute shear forces from the analysis along with the bending moments. So, if the moment acting on the wall together with the low axial compression is reduced to $0.7M_{Edo}$ and that on the wall with the high axial compression is increased to $1.3M_{Edo}$, the flexural capacity will decrease to $M'_{Rdo} < M_{Rdo}$, and the magnification factor $\varepsilon$, which depends on $M'_{Rdo}/1.3M_{Edo}$, will decrease even more. The reduced magnification factor will be applied, though, on $1.3V_{Ed}$, and the benefit to the shear verification will be limited.

The conclusion is that, despite the generally recognized enhancement of seismic performance brought about by coupling the walls, the current provisions in *Section 5* do not offer real incentives for the use of coupled walls, especially in buildings of DCH.

### 5.2.4. Large lightly reinforced walls

Walls with a large horizontal dimension compared with their height cannot be designed effectively for energy dissipation through plastic hinging at the base, as they cannot be easily fixed there against rotation relative to the rest of the structural system. Design of such a wall for plastic hinging at the base is even more difficult if the wall is monolithically connected with one or more transverse walls also large enough not to be considered merely as flange(s) or rib(s) of the first wall. *Section 5* recognizes that such walls, due to their large dimensions, will most likely develop limited cracking and inelastic behaviour in the seismic design situation. Cracking is expected to be mainly horizontal and to coincide with construction joints at floor levels. Flexural yielding, if it occurs, will also take place mainly at these locations. Then, the lateral deflections of large walls, acting as vertical cantilevers, will be produced through a combination of (1) a rotation of the foundation element of the wall relative to the ground, most often with partial uplifting from the ground, and (2) similar rotations concentrated at the locations of horizontal cracking and possibly flexural yielding at one or more floor levels,

with the wall swaying in a multi-rigid-block fashion. Due to the relatively low axial load level in large walls, all these rotations will take place about a 'neutral axis' very close to the compressed tip of the foundation element or the compressed edge of the wall section at the locations of cracking and (possibly) yielding. Such rotations induce significant uplift of the centroid of the sections, raising the floor masses which are tributary to the wall and the ends of beams framing into it, to the benefit of the global response and the stability of the system. For example, part of the input seismic energy is – be it temporarily – harmlessly transformed to potential energy of these tributary masses, in lieu of damaging deformation energy of the wall itself. Moreover, rigid-body rocking of the wall promotes radiation damping, which is particularly effective for reducing the high-frequency components of the input motion.

*Section 5* recognizes the capability of large walls to withstand strong seismic demands, through their geometry, rather than via the strength and hysteretic dissipation capacity provided by reinforcement. It defines a 'large lightly reinforced wall' as a wall with horizontal dimension, $l_w$, at least equal to 4.0 m or to two-thirds of its height, $h_w$ (whichever is less), and provides it with a special role and special design and detailing rules (that result in much less reinforcement than for ductile walls), under the condition that this type of wall is used in a lateral-force-resisting system consisting mainly of such walls (see the definition of the system of large reinforced walls in Section 5.3).

### 5.2.5. Critical regions in ductile elements

The primary, if not the only, mode in which concrete elements can dissipate energy is in bending. Energy dissipation takes place in alternate positive and negative bending at flexural plastic hinges at member ends – although long-span beams also subjected to significant transverse loading may develop one-sided plastic hinges in positive bending at some distance from their end sections. In *Section 5*, dissipative zones in concrete elements are termed 'critical regions'. As used in *Section 5*, the term has a more conventional connotation than the term 'dissipative zone', which is used in *Sections 6–8* of EN 1998-1 to denote the – rather loosely defined – part of an element or connection where energy dissipation will take place by design. In *Section 5*, critical regions are conventionally defined parts of primary seismic elements, up to a certain length from the end section – or in beams from the section of maximum positive (hogging) bending moment under the combination of transverse loads and the design seismic action. The length of critical regions is prescribed in *Section 5*, depending on the type of primary seismic element and on the Ductility Class, as are the special detailing and other rules that apply within that length. A critical region is considered at the end of a primary seismic column or beam, irrespective of whether plastic hinging is expected to take place there, or alterntively in the beams or columns connected to the joint at that particular end of the primary element.

## 5.3. Types of structural systems for earthquake resistance of concrete buildings

*Section 5* identifies the following types of structural systems for concrete buildings, depending on how the system responds to the horizontal components of the seismic action: *Clauses 5.1.2, 5.2.2.1*

- 'Inverted pendulum' systems
- 'Torsionally flexible' systems
- 'Frame' systems
- 'Wall' systems (of coupled or uncoupled walls)
- 'Dual systems' of frames and walls
- 'Systems of large lightly reinforced walls'.

The seismic response and performance of the first two types of systems has certain undesirable features. Consequently, these two types of systems are singled out to be penalized with low values of the behaviour factor $q$. The low $q$ factors aim at protecting better these two

inherently more vulnerable systems by keeping their response closer to the elastic range and at the same time serving as a disincentive (or warning) against the use of such systems.

The systems of large lightly reinforced walls are differentiated from those of uncoupled ductile walls not in the value of the $q$ factor, which is the same, but in the dimensioning and detailing rules, which are fundamentally different.

There is no differentiation among the remaining three types of systems (frame, dual and coupled-wall systems), either in the value of the behaviour factor $q$ (which is the same and the highest among all the types) or in their design rules: the dimensioning and detailing of a beam, column or ductile wall, coupled or not, is the same, regardless of whether the member is part of a frame, dual or coupled-wall system. As far as design is concerned, a very important distinction is between (1) frame or frame-equivalent dual systems on the one hand and (2) wall or wall-equivalent dual systems on the other. The columns of the former should (in general) fulfil the strong-column–weak-beam rule, in order to prevent formation of a soft storey and promote beam-sway mechanisms; in the latter, this ultimate target is meant to be achieved merely by the presence of ductile walls, sufficient in number and dimensions to force the entire structural system to stay straight while swaying. For similar reasons (i.e. owing to their walls), structural systems listed in point 2 are not considered to be affected by any masonry infills during the seismic response, and therefore are not subject to the special design and detailing rules that the systems listed in point 1 have to follow, in the presence of such infills.

The main features of the different types of structural systems recognized in *Section 5* for concrete buildings are discussed in overview below, along with their implications for the design.

### 5.3.1. Inverted-pendulum systems

An inverted pendulum is defined as a system with at least 50% of the total mass in the upper third of the height, or with energy dissipation at the base of a single element. Literally, one-storey concrete buildings normally fall in that category. Nonetheless, one-storey frames with the tops of columns connected (through beams) in the two main directions of the building in plan are explicitly excluded from the category, provided that in the seismic design situation the maximum value of the normalized axial load $\nu_d$ in any column does not exceed 0.3. Such a low value of the axial load, which corresponds to 0.2 for the usual value of 1.5 for the partial factor $\gamma_c$ of concrete, enhances the local ductility at the base of the column. Two-storey frames will not be classified as inverted-pendulum systems, if they have the same mass at the two floors, but will be classified as such if the mass lumped at the roof noticeably exceeds that of the first floor.

### 5.3.2. Torsionally flexible systems

A system is defined as torsionally flexible if at any floor one or both of the conditions of equations (D4.2) are not met (i.e. if the radius of gyration of the floor mass exceeds the torsional radius in one or both of the two main directions of the building in plan).

### 5.3.3. Frame systems

*Section 5* defines a frame system as one in which, according to the results of the analysis, 65% of the seismic base shear is (or rather should be) resisted by frames of primary seismic beams and columns. A key feature of frames is that they develop earthquake resistance mainly through normal action effects: **bending moments** with opposite sign develop at column ends, to give the column shears that resist the storey shear demand; the global overturning moment is resisted by **axial forces** (mainly) in the columns of the perimeter. As frame members normally have a shear span ratio (ratio of moment-to-shear divided by member depth) not less than 2.5, their resistance and ultimate deformation capacity are governed by flexure, and hence they are very ductile. Moreover, by dimensioning their columns in flexure to meet the strong-column–weak-beam rule and all members against pre-emptive shear

failure, and by detailing plastic hinge ('critical') regions for ductility, frame systems can be reliably designed for a controlled and very ductile inelastic response.

### 5.3.4. Wall systems

According to *Section 5*, a system in which, according to the results of the analysis, 65% of the seismic base shear is (or rather should be) resisted by primary seismic walls is termed a wall system. Wall systems resist the overturning moment directly, through bending moments rather than through axial forces in the individual walls. Provided that they comprise walls fixed at the base and with sufficient stiffness and strength relative to the beams to behave as vertical cantilevers, wall systems resist horizontal seismic actions very efficiently: for the same total quantity of concrete and horizontal steel (determining the resistance to base shear), lateral stiffness (which is important for drift control) increases and the total required vertical reinforcement decreases, with increasing horizontal dimension $l_w$ of the walls of the system. The limiting value of $l_w$ is the one that gives a shear span ratio, $L_s/l_w$, not less than 2.5, ensuring flexure-controlled behaviour and enhancing wall ductility.

If more than 50% of the total wall resistance is provided by coupled walls, the system is considered to be a coupled-wall system. As coupled walls dissipate energy not only in plastic hinges at the base of the individual walls but also in the coupling beams, overall they have significantly larger dissipation capacity than uncoupled walls with the same shear force capacity at the base. So, unlike the systems of uncoupled walls, coupled-wall systems are entitled to the same basic values of $q$ as the inherently ductile frame systems.

### 5.3.5. Dual systems

A dual system is one in which, according to the results of the analysis, between 35 and 65% of the seismic base shear is (or rather should be) resisted by frames of primary seismic beams and columns, and the rest of the seismic base shear resisted by primary seismic walls. Dual systems combine the satisfactory stiffness, force resistance and cost-effectiveness of walls with the ductility and large deformation capacity of frames, which can act as a second line of defence in case (some of) the more brittle walls of the system fail. Moreover, dual systems use to advantage the beams and columns that carry (most of the) gravity loads for the lateral force resistance, as well as the capacity of columns to resist lateral forces in both horizontal directions. Their inelastic behaviour, though, is much more uncertain than that of pure frame or wall systems. Examples of uncertainties include:

(1) the capacity of floor diaphragms to transfer forces from walls to frames or vice versa, as these subsystems share the storey shear differently at different storeys
(2) the sharing of lateral forces between walls and frames depending on the rotation at the base of walls and columns due to compliance of the foundation (in systems with vertical elements of about the same size, such rotations do not appreciably affect the distribution of storey shears forces among the vertical elements).

The sensitivity of the response to such uncertainties should be reduced through proper conceptual design and/or addressed through sensitivity analyses.

If more than 50% of the base shear is resisted by primary seismic walls, the dual system is classified as wall-equivalent; otherwise it is defined as frame-equivalent. As noted on p. 90, the distinction between wall- and frame-equivalent dual systems has important practical consequences, as it determines whether the columns of the dual system should be capacity designed against plastic hinging above their base and whether design should account for the presence and the effects of masonry infills.

### 5.3.6. Systems of large lightly reinforced walls

Eurocode 8 is unique among international codes in that it includes special provisions for systems consisting of a fairly large number of large but lightly reinforced concrete walls

which are designed to sustain seismic demands not by dissipating kinetic energy through hysteresis in plastic hinges but by converting part of this energy into potential energy of the masses and returning part to the ground through radiation from their foundation. To qualify for the special design provisions of Eurocode 8 such a system should have in each horizontal direction at least two walls (for redundancy and torsional resistance) that qualify as 'large lightly reinforced walls' in the sense of Section 5.2.4, resist together at least 65% of the seismic base shear in the horizontal direction of interest (for the system to qualify as a wall system) and support together at least 20% of the total gravity load (i.e. at least 40% in total for the walls of the two directions). The building should also have a fundamental period in each horizontal direction for assumed fixity of all vertical elements at the base against rotation of not longer than 0.5 s. This last condition promotes walls with a low aspect ratio and/or a large total cross-sectional area as a percentage of the total plan area of the floors, and takes into account better the effect of openings in the wall than a mere geometrical criterion would have done. The condition of at least 20% of the total gravity load carried by the walls of each horizontal direction ensures that the rocking motion of these walls increases the potential energy of at least that part of the total mass of the building. The condition of at least two large walls per horizontal direction may be relaxed, provided that (1) the two other conditions – for at least of 20% of total gravity load and for a period not more than 0.5 s – can be met with a single large wall in that direction, (2) there are at least two large walls in the orthogonal direction and (3) the $q$ factor in the direction with just one large wall is reduced by one-third.

If the structural system meets all the conditions above, *Section 5* permits all the walls that qualify as large to be designed and detailed in a very economic way according to the special rules for large lightly reinforced walls outlined in Section 5.8 below. The system of large lightly reinforced walls is considered to qualify for a basic $q$ factor equal to that for wall systems with uncoupled ductile walls designed and detailed according to the much more demanding rules for ductile walls of Ductility Class Medium (DCM). Walls with length $l_w$ of less than 4 m (or two-thirds of the total height in buildings less than 6 m tall) in a system of large lightly reinforced walls should be designed and detailed according to the rules for ductile walls of DCM. These latter rules should also be followed by any wall with length $l_w$ over 4 m (or two-thirds of the total height in buildings less than 6 m tall), if in the direction of $l_w$ the system does not qualify as a system of large lightly reinforced walls.

## 5.4. Design concepts: design for strength or for ductility and energy dissipation – ductility classes

*Clause 5.2.1*

As already mentioned in Section 2.2.2, Eurocode 8 gives the option to design concrete buildings for more strength and less ductility, or vice versa. This option is exercised through the ductility classification of concrete buildings: Eurocode 8 permits trading ductility and dissipation capacity for strength by providing for three alternative ductility classes: low (DCL), medium (DCM) and high (DCH).

Buildings of DCM or DCH have $q$ factors higher than the value of 1.5 considered to be available owing to overstrength alone. DCH buildings are allowed to have higher values of $q$ than DCM ones. They also have to meet more stringent detailing requirements for members and to provide higher safety margins in capacity design calculations aiming at ensuring ductile global behaviour. The two upper ductility classes represent two different possible combinations of strength and ductility, approximately equivalent in terms of total material cost and achieved performance under the design seismic action. DCM is slightly easier to design for and achieve at the construction site, and may provide better performance in moderate earthquakes. DCH is believed to provide higher safety margins against local or global collapse under earthquakes (much) stronger than the design seismic action.

*Section 5* itself does not link selection between the two higher ductility classes to seismicity of the site or importance of the structure, nor puts any limit to their application. It is up to a

CEN member state to make a choice for the various parts of its territory, or – preferably – to leave the choice to the designer, depending on the particular design project.

*Clauses 5.2.1(2), 5.3.1, 5.3.3*

Buildings of DCL are designed not for dissipation capacity and ductility but only for strength: they have to follow, in practice, only the dimensioning and detailing rules of Eurocode 2, and are designed to accommodate earthquakes in exactly the same way as for other lateral actions, such as wind. Although design to Eurocode 2 alone implies that the structure essentially remains elastic under its design actions, the members of DCL concrete buildings are dimensioned for internal forces derived by dividing the elastic response spectrum by a $q$ factor of 1.5 instead of 1.0. This value of $q$ is considered not to be due to any presumed energy dissipation capacity of the so-designed buildings, but only to overstrength of its members with respect to the seismic internal forces they are dimensioned for. This overstrength is a result of:

- the systematic difference between the expected strength of steel and concrete *in situ* from the corresponding design values (mean strength is considered to exceed the nominal value by 8 MPa for concrete or by about 15% for reinforcing steel – on top of that difference, nominal strengths are divided by the partial factors for materials to arrive at the design values)
- rounding-up of the number and the diameter of rebars
- placement of the same reinforcing bars at the two cross-sections of a beam or column across a joint, determined by the maximum required steel area at these two sections
- the frequent control of the amount of reinforcement by non-seismic actions and/or minimum reinforcement requirements, etc.

In moderate-to-high-seismicity regions, DCL buildings may not be cost-effective. Moreover, as they do not possess engineered ductility and energy dissipation capacity, they may not have a reliable safety margin against an earthquake significantly stronger than their design seismic action. So, they are not considered appropriate for regions of moderate or high seismicity. Eurocode 8 recommends the use of DCL only in cases of low seismicity, but it will be up to a CEN member state to decide whether it will follow this recommendation or not. It should be recalled that the definition of what constitutes a low-seismicity case is also left to member states, with Eurocode 8 recommending a ceiling for low-seismicity cases of $0.08g$ for the design ground acceleration on rock, $a_g$, or of $0.1g$ for the design ground acceleration on the type of ground of the site, $a_g S$, with $a_g$ including the importance factor $\gamma_I$.

## 5.5. Behaviour factor $q$ of concrete buildings designed for energy dissipation

*Clause 5.2.2.2*

In building structures designed for energy dissipation and ductility, the value of the behaviour factor $q$, by which the elastic spectrum used in linear analysis is reduced, depends on the type of lateral-force-resisting system and on the ductility class selected for the design. As we will see in Section 5.6.3.2 the value of the $q$ factor is linked, directly or indirectly, to the local ductility demands in members and hence to the corresponding detailing requirements.

As in DCL buildings, overstrength of materials and elements is presumed to correspond to a $q$ factor of 1.5, already built into the $q$ factor values given for buildings of DCM or DCH. In addition, overstrength of the structural system due to redundancy is explicitly included in the $q$ factor, through the ratio $\alpha_u/\alpha_1$. This is the ratio of the seismic action that causes development of a full plastic mechanism to the seismic action at the formation of the first plastic hinge in the system – both in the presence of the gravity loads considered to act simultaneously with the seismic action. If $\alpha_1$ is considered as a multiplicative factor on seismic action effects from the elastic analysis for the design seismic action, the value of $\alpha_1$ may be computed as the lower value over all member ends in the structure of the ratio $(M_{Rd} - M_V)/M_E$, where $M_{Rd}$ is the design value of the moment capacity at the member end and $M_E$ and $M_V$ are the bending moments there from the elastic analysis for the design

seismic action and for the gravity loads included in the load combination of the seismic design situation. The value of $\alpha_u$ may be found as the ratio of the base shear on development of a full plastic mechanism according to a pushover analysis to the base shear due to the design seismic action (Fig. 5.2). Gravity loads considered to act simultaneously with the seismic action should be maintained constant in the pushover analysis, while lateral forces increase. For consistency with the calculation of $\alpha_1$, the moment capacities at member ends in the pushover analysis should be the design values, $M_{Rd}$. If the mean values of moment capacities are used instead, as customary in pushover analysis, the same values should also be used for the calculation of $\alpha_1$.

In most cases the designer will not consider it worthwhile performing iterations of pushover analyses and design based on elastic analysis, just for the sake of computing the ratio $\alpha_u/\alpha_1$ that may enter into the determination of the $q$ factor. For this reason, *Section 5* gives default values of this ratio. For buildings regular in plan, the default values are:

- $\alpha_u/\alpha_1 = 1.0$ for wall systems with just two uncoupled walls per horizontal direction
- $\alpha_u/\alpha_1 = 1.1$ for (1) one-storey frame or frame-equivalent dual systems and (2) for wall systems with more than two uncoupled walls per direction
- $\alpha_u/\alpha_1 = 1.2$ for (1) one-bay multi-storey frame or frame-equivalent dual systems, (2) wall-equivalent dual systems and (3) coupled-wall systems
- $\alpha_u/\alpha_1 = 1.3$ for multi-storey multi-bay frame or frame-equivalent dual systems.

In buildings which are not regular in plan, the default value of $\alpha_u/\alpha_1$ is the average of (1) 1.0 and (2) the default values given above for buildings regular in plan.

Values higher than the default ones may be used for $\alpha_u/\alpha_1$ up to a maximum of 1.5, provided that the higher value is confirmed through a pushover analysis, after design with the resulting $q$ factor.

For concrete buildings regular in elevation, *Section 5* specifies the values of the $q$ factor given in Table 5.1.

Inverted-pendulum systems are assigned very low $q$ factors: the value for DCM does not exceed that considered available due to overstrength alone without any design for ductility. The low $q$ factor values are due to concerns for potentially large $P$–$\Delta$ effects or overturning moments and reduced redundancy. In view of the $q$ factors of 3.5 for bridges with concrete (single-)piers and more than 50% of the mass at the level of the deck, inverted-pendulum buildings may seem unduly penalized. For this reason, *Section 5* allows the value of $q_o$ of inverted-pendulum systems to be increased, provided that it is shown that a correspondingly higher energy dissipation is ensured in the critical regions.

The values of $q$ in Table 5.1 are called basic values, $q_o$, of the $q$ factor. They are the ones to be used for the estimation of the curvature ductility demands and for the detailing of the 'critical regions' of elements (see equations (D5.11) in Section 5.6.3.2). For the purposes of calculation of seismic action effects from linear analysis, the value of $q$ may be reduced with respect to $q_o$ as follows:

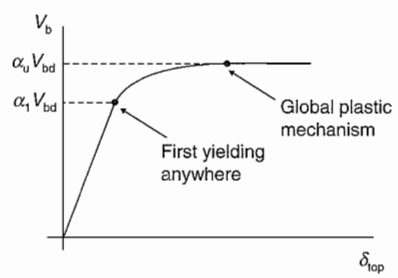



**Fig. 5.2.** Definition of factors $\alpha_u$ and $\alpha_1$ on the basis of base shear versus top displacement diagram from pushover analysis ($V_b$ is the base shear and $V_{bd}$ is the design base shear)

**Table 5.1.** Basic value, $q_o$, of the behaviour factor for regular-in-elevation concrete buildings

| Lateral-load-resisting structural system | DCM | DCH |
|---|---|---|
| Inverted-pendulum system | 1.5 | 2 |
| Torsionally flexible structural system | 2 | 3 |
| Uncoupled-wall system, not belonging in one of the two categories above | 3 | $4\alpha_u/\alpha_1$ |
| Any structural system other than the above | $3\alpha_u/\alpha_1$ | $4.5\alpha_u/\alpha_1$ |

- In buildings which are irregular in elevation, the $q$ factor value is reduced by 20%.
- In wall, wall-equivalent dual or torsionally flexible systems, the value of $q$ is the basic value $q_0$ (reduced by 20% in the presence of irregularity in elevation) multiplied by a factor which assumes values between 0.5 and 1 and is otherwise equal to $(1 + \alpha_0)/3$, where $\alpha_0$ is the (mean) aspect ratio of the walls in the system (sum of wall heights, $h_{wi}$, divided by the sum of wall cross-sectional lengths, $l_{wi}$). This factor reflects the adverse effect of a low shear span ratio on the ductility of walls. It is equal to 1 if $\alpha_0$ is at least equal to 2, and equal to 0.5 when $\alpha_0$ is less than 0.5. Given that in walls with such a low aspect ratio the shear span (moment-to-shear ratio at the base) is about equal to two-thirds of the wall height $h_w$, the $(1 + \alpha_0)/3$ factor is less than 1.0 when the mean shear span ratio of the walls in the system is less than 1.33; these are really squat walls with not so ductile behaviour.

Regardless of the above reductions of $q$, DCM and DCH buildings are permitted a final $q$ factor value of at least 1.5, which is considered to be always available owing to overstrength alone.

Systems of large lightly reinforced walls can only belong to DCM. Therefore, the basic value of their $q$ factor is 3 (or 2, if there is only one large wall in the horizontal direction of interest) to be multiplied by $(1 + \alpha_0)/3$ if the mean aspect ratio of their walls, $\alpha_0$, is less than 2. Normally, such systems are not irregular in elevation, so their $q$ factor is not reduced any further.

A building which is not characterized as an inverted-pendulum system or as torsionally flexible may have different $q$ factors in the two main horizontal directions, depending on the structural system and its vertical regularity classification in these two directions, but not due to the ductility class, which should be chosen to be the same for the whole building.

## 5.6. Design strategy for energy dissipation

### 5.6.1. Global and local ductility through capacity design and member detailing: overview

*Clause 5.2.3*

As already noted in Section 4.11.2.2, to achieve a value of the global displacement ductility factor, $\mu_\delta$, that corresponds according to equations (D2.1) and (D2.2) to the value of the $q$ factor used in the design of multi-storey buildings, a stiff and strong vertical spine should be provided up the height of the building, to spread the inelastic deformation demands throughout the structural system. As shown in Figs 4.4b and 4.4d, in concrete buildings this is accomplished either by using a wall system (or a wall-equivalent dual system), or by designing the columns of frames (and of frame-equivalent dual systems) to be stronger than their beams, so that they do not hinge except at the base of the building.

Wall systems (or wall-equivalent dual systems) are indirectly promoted not only through the strict interstorey drift limits for the damage limitation seismic action (see Section 4.11.2.1), which are difficult to meet with concrete frames alone, but also through their $q$ factors. The $q$ factors of dual and coupled-wall systems are the same as in frames, while those of uncoupled-wall systems are only 10–20% lower.

In frame systems (and frame-equivalent dual systems), strong columns are promoted, indirectly through the interstorey drift limits of Section 4.11.2.1, and directly through the

capacity design of columns in flexure in accordance with Section 4.11.2.3 and equation (D4.23), so that formation of plastic hinges in columns before beam hinging is prevented.

Further to the control of the global inelastic response mechanism through selection of the structural configuration and dimensioning of vertical members to remain elastic above the base, the design strategy aims at ensuring that those individual members where the demand for global ductility and energy dissipation is spread possess the necessary local capacity to sustain this demand. As concrete members can dissipate energy and develop significant cyclic ductility only in flexure – and this only if certain conditions on material ductility and detailing are met – failure of members in shear before they yield in flexure should be precluded. To this end, prevention of pre-emptive shear failure is pursued by establishing the shear force demands on primary seismic beams, columns and walls in DCM and DCH buildings and beam–column joints in DCH frames not from the analysis for the seismic design situation but through capacity design calculations, as outlined in Section 5.6.4. In addition, the aforementioned conditions for the development of flexural ductility should be met, at least in those element zones where it is expected that inelastic deformations will be concentrated and energy dissipation will take place (plastic hinges – 'critical regions'). Section 5.6.3 outlines the conditions imposed by *Section 5* on the ductility of materials used in plastic hinge zones and on the curvature ductility required from these zones; it also presents the rationale and background of these ductility conditions.

## 5.6.2. Implementation of capacity design of concrete frames against plastic hinging in columns

### *5.6.2.1. The left-hand side of equation (D4.23)*

*Clause 5.2.3.3(2)* The design value of the flexural capacity of a beam in negative (hogging) bending may be computed as

$$M^{-}_{\mathrm{Rd,b}} = A_{\mathrm{s2}} f_{\mathrm{yd}} (d - d_2) + (A_{\mathrm{s1}} - A_{\mathrm{s2}}) f_{\mathrm{yd}} [d - 0.5(A_{\mathrm{s1}} - A_{\mathrm{s2}}) f_{\mathrm{yd}} / b f_{\mathrm{cd}}] \qquad \text{(D5.1)}$$

where $A_{\mathrm{s1}}$ and $A_{\mathrm{s2}}$ ($A_{\mathrm{s1}} \geq A_{\mathrm{s2}}$) are the cross-sectional areas of the top and bottom reinforcement, respectively, $b$ is the width of the web, $d$ is the effective depth of the section, $d_2$ is the distance of the centre of $A_{\mathrm{s2}}$ from the bottom of the section, and $f_{\mathrm{cd}}$ and $f_{\mathrm{yd}}$ are the design strengths of steel and concrete, respectively. In the very uncommon case where $A_{\mathrm{s1}} < A_{\mathrm{s2}}$, the second term on the right-hand side is omitted, and $A_{\mathrm{s1}}$ is used instead of $A_{\mathrm{s2}}$ in the first term.

The design value of the beam flexural capacity in positive (sagging) bending may be computed as

$$M^{+}_{\mathrm{Rd,b}} = A_{\mathrm{s2}} f_{\mathrm{yd}} \max[(d - 0.5 A_{\mathrm{s2}} f_{\mathrm{yd}} / b_{\mathrm{eff}} f_{\mathrm{cd}}); (d - d_1)] \qquad \text{(D5.2)}$$

where $d_1$ is the distance of the centre of $A_{\mathrm{s1}}$ from the top of the section and $b_{\mathrm{eff}}$ is the effective width of the slab in compression.

The factor 1.3 in equation (D4.23) is meant to cover overstrength of beams, mainly due to strain hardening of steel. This value covers more than sufficiently this type of overstrength, as the reinforcing steels currently used in Europe (including its most seismic regions) are mainly of the Tempcore type, and do not exhibit large strain hardening; moreover, the overstrength of the column due to confinement of concrete is not taken into account on the left-hand side of equation (D4.23). Nonetheless, the value of 1.3 may not always be sufficient to also fully cover two other adverse effects: (1) the increased flexural capacity of the beam in negative (hogging) bending due to slab reinforcement which is parallel to the beam and is anchored in the slab within the extent of the joint or beyond (see next paragraph); and (2) plastic hinging of columns of two-way frames due to biaxiality of the bending moments.

There is ample experimental and practical evidence that, when the beam is driven past flexural yielding in negative bending and into strain hardening, such slab reinforcement up to a significant distance from the web of the beam is fully activated and contributes to the beam negative flexural capacity as tension reinforcement. *Section 5* (*clause 5.4.3.1.1(3)*) specifies the effective in tension width of the slab on each side of the column into which the beam frames as four times the slab thickness, $h_{\mathrm{f}}$, at interior columns if a transverse beam of

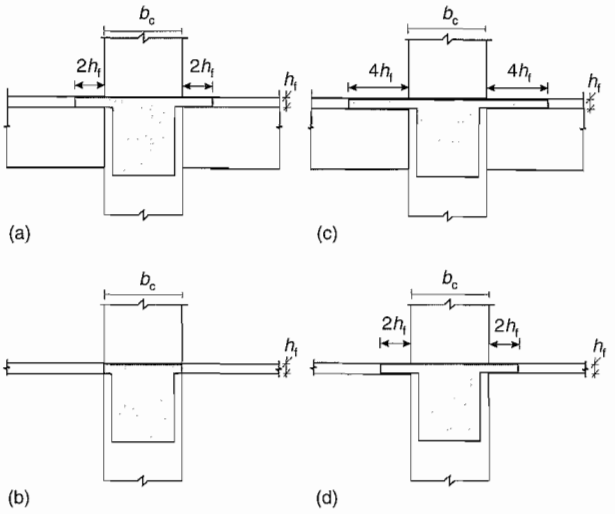



**Fig. 5.3.** Slab width effective as the tension flange of a beam at the support to a column, according to *Section 5*: (a, b) at the exterior column; (c, d) at the interior column

similar size frames into the joint on the side in question, or just $2h_f$ if there is no such transverse beam. At the two exterior columns within the plane of the frame where equation (D4.23) is checked, the above effective in-tension slab widths on each side of the web are reduced by $2h_f$. These slab widths, shown in Fig. 5.3, are specified in *Section 5* for the dimensioning of beams at the supports to columns against the negative (hogging) bending moment from the analysis for the seismic design situation: any slab bars which are parallel to the beam and are well anchored within the extent of the joint or beyond may count as top beam reinforcement, and reduce the amount of tension reinforcement that needs to be placed within the width of the web. In that context, the value of the effective in-tension width of the slab on each side of the web has been chosen to be lower than the values of about one-quarter of the beam span suggested by practical and experimental evidence, so that it is conservative (safe-sided) for the dimensioning of beam top bars. However, it leads to underestimation of $M_{Rd,b}$ for negative bending, and hence it is on the unconservative (unsafe) side regarding prevention of column hinging through fulfilment of equation (D4.23).

### *5.6.2.2. The right-hand side of equation (D4.23)*

The flexural capacity of a column depends on its cross-sectional shape and the arrangement of the reinforcement in it. The most common case is that of a rectangular section, with depth $h$ (parallel to the plane within which equation (D4.23) is checked), width $b$, tension and compression reinforcement with cross-sectional area $A_{s1}$ and $A_{s2}$, each concentrated at a distance $d_1$ from the nearest extreme fibres of the section in the direction of $h$, and additional reinforcement with cross-sectional area $A_{sv}$ approximately uniformly distributed along the length $(h - 2d_1)$ of the depth $h$ between the tension and the compression reinforcement. Most often the cross-section is symmetrically reinforced: $A_{s1} = A_{s2}$. However, the more general case of unsymmetric reinforcement is considered here, as it may apply also to cross-sections consisting of more than one rectangular part in two orthogonal directions, as in L-, T- or U-shaped sections. For such a section, it is most convenient to compute $M_{Rd,c}$ with respect to centroidal axes parallel to these two orthogonal directions, irrespective of the fact that they may not be principal directions. Normally – and very conveniently – the beams connected to such columns are parallel to the sides of the rectangular parts of their section, defining the framing planes within which equation (D4.23) is checked. The procedure given below for the calculation of $M_{Rd,c}$ may be applied to such sections, provided that the width of the compression zone is constant between the neutral axis and the extreme compression

fibres (i.e. the depth $x$ of the compression zone is within a single one of the rectangular parts of the section). Then, the section may be considered for the present purposes as rectangular, with constant width $b$, equal to that at the extreme compression fibres.

According to Eurocode 2, the design value of the flexural capacity of a cross-section, $M_{Rd}$, is considered to be attained when the extreme compression fibres reach the ultimate strain of concrete, $\varepsilon_{cu}$. The value of $\varepsilon_{cu}$ for use in conjunction with the parabolic–rectangular $\sigma$–$\varepsilon$ diagram of concrete of clause 3.1.7(1) of EN 1992-1-1 is denoted there as $\varepsilon_{cu2}$, and for the concrete classes common in European earthquake-resistant construction (i.e. up to C50/60) is given in Table 3.1 of EN 1992-1-1 as $\varepsilon_{cu2} = 0.0035$. The concrete strain at ultimate strength, $f_c$, i.e. at the peak of the parabolic part of the diagram, is denoted by $\varepsilon_c$, and its value for use in the calculation of the flexural capacity, $\varepsilon_{c2}$, is given in the same table as $\varepsilon_{c2} = 0.002$ (for concrete up to C50/60).

As in primary seismic columns, and especially those which should satisfy equation (D4.23), the axial load in the seismic design situation is relatively low, the tension reinforcement, $A_{s1}$, is expected to have yielded when the strain at the extreme compression fibres reaches the ultimate strain, $\varepsilon_{cu}$. For the grades of reinforcing steel common in Europe, the compression reinforcement, $A_{s2}$, being not far from the extreme compression fibres, will also be beyond its yield strain, $f_y/E_s$, when the strain at the extreme compression fibres reaches $\varepsilon_{cu}$. Under these conditions, the value of the neutral axis depth at ultimate moment, normalized to the effective depth of the section $d = h - d_1$ of the section as $\xi = x/d$, is equal to

$$\xi_{cu} = \frac{(1-\delta_1)(\nu+\omega_1-\omega_2)+(1+\delta_1)\omega_v}{(1-\delta_1)(1-\varepsilon_{c2}/3\varepsilon_{cu2})+2\omega_v} \tag{D5.3}$$

The value from equation (D5.3) (indexed by cu, to show ultimate condition controlled by the ultimate concrete strain, $\varepsilon_{cu}$) can be used as $\xi$ in the following equation for the flexural capacity of the column:

$$M_{Rc} = bd^2 f_c \left\{ \frac{(1-\delta_1)(\omega_1+\omega_2)}{2} + \frac{\omega_v}{1-\delta_1}\left[ (\xi-\delta_1)(1-\xi) - \frac{1}{3}\left( \frac{\xi f_y}{E_s \varepsilon_{cu}} \right)^2 \right] + \xi\left[ \frac{1-\xi}{2} - \frac{\varepsilon_c}{3\varepsilon_{cu}}\left( \frac{1}{2} - \xi + \frac{\varepsilon_c}{4\varepsilon_{cu}}\xi \right) \right] \right\} \tag{D5.4}$$

The variables in equations (D5.3) and (D5.4) are $\omega_1 = A_{s1}f_y/bdf_c$, $\omega_2 = A_{s2}f_y/bdf_c$, $\omega_v = A_{sv}f_y/bdf_c$, $\nu = N/bdf_c$ and $\delta_1 = d_1/d$. If the design values $f_{yd}$ and $f_{cd}$ are used for $f_y$ and $f_c$, and the conventional values $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$ for $\varepsilon_c$ and $\varepsilon_{cu}$, respectively, then equation (D5.4) gives the design value, $M_{Rd,c}$, of the flexural capacity.

For equation (D5.4) to be applicable for a cross-section consisting of more than one rectangular part in two orthogonal directions, with the width $b$ taken as that of the section at the extreme compression fibres, the depth $x = \xi d$ of the compression zone calculated with the value of $\xi$ from equation (D5.3) should not exceed the other dimension (depth) of the rectangular part to which $b$ belongs.

The column axial force, $N$, to be considered in the calculation of $M_{Rd,c}$ should be derived from the analysis for the seismic design situation and assume the most adverse value for the fulfilment of equation (D4.23) – i.e. minimum compression or maximum net tension – that is physically consistent with $M_{Rd,c}$. The way to determine this value depends on the method of analysis (lateral force or modal response spectrum analysis) and on how the effects of the components of the seismic action are combined (cf. Section 4.9).

*5.6.2.3. Exemptions from the capacity design rule for plastic hinging in columns (equation (D4.23))*

It is extremely unlikely that both the top and bottom ends of a concrete wall within a storey will yield in opposite bending and develop plastic hinging, even when the wall section barely has the minimum dimensions required by Eurocode 8 (e.g. for a rectangular wall, just over 0.2 m × 0.8 m). So, in the horizontal direction of the building that has walls resisting at least

50% of the seismic base shear (wall and wall-equivalent dual systems), Eurocode 8 expects these walls to prevent the occurrence of a soft-storey mechanism, and waives the condition of satisfaction of equation (D4.23) at the joints of primary seismic columns with beams. In frame and frame-equivalent dual systems, fulfilment of equation (D4.23) is also waived:

- at the joints of the top floor, as allowed for all frame structures according to Section 4.11.2.3
- at the joints of the ground storey in two-storey buildings, provided that in none of its columns the axial load ratio $\nu_d$ exceeds 0.3 in the seismic design situation (columns with such a low axial load ratio have good ductility and develop low $P$–$\Delta$ effects; so they can survive a displacement ductility demand equal to twice the displacement ductility factor, $\mu_\delta$, that corresponds to the value of $q$ used in design, when a soft-storey mechanism develops at the ground storey)
- in one out of four columns of plane frames with columns of similar size and hence of similar importance for the earthquake resistance (it may be chosen not to fulfil equation (D4.23) at interior columns rather than at exterior ones, as only one beam frames into exterior joints and it is easier to satisfy equation (D4.23) there).

At all column ends where equation (D4.23) is not checked by virtue of the exemptions above (including the columns of wall or wall-equivalent dual systems ), the rules of *Section 5* for buildings of DCH (but not for those of DCM) aim at a column ductility which is sufficient for development of a plastic hinge there. In fact, these rules provide the same degree of ductility as at the base of these columns, assuming that the global ductility demand is uniformly spread in all storeys.

#### *5.6.2.4. Dimensioning procedure for columns to satisfy equation (D4.23)*

Verification of equation (D4.23) at a beam–column joint pre-supposes that the longitudinal reinforcement at the end sections of the beams framing into the joint has already been dimensioned for the ultimate limit state (ULS) in bending on the basis of the analysis results for the seismic design situation and fully detailed to meet the minimum and maximum reinforcement requirements for the particular ductility class. It should be recalled that the seismic design situation is an abbreviation for the combination of (1) permanent loads entering with their nominal value, $G_k$, and imposed ('live') loads entering with their quasi-permanent (arbitrary point in time) value according to Section 4.4.1 and (2) the design seismic action, which includes separate consideration of each horizontal component with its own accidental eccentricity and combination of the two components (with the most adverse effect of their accidental eccentricity included) through either the square root of the sum of the squares rule of equation (D4.21) (which gives a positive end result), or the 100%–30% rule of equation (D4.22) with the internal action effects from both components normally taken with the same sign.

In principle, equation (D4.23) may well be checked after the vertical reinforcement crossing both column sections right above and below the joint is also dimensioned for the ULS in bending on the basis of the analysis results for the seismic design situation and detailed to meet the relevant detailing provisions for the particular ductility class. However, as fulfilment of equation (D4.23) is normally more demanding than the ULS in bending on the basis of the results of the analysis for the seismic design situation, it makes sense to defer dimensioning of the column vertical reinforcement until the stage at which equation (D4.23) is checked. At that stage, about half of the value of the left-hand side of equation (D4.23) may be assigned to the column section right above the joint, and the rest to the column section right below the joint. Then, the vertical reinforcement which is common in both of these sections may be dimensioned for these two uniaxial bending moments, considered to act together with the corresponding minimum value of the column axial force in the seismic design situation (determined as suggested in the last paragraph of Section 5.6.2.2, p. 98). Since, for a given vertical reinforcement, the flexural capacity increases with the (compressive) axial force, it makes sense to assign a little less than half of the left-hand side of equation

(D4.23) to the column section right above the joint, the most cost-effective apportioning being that which gives the same amount of vertical reinforcement in these two sections (a 45%/55% split is normally appropriate).

*Clause 5.5.3.2.2(14)*

The longitudinal reinforcement at the base section of the bottom storey of a column (where the column is connected to the foundation) is dimensioned for the ULS in bending with axial force under the action effects from the analysis for the seismic design situation, without any capacity design considerations. Specifically for columns of DCH, where the seismic action effects are computed on the basis of a fairly high $q$ factor value and may then have relatively low values, *Section 5* requires that the longitudinal reinforcement placed at the base of the bottom storey is not less than that provided at the top of the storey. The objective of this requirement is to make sure that after the plastic hinge develops at the base of that column, the moment at the top does not increase to become (much) larger than at the bottom. Such an increase may unduly reduce the value of the shear span at the plastic hinge, $L_s = M/V$, in comparison with its value at yielding at the base, reducing also the plastic rotation capacity of the very crucial hinge at the base of the column. In terms of equations (D5.5) and (D5.8), the value of $L_s$ in equation (D5.5) would be the initial one at yielding at the base – normally more than half the clear height of the column – while that in equation (D5.8), which determines the plastic rotation capacity, would be the subsequent smaller one.

*Clause 5.4.3.2.1(2)*

According to *Section 5*, the ULS verification of columns under the various combinations of biaxial bending moments and axial force resulting from the analysis for the seismic design situation may be performed in a simplified – and safe-sided – way, neglecting one component of the biaxial bending moment at a time, provided that the other component is less than 70% of the corresponding uniaxial flexural resistance under the axial force of the combination. As one of the two components of the biaxial bending moment is normally much larger than the other in the combination, the simplified verification – devised to also cover the case of biaxial bending with about equal components – is quite conservative for the column vertical reinforcement. Where applied, it results in a sum of column flexural capacities above and below the joint, $\sum M_{Rd,c}$, that exceeds the – maximum over all combinations included in the seismic design situation of the – sum of column moments above and below of the joint from the analysis, $\max \sum M_{E,c}$, multiplied by 1/0.7. As $\max \sum M_{E,c}$ is (about) equal to the corresponding maximum sum of beam moments on opposite sides of the joint, $\max \sum M_{E,b}$, the simplified biaxial ULS verification gives $\sum M_{Rd,c} \geq \max \sum M_{E,b}/0.7 = 1.43 \max \sum M_{E,b}$. Normally a substantial margin over $\max \sum M_{E,b}$ is provided by the value of $\sum M_{Rd,b}$ that results from dimensioning of the beam sections next to the joint for each one of the beam moments $M_{E,b}$ from the analysis for the seismic design situation, rounding up the reinforcement and detailing it to meet the minimum requirements (especially at the bottom of the beam). If that strength margin in the beams is about 10%, the simplified biaxial verification of the column moments gives a value of $\sum M_{Rd,c}$ that automatically satisfies equation (D4.23). The implication is that dimensioning of the vertical reinforcement of the column for about half of the moment on the right-hand side of equation (D4.23) gives about the same end result as the simplified biaxial ULS verification of columns on the basis of the analysis for the seismic design situation (especially if column moments from the analysis are redistributed between the two sections above and below the joint, as permitted by *clauses 4.4.2.2(1)* and *5.4.2.1(1)* of EN 1998-1). If the strength margin in the beams is more than 10% and/or the designer opts for a truly biaxial ULS verification of the column on the basis of the analysis results for the seismic design situation, this latter verification requires even less vertical reinforcement in the column than fulfilment of equation (D4.23), and therefore is redundant.

The conventional wisdom holds that capacity design of columns to satisfy equation (D4.23) complicates the design process. The arguments above lead to the opposite conclusion: straightforward dimensioning of the column vertical reinforcement to meet equation (D4.23) is less tedious than ULS verification of the columns on the basis of the analysis results for the seismic design situation, even when this is done with the simplified biaxial verification. If nothing else, it has to be done once in each horizontal direction (transverse axis of the

column) in which equation (D4.23) has to be satisfied, whereas – due to the need to combine the components of the seismic action according to Section 4.9 and to account for the effects of accidental eccentricity – ULS verification of the columns on the basis of the analysis results for the seismic design situation normally involves four, but possibly 16, different combinations of moments with axial force. Last but not least, if the overstrength of beams relative to the requirements of the analysis for the seismic design situation is not large, then fulfilment of equation (D4.23) – at least with the value of 1.3 for the overstrength factor – does not over-penalize the column vertical reinforcement either. So, except for the top-storey columns, there is no real motivation to use the exemptions from equation (D4.23) allowed by *Section 5* just for the sake of economy or simplification of the design process.

### 5.6.3. Detailing of plastic hinge regions for flexural ductility

#### *5.6.3.1. Material requirements*

*Clauses 5.3.2, 5.4.1.1, 5.5.1.1*

Deformation and ductility capacity depends not only on the detailing of members but on the inherent ductility of their materials as well. Local deformation and ductility demands increase as the ductility class (and with it the value of $q$) increases. As a result, ductility requirements on materials increase with the ductility class.

Because concrete strength positively affects member ductility and energy dissipation capacity in practically every respect (from the increase of bond and shear resistance to the direct enhancement of deformation capacity), *Section 5* sets a lower limit on the nominal cylindrical concrete strength in primary seismic elements, equal to 16 MPa (concrete class C16/20) in buildings of DCM, or 20 MPa (concrete class C20/25) in those of DCH. No upper limit on concrete strength is set, as there is no experimental evidence that the lower apparent ductility of high-strength concrete in compression (due to which the values specified in Table 3.1 of EN 1992-1-1 for $\varepsilon_{c2}$ and $\varepsilon_{cu2}$ converge from $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$ for concrete class C50/60 to a single value of 0.0026 at C90/100) has any adverse effect on member ductility and energy dissipation capacity.

In primary seismic elements of buildings of DCM or even DCL, reinforcing steel should have a hardening ratio, $f_t/f_y$, of at least equal to 1.08 and a strain at maximum stress (often called uniform elongation at failure), $\varepsilon_{su}$, of at least 5% (both values refer to the lower 10% fractiles). These are steels of class B or C according to Eurocode 2, Table C.1. In the critical regions of primary seismic elements of DCH buildings, $\varepsilon_{su}$ should be at least 7.5%, the hardening ratio of tensile to yield strength, $f_t/f_y$, should be between 1.15 and 1.35, and the upper characteristic (95% fractile) of the actual yield stress, $f_{yk,\,0.95}$, should not exceed the nominal yield strength, $f_{yk}$, by more than 25%. The first two conditions are met by steels of class C according to Eurocode 2, Table C.1. The purpose of the lower limit on $\varepsilon_{su}$ is to ensure a minimum curvature ductility and flexural deformation capacity, by preventing bar fracture prior to concrete crushing, or simply delaying it until a target flexural deformation is reached (see equation (D5.7)). The lower limit on $f_t/f_y$ aims at ensuring a minimum length of the flexural plastic hinge, as theoretically the plastic hinge length, $L_{pl}$, is equal to the shear span, $L_s$, multiplied by $(1 - M_y/M_u)$, with the ratio of the yield moment, $M_y$, to the ultimate moment, $M_u$, being approximately equal to $f_y/f_t$. Finally, the purpose of the ceiling on the values of $f_t/f_y$ and $f_{yk,\,0.95}/f_{yk}$ is to limit flexural overstrength, and hence shear force demands on members and joints, as controlled by flexural yielding at the end of members, as well as the moment input from beams to columns (cf. equation (D4.23)).

Strictly speaking, for buildings belonging to DCM the requirement for the use of steel of at least class B applies only to the critical regions of their primary seismic elements. As in DCL buildings critical regions are not defined, the requirement for the use of steel of at least class B applies throughout the length of primary seismic elements. As the local ductility of a DCM or DCH building should not in any respect be inferior to a DCL structure, the whole length of primary seismic elements of DCM and DCH buildings should have reinforcing steel of at least class B. The additional requirements on the steel of the critical regions of DCH buildings essentially apply (1) thoughout the entire height of primary seismic columns, (2) in the critical region at the base of primary seismic walls and (3) in the critical regions near the

supports of primary seismic beams to columns or walls (including the slab bars which are parallel to the beam and fall within the effective tension flange width defined in Fig. 5.3). Obviously, it is not easy to implement material specifications which differ in a certain part of a concrete element (its dissipative zones or critical regions) from the rest of its length. Therefore, in practice the requirements on reinforcing steel of critical regions are expected to be applied over the entire primary seismic element, including the slab it may be working with.

*5.6.3.2. Curvature ductility requirements*

*Clauses 5.2.3.4(1), 5.2.3.4(2)(a), 5.2.3.4(2)(b)*

Of the two constituent materials of concrete members, only reinforcing steel is inherently ductile – and then only when in tension, as bars in compression may buckle, shedding their force resistance and risking immediate or subsequent fracture. Concrete is not ductile, unless its lateral expansion is effectively restrained through confinement.

The only mechanism of force transfer that allows using to advantage and in a reliable way the fundamental ductility of tensile steel and effectively enhancing the ductility of concrete and of the compression steel through lateral restraint is flexure. Even under cyclic loading, flexure creates stresses and strains in a single and well-defined direction, and therefore lends itself to the effective use of the reinforcing bars, both to take up directly the tension as well as to restrain concrete and compression steel exactly transverse to their compression stresses. An inelastic stress field dominated by shear is two-dimensional, induces principal stresses and strains in any inclined direction (especially with load cycling), and does not lend itself to effective inelastic action in the reinforcement, control of the extent of cracking (which, if not effectively restrained, may extend into the compression zone and completely destroy it) and confinement of the concrete. So, unlike steel members, where shear is considered as a ductile force transfer mechanism because the ductility of steel is always available in the rotating direction of principal strains, in concrete, shear is considered brittle and constrained by design in the elastic range of behaviour. Energy dissipation and cyclic ductility is entrusted only to flexure, in the plastic hinges that develop at member ends, where seismic bending moments are at a maximum. The plastic hinge regions are then detailed for the inelastic deformation demands expected to develop there under the design seismic action.

*Clauses 5.2.3.4(2)(a), 5.2.3.4(3)*

*Section 5* aims at linking the local displacement and deformation demands on plastic hinges to the behaviour factor $q$ used in the design. As the introduction of the system overstrength factor $\alpha_u/\alpha_1$ in the value of $q$ produces a spectrum of continuous $q$ values, the link between $q$ and the local displacement and deformation demands has to be algebraic. The link is provided through the global displacement ductility factor, $\mu_\delta$, linked to $q$ through equations (D2.1) and (D2.2). It should be recalled that a (materials and elements) overstrength factor of 1.5 is already built into the $q$ factor values given in Table 5.1 for buildings of DCM or DCH. So, normally, equations (D2.1) and (D2.2) should be applied using on the right-hand side the value $q/1.5$ that corresponds to inelastic action and ductility. If $q$ is used instead, a safety factor of 1.5 is hidden in the resulting values of $\mu_\delta$.

The link between local displacement and deformation demands on plastic hinges and the global displacement ductility factor, $\mu_\delta$, is based on the kinematics of the beam-sway mechanism ensured by the dominance of walls in the structural system or by the fulfilment of equation (D4.23) at practically all beam–column joints. It is obvious from Figs 4.4b and 4.4d that in such a mechanism the demand value of the local ductility factor of the chord rotation at all member ends where a plastic hinge forms, $\mu_\theta$, is approximately equal to the demand value of the global displacement ductility factor, $\mu_\delta$. It should be recalled that the chord rotation $\theta$ at a member end is the deflection of the point of contraflexure with respect to the tangent to the member axis at the end of interest, divided by $L_s$; so it is a measure of member displacement and not of relative rotation between sections. In turn, the demand value of $\mu_\theta$ may be linked to that of the curvature ductility factor of the end section, $\mu_\phi$, as

$$\mu_\theta = 1 + (\mu_\phi - 1)\frac{3L_{pl}}{L_s}\left(1 - \frac{L_{pl}}{2L_s}\right) \qquad \text{(D5.5)}$$

where $L_s$ is the shear span (moment-shear-ratio) at the end of interest and $L_{pl}$ is the plastic hinge length. The latter is a conventional quantity, defined on the basis of the assumptions of (1) purely flexural deformations within the shear span and (2) constant inelastic curvature up to a distance from the end equal to $L_{pl}$. Empirical relations are then fitted to $L_{pl}$ so that equation (D5.5) is fulfilled on average at failure of the structural member in tests. In such an exercise, $\mu_\phi$ is taken as $\phi_u/\phi_y$ and $\mu_\theta$ as $\theta_u/\theta_y$, with the yield curvature $\phi_y$ computed from first principles, $\theta_y$ taken as $\theta_y = \phi_y L_s/3$, and $\phi_u$, $\theta_u$ as the ultimate curvature of the end section and the ultimate chord rotation (drift ratio) of the member. These ultimate deformations are conventionally identified with a drop in peak force during a load cycle below 80% of the ultimate strength (maximum force resistance) of the section or of the member. The ultimate curvature is computed from first principles, while the ultimate chord rotation is – at least for the purposes of fitting an empirical relation to $L_{pl}$ – taken to be equal to the experimental value.

First principles employed for the calculation of $\phi_u$ and $\phi_y$ are (1) the plane-sections hypothesis, (2) equilibrium of forces in the direction of the member axis and (3) the material $\sigma$–$\varepsilon$ laws. Calculation of $\phi_y$ is based on linear-elastic behaviour, while for that of $\phi_u$ an elastic–perfectly plastic $\sigma$–$\varepsilon$ law is considered for steel and the parabolic–rectangular $\sigma$–$\varepsilon$ relation of Eurocode 2 for confined concrete. This latter relation entails enhancement of the ultimate strain of concrete, $\varepsilon_{cu}$, due to the confining pressure, $\sigma_2$, as follows:

$$\varepsilon_{cu2,c} = 0.0035 + 0.1\alpha\omega_w \tag{D5.6}$$

where $\omega_w = \rho_w f_{yw}/f_c$ denotes the mechanical volumetric ratio of confining steel with respect to the confined concrete core, $f_{yw}$ is its yield stress and $\alpha$ is the confinement effectiveness ratio, given for rectangular sections by

$$\alpha = \left(1 - \frac{s_h}{2b_o}\right)\left(1 - \frac{s_h}{2h_o}\right)\left(1 - \frac{\sum b_i^2}{6h_o b_o}\right) \tag{D5.7}$$

In equation (D5.7) $b_o$ and $h_o$ are the dimensions of the confined core to the centreline of the hoop, and $b_i$ is the spacing of the centres of longitudinal bars (indexed by $i$) which are laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section. Failure of the section takes place either when the tension reinforcement reaches its strain at maximum stress, $\varepsilon_{su}$, or when the ultimate strain of concrete, $\varepsilon_{cu}$, is exhausted. Then, $\phi_u$ is

$$\phi_u = \min\left(\frac{\varepsilon_{su}}{d - x_{su}}; \frac{\varepsilon_{cu}}{x_{cu}}\right) \tag{D5.8}$$

in which the compression zone depth, $x$, depends on the mode of failure, and is indexed accordingly. Ultimate deformation normally takes place well after spalling of the concrete cover, and equation (D5.7) is applied with the values of $d$ and $x$ of the confined core of the section. Steel rupture under load cycling is found to take place at a strain, $\varepsilon_{su}$, lower than the mean value of the strain at maximum stress: for steel Classes A or B at the minimum values of 2.5 and 5% given in EN 1992-1-1 (Table C.1) for the 10% fractile of the strain at maximum stress, or at $\varepsilon_{su} = 6\%$ for steel Class C. When $\phi_u$ is computed using these values for $\varepsilon_{su}$, and equation (D5.6) for $\varepsilon_{cu}$, then the following expression for $L_{pl}$ provides the best fit to cyclic test results on member chord rotation at flexure-controlled failure:

$$L_{pl} = 0.1L_s + 0.17h + 0.24\frac{d_{bL} f_y(\text{MPa})}{\sqrt{f_c(\text{MPa})}} \tag{D5.9}$$

where $h$ is the depth of the member and $d_{bL}$ is the (mean) diameter of the tension reinforcement.

For the range of parameters $L_s$, $h$, $d_{bL}$, $f_y$ and $f_c$ common in structural elements of buildings, the range of values of $L_{pl}$ from equation (D5.9) is from $0.35L_s$ to $0.45L_s$ for columns (mean value $0.4L_s$), $0.25L_s$ to $0.35L_s$ for beams (mean value $0.3L_s$) and $0.18L_s$ to $0.24L_s$ for

walls (mean value $0.21L_s$). These values are on the high side, because the Eurocode 2 model underestimates the ultimate strain, $\varepsilon_{cu}$, especially for heavily confined members; so $\mu_\phi = \phi_u/\phi_y$ is also underestimated. To avoid propagating the bias further to $\mu_\theta$ through equation (D5.5), equation (D5.9) overestimates $L_{pl}$ with respect to the values that should be used along with a more realistic estimate of $\varepsilon_{cu}$.

In principle, for the value of $\mu_\theta = \mu_\delta$ that corresponds to the value of $q$ used for the design through equations (D2.1) and (D2.2), the demand value of the curvature ductility factor of the end section, $\mu_\phi$, can be computed for each member from equation (D5.5), using the particular value of $L_{pl}$ from equation (D5.9). However, in *Section 5* it was chosen to give a single relation linking $\mu_\phi$ and $q$, based on the following conservative approximation of equation (D5.5):

$$\mu_\theta = 1 + 0.5(\mu_\phi - 1) \qquad \text{i.e. } \mu_\phi = 2\mu_\theta - 1 \tag{D5.10}$$

This option was chosen not only due to its simplicity but also for continuity with the ENV that preceded EN 1998-1, namely EN 1993-1-3. There, equation (D5.10) was behind the discrete values of $\mu_\phi$ that were given for the three ductility classes with the then discrete values of $q$, the main difference with the approach adopted in EN 1998-1 being that the average of the outcome of equations (D2.1) and of the 'equal-energy' approximation: $\mu_\delta = (q^2 + 1)/2$ was used for $\mu_\delta$, irrespective of the value of the period $T$.

Within the full range of possible values of $q$ for DCM and DCH buildings and the usual ranges of $L_{pl}$ for the three types of concrete members, equation (D5.10) gives a safety factor of about 1.65 for columns, about 1.35 for beams and about 1.1 for ductile walls, with respect to the more realistic values provided by inverting equation (D5.5). These values presume that the full value of $q$ corresponds to inelastic action and ductility. When it is realized that only $q/1.5$ produces inelastic deformation and ductility demands, the average safety factor implicit in the demand value of $\mu_\phi$ is 2.45 in columns, 1.9 in beams and 1.2 in ductile walls. This safety factor is increased further when the value of $\mu_\phi$ is used for the calculation of the confining reinforcement required in the 'critical regions' of columns (see Section 5.7.7) and in the boundary elements of the 'critical region' of ductile walls (see Sections 5.7.7 and 5.7.8), as well as of the compression reinforcement in beam end sections (see Section 5.7.2).

The relations in *Section 5* give the demand value of $\mu_\phi$ in terms of the basic value of the behaviour factor, $q_o$, by combining equation (D5.10) with equations (D2.1) and (D2.2), along with $\mu_\theta = \mu_\delta$:

$$\mu_\theta = 2q_o - 1 \qquad \text{if } T \geq T_C \tag{D5.11a}$$

$$\mu_\phi = 1 + 2(q_o - 1)\frac{T_C}{T} \qquad \text{if } T < T_C \tag{D5.11b}$$

where $T$ and $T_C$ are as in equations (D2.1) and (D2.2), with both $q_o$ and $T$ referring to the vertical plane in which bending of the element detailed takes place. The basic value $q_o$ of the behaviour factor is used in equations (D5.11), instead of the final value $q$ that may be lower than $q_o$ due to irregularity in elevation or a low aspect ratio of the walls, because these factors are considered to reduce the global ductility capacity for given local ductility capacities (e.g. due to non-uniform distribution of the ductility and deformation demand to elements in the case of heightwise irregular buildings). By the same token, in torsionally flexible systems a $q$ factor value higher than that used to reduce the elastic spectrum should be specified for use in equations (D5.11), as the elements on the perimeter of these systems may be subjected to higher ductility and deformation demands than the rest of the system; if this is the case, the designer is advised to detail the elements on the perimeter of torsionally flexible systems with additional caution and conservatism. This is not necessary for buildings characterized as inverted-pendulum systems, because with the already low basic values $q_o$ of the behaviour factor, such systems will essentially respond elastically to the design seismic action.

It should be recalled that the basic values of the $q$ factor in Table 5.1 (as well as the final $q$ factor value derived from them after any reduction due to irregularity in elevation or wall

aspect ratio) represent the upper limit of $q$ to be used in the derivation of the design spectrum from the elastic response spectrum. Even if the designer chooses to use a lower value than the upper limit he or she is entitled to for the ductility class used in a particular project, neither the required curvature ductility factor from equations (D5.11) nor the prescriptive detailing rules for elements are relaxed.

In recognition of the possible reduction of member flexural ductility when less-ductile steel is used as longitudinal reinforcement (cf. term involving $\varepsilon_{su}$ in equation (D5.8)), *Section 5* requires increasing by 50% the value of $\mu_\phi$ over that given by equations (D5.11), in the 'critical regions' of primary seismic elements where steel of Class B in EN 1992-1-1 (Table C.1) is used (as allowed in buildings of DCM). Nonetheless, because detailing measures that use the resulting value of $\mu_\phi$ refer to the ductility of the section as controlled by the compression reinforcement and confinement of the compression zone, this measure will not compensate directly for the possible reduction in ductility due to the use of more brittle steel. It may have significant indirect effects, though, by alerting the designer to the increased risk from the use of such steels and encouraging him or her to use steel Class C – or choosing DCL instead, where there is no penalty for the use of steel Class B, as design does not rely on ductility. *Clause 5.2.3.4(4)*

As mentioned on pp. 102 and 104, the factor of 1.5 for overstrength of materials and elements which is built into the $q$ factor value is not removed when $q$ is used in equations (D5.8) for the calculation of $\mu_\phi$. In ductile walls designed to Eurocode 8, the lateral force resistance – which is the quantity directly related to the $q$ factor – depends only on the flexural capacity of the base section. So, the ratio $M_{Rd}/M_{Ed}$ – where $M_{Ed}$ is the bending moment at the base from the analysis in the seismic design situation and $M_{Rd}$ is the design value of the resistance under the corresponding axial force from the analysis – expresses the element overstrength. *Section 5* allows calculation of $\mu_\phi$ at the critical regions of ductile walls using in equations (D5.11) the value of $q_o$, divided by the minimum value of the ratio $M_{Rd}/M_{Ed}$ in the seismic design situation. It might be more representative – albeit less convenient at the design stage – to use instead the ratio $\sum M_{Rd}/\sum M_{Ed}$, where both summations refer to all the walls in the system. On the same grounds, a reduction of the demand value of $\mu_\phi$ in the critical regions of beams and columns due to overstrength might also be justified. But, unlike the plastic hinge at a wall base, which controls the force capacity of an entire wall, which in turn may be an important individual contributor to the lateral strength of the structural system, plastic hinges in individual beams and columns are minor contributors to the global force capacity; so there is no one-to-one correspondence between the deformation demands on a plastic hinge and its flexural overstrength to support a simple rule for a reduction of the demand value of $\mu_\phi$ locally. *Clauses 5.4.3.4.2(2), 5.5.3.4.5(2)*

## 5.6.4. Capacity design of members against pre-emptive shear failure

### *5.6.4.1. Introduction*

As already noted, a mechanism of force transfer dominated by shear does not provide energy dissipation under cyclic loading. More importantly, once the shear reinforcement yields, the resistance degrades fast with cycling, leading to failure at relatively low deformations. So, this mechanism does not lend itself to ductile inelastic behaviour, and should be constrained in the elastic range. This is achieved by dimensioning concrete members in shear, not for their force demands from the analysis but for the maximum shear forces that may physically develop in them. This maximum value of the shear force is computed by expressing (through equilibrium) the shear force in terms of the bending moments at the nearest sections where plastic hinges may form and assuming that these bending moments are equal to the corresponding flexural capacities. As the bending moment in these sections cannot physically exceed the capacity in flexure, including the effect of strain hardening, the so-computed shear force is the maximum possible. Once dimensioned for this design force, a member will remain elastic in shear until and after the development of plastic hinges in the sections that affect the value of the shear force. *Clause 5.2.3.3(1)*

*5.6.4.2. Capacity design shear force in beams and columns*

*Clauses 5.4.2.3, 5.5.2.2(3)*

The column of Fig. 5.4 may develop plastic hinges at the two end sections 1 and 2, unless at one or both of these ends, plastic hinges develop first in the beams framing into the same joint as the end in question (as is normally the case in columns designed to fulfil equation (D4.23)). At the moment this happens the sum of column moments above and below the joint is equal to the total flexural capacity of the beam on opposite sides of that joint, $\sum M_{\mathrm{Rd,\,b}}$. It may be assumed that this sum is shared by the two column sections above and below the joint in proportion to their own flexural capacities. Then, the bending moment at the end section $i$ (= 1, 2) of the column may be taken equal to the design value of the moment resistance of the column at that end, $M_{\mathrm{Rd,\,ci}}$, mutiplied by $\sum M_{\mathrm{Rd,\,b}}/\sum M_{\mathrm{Rd,\,c}}$, where $\sum M_{\mathrm{Rd,\,b}}$ refers to the sections of the beam on opposite sides of the joint at end $i$, and $\sum M_{\mathrm{Rd,\,c}}$ to the sections of the column above and below the same joint. The sense of action of $\sum M_{\mathrm{Rd,\,c}}$ on the joint is the same as that of $M_{\mathrm{Rd,\,ci}}$, while that of $\sum M_{\mathrm{Rd,\,b}}$ is opposite. So, the design shear value of the column $i$ is taken as

$$\max V_{\mathrm{CD,\,c}} = \frac{\gamma_{\mathrm{Rd}}\left[M_{\mathrm{Rd,\,c1}}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,b}}}{\sum M_{\mathrm{Rd,\,c}}}\right)_1 + M_{\mathrm{Rd,\,c2}}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,b}}}{\sum M_{\mathrm{Rd,\,c}}}\right)_2\right]}{l_{\mathrm{cl}}} \qquad \text{(D5.12)}$$

In equation (D5.12) the factor $\gamma_{\mathrm{Rd}}$ accounts for possible overstrength due to steel strain hardening, and is taken equal to $\gamma_{\mathrm{Rd}} = 1.1$ for columns of DCM and to $\gamma_{\mathrm{Rd}} = 1.3$ for those of DCH; $l_{\mathrm{cl}}$ is the clear length of the beam between the end sections.

*Clauses 5.4.2.2, 5.5.2.1(3)*

The beam of Fig. 5.5 will develop plastic hinges at the two end sections 1 and 2, except in the rare case that at one or both of these ends, plastic hinges develop first in the column framing into the same joint as the end in question. With the same reasoning as for equation (D5.12), the design value of the maximum shear at a section $x$ in the part of the beam closer to end $i$ is taken as

$$\max V_{i,\mathrm{d}}(x) = \frac{\gamma_{\mathrm{Rd}}\left[M^{-}_{\mathrm{Rd},bi}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_i + M^{+}_{\mathrm{Rd},bj}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_j\right]}{l_{\mathrm{cl}}} + V_{g+\psi 2\mathrm{q,o}}(x) \qquad \text{(D5.13a)}$$

In equation (D5.13a) $j$ denotes the other end of the beam (i.e. if $i = 1$, then $j = 2$); the capacity of the beam $M_{\mathrm{Rd,\,b}}$ is taken for negative (hogging) bending at end $i$ and in positive (sagging) bending at the opposite end $j$. All moments and shears in equation (D5.13a) have positive sign. The sense of action of $(\sum M_{\mathrm{Rd,\,b}})_i$ on the joint is the same as that of $M_{\mathrm{Rd,\,}bi}$, while that of $(\sum M_{\mathrm{Rd,\,c}})_i$ is opposite (the same at end $j$). Factor $\gamma_{\mathrm{Rd}}$ accounts again for possible overstrength due to steel strain hardening, and is taken equal to $\gamma_{\mathrm{Rd}} = 1$ for beams of DCM and to $\gamma_{\mathrm{Rd}} = 1.2$ for beams of DCH. $l_{\mathrm{cl}}$ is the clear length of the beam between the end sections, and $V_{g+\psi 2\mathrm{q,\,o}}(x)$ is the shear force at cross-section $x$ due to the vertical loads in the seismic design situation, $g + \psi_2 q$, with the beam considered as simply supported (index: o). $V_{g+\psi 2\mathrm{q,\,o}}(x)$ may be conveniently computed (especially if the loads $g + \psi_2 q$ are not uniformly distributed along the length of the beam) from the results of the analysis of the structure for the vertical loads, $g + \psi_2 q$, alone, as the shear force $V_{g+\psi 2\mathrm{q,\,o}}(x)$ at cross-section $x$ in the full structure, corrected for the shear force $(M_{g+\psi 2\mathrm{q,\,1}} - M_{g+\psi 2\mathrm{q,\,2}})/l_{\mathrm{cl}}$ due to the bending moments $M_{g+\psi 2\mathrm{q,\,1}}$ and $M_{g+\psi 2\mathrm{q,\,2}}$ at the end sections 1 and 2 of the beam in the full structure. With $V_{g+\psi 2\mathrm{q,\,o}}(x)$ taken as positive at sections $x$ in the part of the beam closer to end $i$, the minimum shear in that section is

$$\min V_{i,\mathrm{d}}(x) = -\frac{\gamma_{\mathrm{Rd}}\left[M^{+}_{\mathrm{Rd},bi}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_i + M^{-}_{\mathrm{Rd},bj}\min\left(1,\ \frac{\sum M_{\mathrm{Rd,c}}}{\sum M_{\mathrm{Rd,b}}}\right)_j\right]}{l_{\mathrm{cl}}} + V_{g+\psi 2\mathrm{q,o}}(x) \qquad \text{(D5.13b)}$$

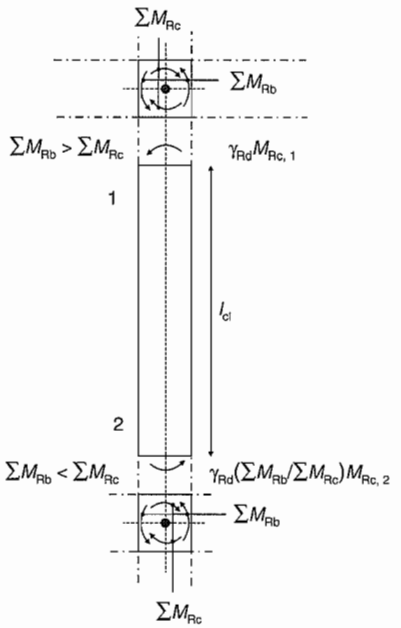



**Fig. 5.4.** Determination of the capacity design shear force in columns

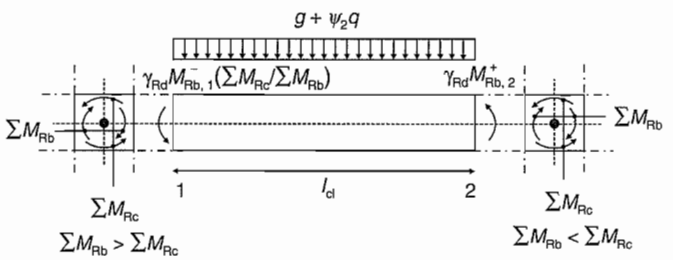



**Fig. 5.5.** Determination of the capacity design shear force in beams

As the moments and shears on the right-hand side of equation (D5.13b) are positive, the outcome may be positive or negative. If it is positive, the shear at section $x$ will not change the sense of action despite the cyclic nature of the seismic loading; if it is negative, the shear does change sense. As described in detail in Section 5.7.6, the ratio

$$\zeta_i = \frac{\min V_{i,d}(x_i)}{\max V_{i,d}(x_i)} \tag{D5.14}$$

is used in the dimensioning of the shear reinforcement of DCH beams as a measure of the reversal of the shear force at end $i$ (similarly at end $j$).

The design shear force in primary seismic columns and beams of buildings of DCM or DCH is always computed through equations (D5.12) and (D5.13), without exemptions. In beams and columns with short clear length $l_{cl}$, these expressions give a large value of the design shear force. Short columns are very vulnerable to the high shear force resulting from equation (D5.12), and special precautions should be taken at the conceptual design stage to avoid them. In short beams the last term in equations (D5.13) is small, and equation (D5.14) gives a value of $\zeta_i$ close to –1. Although not so problematic as short columns, short beams are difficult to dimension for the high shear force from equation (D5.13a) and for a value of $\zeta_i$ close to –1. (cf. Section 5.7.6, p. 122). So they should also be avoided, through proper spacing of the columns. It is noted at this point that although coupling beams of shear walls may be short, they are subject to special dimensioning and detailing rules to ensure ductile behaviour under their high and fully reversing shear forces.

The values of the $M_{Rd,b}$ in equations (D5.12) and (D5.13) can be computed from equations (D5.1) and (D5.2), and those of $M_{Rd,c}$ from equations (D5.3) and (D5.4). $M_{Rd,c}$ should be computed for the value of the axial load that is most unfavourable for the verification in shear. For the columns, as

- the shear resistance increases with the value of the axial load (both the shear resistance controlled by transverse reinforcement, $V_{Rd,s}$, and that controlled by diagonal compression in the web of the member, $V_{Rd,max}$, cf. Section 5.7.6) and
- the shear force demand from equation (D5.12) increases with the moment resistance of the column, $M_{Rd,c}$, and in turn $M_{Rd,c}$ increases when the axial load increases up to the balance load (i.e. the load at which crushing at the extreme compression fibres takes place is exactly when the tensile reinforcement reaches its yield stress)

the most unfavourable of the following two cases should be considered,

(1) the minimum value of column axial forces in the seismic design situation from the analysis
(2) the value of the axial load, within its range of variation in the seismic design situation, for which $M_{Rd,c}$ becomes a maximum. This is the value of $M_{Rd,c}$ computed for the minimum of the following two values: the maximum value of the column normalized axial load in the seismic design situation, $\nu_{max}$, and the balance load, $\nu_b$,

$$\nu_b = \frac{(\varepsilon_{cu} - \varepsilon_c/3) + (\varepsilon_{cu} - \varepsilon_y)\omega_v/(1-\delta_1)}{\varepsilon_{cu} + \varepsilon_y} - \omega_1 - \frac{\delta_1}{1-\delta_1}\omega_v + \omega_2 \qquad \text{(D5.15)}$$

The value of $M_{Rd,c}$ for the balance load $\nu_b$, can be computed from equation (D5.2) with $\xi$ taken as

$$\xi_{cu} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} \qquad \text{(D5.16)}$$

The variables in equations (D5.15) and (D5.16) are as defined for equations (D5.3) and (D5.4), and are computed using the design values $f_{yd}$ and $f_{cd}$ as $f_y$ and $f_c$, respectively; the conventional values, $\varepsilon_{c2} = 0.002$, $\varepsilon_{cu2} = 0.0035$, are used as $\varepsilon_c$ and $\varepsilon_{cu}$, respectively, in equations (D5.15) and (D5.16).

The axial load in beams is normally zero, so the values of $M_{Rd,c}$ in equations (D5.13) should be the maximum ones determined according to point 2 above.

When the value of the design shear force from equations (D5.12) and (D5.13) is so high that it exceeds the shear resistance, as this is controlled by diagonal compression (web crushing), then it will normally be more effective for the eventual fulfilment of the verification of the beam or column in shear to reduce its cross-sectional dimensions, than to increase them. The member flexural capacity, $M_{Rd}$, that determines to a large extent the magnitude of the design shear force from equations (D5.12) and (D5.13) is more sensitive to the cross-sectional dimensions of the member than its shear resistance, as this is controlled by diagonal compression, $V_{Rd,max}$. This is more so when the member longitudinal reinforcement is controlled by minimum requirements, or if the change in cross-sectional dimensions has a more-than-proportional effect on the moments (from the analysis) for which the longitudinal reinforcement is proportioned (this is normally the case in columns exempt from the satisfaction of equation (D4.23) and in beams with reinforcement at the supports controlled by the seismic design situation and not by vertical loads).

#### *5.6.4.3. Capacity design shear force in ductile walls*

*Clauses 5.4.2.4(6), 5.5.2.4.2*

Ductile walls are designed to develop a plastic hinge only at the base section and to remain elastic throughout the rest of their height. The value of the flexural capacity at the base section of the wall, $M_{Rdo}$, and equilibrium alone are not sufficient for the determination of the maximum seismic shears that can develop at various levels of the wall, because, unlike in

the beam of Fig. 5.5, the horizontal forces and the moments applied to the wall at floor levels are not constant but change during the seismic response. In the face of this difficulty a first assumption is that if $M_{Rdo}$ exceeds the bending moment at the base as obtained from the elastic analysis for the design seismic action, $M_{Edo}$, seismic shears at any level of the wall will exceed those from the same elastic analysis in proportion to $M_{Rdo}/M_{Edo}$. So, the shear force from the elastic analysis for the design seismic action, $V'_{Ed}$, is multiplied by a capacity design magnification factor $\varepsilon$ which takes up the following values:

In buildings of DCH:

- for 'squat' walls (those with a ratio of height to horizontal dimension, $h_w/l_w$, $\leq 2$):

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = 1.2\left(\frac{M_{Rdo}}{M_{Edo}}\right) \leq q \qquad \text{(D5.17)}$$

- for 'slender' walls (those with a ratio of height to horizontal dimension, $h_w/l_w$, $> 2$): *Clauses 5.5.2.4.1(6), 5.5.2.4.1(7), 5.4.2.4(7)*

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \sqrt{\left(1.2\frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \leq q \qquad \text{(D5.18)}$$

In buildings of DCM:

- for simplicity:

$$\varepsilon = 1.5 \qquad \text{(D5.19)}$$

The value of $\varepsilon$ from equations (D5.17) and (D5.18) should not be taken greater than the value of the $q$ factor, so that the final design shear, $V'_{Ed}$, does not exceed the value $qV'_{Ed}$ corresponding to fully elastic response. Moreover, it should not be taken as less than the constant value of 1.5 provided for DCM.

As described in Section 5.8.3, $\varepsilon$ values higher than those of equations (D5.17) and (D5.18) are specified for large lightly reinforced walls, which are always designed for DCM, and are often squat.

The factor 1.2 in equations (D5.17) and (D5.18) attempts to capture the overstrength at the base over the design value of the flexural capacity there, $M_{Rdo}$, e.g. owing to strain hardening of vertical steel. In the second term under the square root sign of equation (D5.18), $S_e(T_1)$ is the value of the elastic spectral acceleration at the period of the fundamental mode in the horizontal direction (closest to that) of the wall shear force which is multiplied by $\varepsilon$, and $S_e(T_C)$ is the spectral acceleration at the corner period, $T_C$, of the elastic spectrum. This latter term aims at capturing the increase of shear force over the elastic overstrength value represented by the first term, due to higher-mode effects in the elastic and the inelastic regime of the response, after a proposal by Eibl and Keintzel.[63] In modes higher than the first one, the ratio of the shear force to the bending moment at the base exceeds the corresponding value at the fundamental mode considered to be primarily (if not exclusively) reflected by the results of the elastic analysis. The longer the period $T_1$ of the fundamental mode, the lower the value of $S_e(T_1)$ and the higher that of $\varepsilon$, reflecting the more significant effect of higher modes on the shears. It should be pointed out, though, that equation (D5.18) has been proposed as a correction factor primarily on the results of the 'lateral force' (equivalent static) procedure of analysis for the design seismic action. If the elastic analysis is indeed dynamic ('modal response spectrum' analysis), then its results reflect the effects of higher modes on – at least the elastic – seismic shears.

Higher-mode effects on inelastic shears are larger in the upper storeys of the wall, and indeed more so in dual structural systems. The frames of such systems restrain the walls in the upper storeys, and the shear forces at the top storey of the walls from the 'lateral force procedure' of elastic analysis are opposite to the total applied seismic shear, becoming zero one or two storeys below. Multiplication of these very low storey shears by the factor $\varepsilon$ of equations (D5.16)–(5.18) will not bring their magnitude anywhere close to the relatively high *Clause 5.4.2.4(8)*

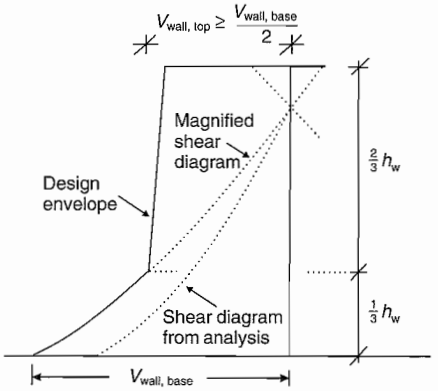



**Fig. 5.6.** Design shear forces in walls of dual structural systems

storey shears that may develop there due to higher modes (cf. dotted curves representing the shear forces from the analysis and their magnification by $\varepsilon$ in Fig. 5.6). In the face of the unrealistically low magnified shear forces in the upper storeys, *Section 5* requires that the minimum design shear of ductile walls in dual systems is at the top at least equal to half of the magnified shear at the base, increasing linearly towards the magnified value of the shear, $\varepsilon V'_{Ed}$, at one third of the wall height from the base (Fig. 5.6).

If the axial force in the wall from the analysis for the design seismic action is large (e.g. in slender walls near the corner of high-rise buildings, or in coupled walls), there will be a large difference between the absolutely maximum and minimum axial force in the individual walls in the seismic design situation (including the axial force due to gravity loads). As the vertical reinforcement at the base of the wall is controlled by the case in which the bending moment from the analysis, $M_{Edo}$, is combined with the minimum axial compression, the flexural capacity when the maximum axial compression is considered at the base, $M_{Rdo}$, is much larger than $M_{Edo}$. Then, the value of $\varepsilon$ from equation (D5.17) may be so high that the verification of the individual walls in shear (especially against failure by diagonal compression) may be unfeasible.

#### *5.6.4.4. Capacity design shear in beam–column joints*

*Clause 5.5.2.3*

Unlike gravity loading, which normally induces bending moments in beams which are of the same sign at opposite sides of a joint, seismic loading induces very high shear forces in beam–column joints. The magnitude of the shear in a joint can be appreciated if that joint is considered as part of the beam and it is noticed that the beam bending moment changes from a (high) negative value to a positive one across the joint, producing a vertical shear force, $V_{jv}$, equal to the average of the product of the seismic shear force in the beams, $V_b$, and their clear span, $L_{bn}$, divided by the column depth, $h_c$. Similarly, if the joint is considered as part of the column, the change in the column bending moment from a high value at the face of the joint above to an equally high value of opposite sign at the face below produces a horizontal shear force, $V_{jh}$, equal to the average of the product of the seismic shear force in the columns above and below the joint, $V_c$, and their clear storey height, $h_{stn}$, divided by the beam depth, $h_b$. These shear forces correspond to a nominal shear stress in the concrete of the joint equal to the ratio of $\sum M_c = \sum M_b$ to the volume of the joint, taken equal to $h_c h_b b_j$, where $b_j$ is the effective width of the joint, taken according to *Section 5* as

$$\text{if } b_c > b_w, \text{ then } b_j = \min\{b_c;\ (b_w + 0.5h_c)\};\ \text{otherwise } b_j = \min\{b_c;\ (b_w + 0.5h_c)\} \qquad \text{(D5.20)}$$

Shear stresses are introduced into a joint mainly through bond stresses along the beam and column bars framing the core of the joint. Because the nominal shear stress in the concrete of the joint is the same, regardless of whether it is computed from the horizontal or the vertical shear force, $V_{jh}$ or $V_{jv}$ respectively, from the capacity design point of view it is

more convenient to compute it from $V_{jh}$ based on the forces transferred via bond stresses along the top bars of the beam, as beams – even those not fulfilling equation (D4.23) – normally yield before the columns (this is on the safe side for the joint, even if beams do not yield). If bond failure along the top bars of the beam does not occur, the maximum possible value of $V_{jh}$ can be computed as the sum of the maximum possible tensile force in the top bars $A_{sb1}f_y$ on one side of the joint plus the maximum possible compressive force in the top flange on the opposite side, minus the shear force $V_c$ in the column above the joint. Irrespective of how it is shared by the concrete and the top reinforcement, the maximum possible compression force in the top flange will be controlled by the bottom reinforcement, and will be equal to its maximum possible tensile force, $A_{sb2}f_y$. Therefore, the design value of the horizontal shear force in the joint is

$$V_{jhd} = \gamma_{Rd}(A_{sb1} + A_{sb2})f_{yd} - V_c \qquad \text{(D5.21)}$$

where the beam reinforcement is taken at its overstrength, $\gamma_{Rd}f_{yd}$, and the shear force $V_c$ in the column above may be taken equal to the value from the analysis for the seismic design situation. It is obvious from the derivation of equation (D5.21) that in the sum $(A_{sb1} + A_{sb2})$ the top beam reinforcement area, $A_{sb1}$, refers to one vertical face of the joint and the bottom one, $A_{sb2}$, to the opposite face, so that the larger of the two sums should be considered. Normally, though, no such distinction needs to be made, especially as in interior joints the same bar area is provided at either side of the joint. At exterior joints only one term in the sum $(A_{sb1} + A_{sb2})$ should be considered.

Equation (D5.21) is applied with an overstrength factor of $\gamma_{Rd} = 1.2$ for beam–column joints of DCH buildings. For simplicity, in DCM buildings the beam–column joints are not dimensioned in shear on the basis of the shear force computed from equation (D5.21) but are treated through prescriptive detailing rules that have proved fairly effective in protecting joints in past earthquakes.

## 5.7. Detailing rules for the local ductility of concrete members

### 5.7.1. Introduction

Some of the detailing rules in *Section 5* for beams, columns and walls are prescriptive and originate from the tradition of earthquake-resistant design in the different seismic regions of Europe. The most important of the detailing and special dimensioning rules, though, have a rational basis. These rules and their justification/derivation are given in the following sections. Prescriptive detailing rules in *Section 5* are overall slightly stricter than those provided by US codes[39,40] for the corresponding ductility class (with 'Intermediate' considered to correspond to DCM and 'Special' to DCH). Rules for anchorage of beam bars at or through beam–column joints are more detailed and more demanding than in US codes.

### 5.7.2. Minimum longitudinal reinforcement in beams

Although:

- earthquakes impose deformations on structures and their members, not forces, and
- under deformation-controlled conditions, concrete members fail in flexure when their ultimate deformation capacity is reached, regardless of their force capacity,

*Clauses 5.2.3.7(3)(b), 5.4.3.1.2(5), 5.5.3.1.3(5), 5.2.3.7(2)(d)*

an underreinforced beam may fail abruptly in flexure in a force-controlled manner, if its cracking moment exceeds its yield moment. The reason is the inherently brittle nature of concrete cracking and the large deformation energy released when this happens, especially if the beam cross-sectional area is large and that of the longitudinal reinforcement is small. So, enough longitudinal reinforcement should be provided to ensure that the yield moment of the beam exceeds its cracking moment. Because the seismic bending moments in the beam are very uncertain, this requirement is imposed on all sections of a beam and for both signs of bending, irrespective of the moment from the analysis for the seismic design situation.

The minimum reinforcement area, $A_{s,\,min}$, should be sufficient to sustain, through its yield force, $A_{s,\,min} f_y$, the full tensile force released when concrete cracks. For a linear stress distribution in the cross-section, this force is equal to $0.5 f_{ct} b h_t$, where $b$ and $h_t$ are the width and depth of the tension zone, respectively, before cracking. Beams commonly have a T section, and the neutral axis of the uncracked section is very close to the compression flange (in the slab) for positive moments, so that it can be conservatively assumed that $h_t \approx 0.9h \approx d$. For negative moments the tension zone normally extends over the effective flange (in the slab), and its depth and width are quite uncertain; however, it can be assumed again that $bh_t \approx bd$, where $b$ and $d$ are the width and effective depth of the rectangular web of the T section. Then, the minimum ratio of reinforcement with respect to $bd$ is

$$\rho_{min} = \frac{A_{s,\,min}}{bd} = \frac{0.5 f_{ct} b h_t}{bd f_{yk}} \approx 0.5 \frac{f_{ctm}}{f_{yk}} \qquad \text{(D5.22)}$$

where the mean value, $f_{ctm}$, is used for the tensile strength of concrete, and the characteristic or nominal value, $f_{yk}$, for the yield stress of the longitudinal reinforcement. It is noted that the real danger for the section is fracture of the minimum reinforcement and that the margin between its tensile strength, $f_t$, and $f_{yk}$, which is of the order of 25%, provides some safety against overstrength of the concrete in tension (the 95% fractile of $f_{ct}$ exceeds $f_{ctm}$ by 30%).

### 5.7.3. Maximum longitudinal reinforcement ratio in the critical regions of beams

*Clauses 5.2.3.4(2), 5.2.3.7(3)(a), 5.4.3.1.2(3), 5.4.3.1.2(4)*

In beams the value of $\mu_\phi$ specified via equations (D5.11) for plastic hinge regions is provided through an upper limit on the ratio of the tension longitudinal reinforcement in the critical regions, $\rho_{1,\,max} = A_{s1,\,max}/bd$. The value of $\rho_{1,\,max}$ is derived as follows.

When the tension reinforcement is less than that in compression, $A_{s1} < A_{s2}$, the ultimate deformation at the end of the beam will take place when the effective ultimate strain of the tension reinforcement, $\varepsilon_{su}$, is exhausted. With the restrictions on steel classes allowed in DCM or DCH buildings posed in *Section 5* and the penalty on $\mu_\phi$ when steel of Class B is used in DCM buildings as noted in Section 5.6.3.2 (see p. 105), it is expected that this condition will not be reached before the end of the beam attains its ultimate deformation by failure of the compression zone, when the larger of the two reinforcements is in tension: $A_{s1} > A_{s2}$. The limit of $\rho_{1,\,max}$ refers to this latter situation. Therefore, with $\mu_\phi$ taken as $\phi_u/\phi_y$, $\phi_u$ is given by the second term in parentheses in equation (D5.8). In that term, $\varepsilon_{cu}$ is taken equal to the ultimate strain given in Eurocode 2 for unconfined concrete, $\varepsilon_{cu2} = 0.0035$, because ductility of the beam critical regions does not rely on confinement of the compression zone; $x_{cu}$ is taken equal to $x_{cu} = \xi_{cu} d$, with $\xi_{cu}$ given by equation (D5.3) with $\omega_v = 0$, $\nu = 0$ and with the conventional values $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$ for $\varepsilon_c$ and $\varepsilon_{cu}$, respectively. Using in $\mu_\phi = \phi_u/\phi_y$ the semi-empirical value $\phi_y = 1.5\varepsilon_y/d$ derived from test results of beams at yielding, the outcome for the upper limit value of the beam tension reinforcement ratio, $\rho_1$, is

$$\rho_{1,\,max} = \rho_2 + \frac{0.0019}{\varepsilon_y \mu_\phi} \frac{f_c}{f_y} \qquad \text{(D5.23)}$$

where $\rho_2 = A_{s2}/bd$ is the compression reinforcement ratio. Both $\rho_1$ and $\rho_2$ are normalized to the width $b$ of the compression flange, not of the web. The expression adopted in *Section 5* for the upper limit value of the beam tension reinforcement ratio, $\rho_1$, involves the design values, $f_{cd} = f_{ck}/\gamma_c$ and $f_{yd} = f_{yk}/\gamma_s$, of the concrete and steel strengths and the corresponding value $\varepsilon_{yd} = f_{yd}/E_s$ of $\varepsilon_y = f_y/E_s$:

$$\rho_{1,\,max} = \rho_2 + \frac{0.0018}{\varepsilon_{yd} \mu_\phi} \frac{f_{cd}}{f_{yd}} \qquad \text{(D5.24)}$$

As noted in Section 5.6.3.2 (see p. 104), for the value of 0.3 of the ratio $L_{pl}/L_s$ representative of typical beams in buildings, application of equation (D5.10) gives a safety factor of about 1.35 with respect to the more realistic values provided by inverting equation

(D5.5) – or of 1.9 if it is recognized that only $q/1.5$ produces inelastic deformation and ductility demands. With the value of 0.0018 of the coefficient in the second term on the right-hand side of equation (D5.24), the safety factor on $\mu_\phi$ becomes $1.35 \times 0.0019 \times 1.5/(1.15)^2/0.0018 \approx 1.6$ when the values $\gamma_c = 1.5$ and $\gamma_s = 1.15$ recommended in Eurocode 2 for the persistent and transient design situation are used, or $1.35 \times 0.0019/0.0018 \approx 1.4$ if the values $\gamma_c = 1.0$ and $\gamma_s = 1.0$ recommended in Eurocode 2 for the accidental design situation are used instead. The ratio between these implicit safety factors is: $1.6/1.4 \approx 1.15$, i.e. equal to the partial factor of steel in the persistent and transient design situation, consistent with adopting, or not, this safety factor in the seismic design situation. This 'theoretical' safety factor can be compared with the ratio of (1) the real value of $(\rho_1 - \rho_2)$ in beams cyclically tested to flexural failure to (2) the value obtained from equations (D5.24) and (D5.10) for the value of $\mu_\theta$ at beam ultimate deflection. The median value of the ratio in 52 beam tests is 0.725 for $\gamma_c = 1$ and $\gamma_s = 1$, or 0.825 if $\gamma_c = 1.5$ and $\gamma_s = 1.15$ is used. Being less than 1.0, these values suggest that equation (D5.24) is unconservative. However, if the value of $\mu_\theta$ is determined not as the ratio of beam ultimate deflection to the experimental yield deflection but to the value $M_y L_s/3(0.5EI)$ that corresponds to the assumed effective elastic stiffness of $0.5EI$ in Eurocode 8, the median ratio in the 52 tests becomes 2.5 for $\gamma_c = 1$ and $\gamma_s = 1$, or 2.85 for $\gamma_c = 1.5$ and $\gamma_s = 1.15$, i.e. above the 'theoretical' safety factors of 1.4 or 1.6 above.

Equation (D5.24) is quite restrictive for the top reinforcement ratio at beam supports, especially if the value of $\mu_\phi$ is high, as in, for example, DCH buildings with high basic values of the $q$ factor. To accommodate the area of top reinforcement required to satisfy the ULS in bending at beam supports for the seismic design situation without an undue increase in the beam cross-section, the bottom reinforcement ratio $\rho_2$ may be increased beyond the value $\rho_{min}$ from equation (D5.22), and the prescriptive minimum of $0.5\rho_1$ specified by *Section 5* for the bottom reinforcement in beam critical regions.

### 5.7.4. Maximum diameter of longitudinal beam bars crossing beam–column joints

*Clause 5.6.2.2(2)*

Shear forces are introduced to beam–column joints primarily through bond stresses along the beam and column longitudinal bars framing the joint core. Equation (D5.21) above giving the design shear force in the joint presumes that bond strength along the beam top bars is sufficient for the transfer of this shear force. Although loss of bond along these bars will not have dramatic global consequences, it would be better avoided through verification of bond along the bars of the beam. This verification has the form of an upper limit of the diameter of the longitudinal bars of the beam, $d_{bL}$, that pass through interior beam–column joints or are anchored at exterior ones. This upper limit is derived as follows.

If l and r (denoting 'left' and 'right') index the two vertical faces of the joint, $\sigma_s$ is the stress in the beam bars, and if $h_{co}$ is the width of the confined core of the joint parallel to the depth $h_c$ of the column, then the average bond stress along these beam bars is

$$\tau_b = \frac{\pi d_{bL}^2}{4} \frac{|\sigma_{s1} - \sigma_{s2}|}{\pi d_{bL} h_{co}} = \frac{d_{bL}}{4} \frac{|\sigma_{s1} - \sigma_{s2}|}{h_{co}} \qquad \text{(D5.25)}$$

with bond stresses along the length of the bars outside the confined core considered negligible. Plastic hinges are assumed to develop in the beam at both the left and right faces of the joint. As the top flange is normally much stronger than the bottom flange both in tension and in compression, its force cannot be balanced unless the bottom bars yield. So, in the bottom bars we have $\sigma_{s,l} = -f_y$ and $\sigma_{s,r} = f_y$, and $\tau_b$ is equal to $d_{bL} f_y / 2h_{co}$. Regarding the top bars, it is assumed that at beam plastic hinging they yield at the face at which they are in tension: $\sigma_{s,l} = f_y$. At the right face of the joint their compressive stress, $\sigma_{s,r}$, is such that, together with the force of the concrete in the top flange, $F_{c,r}$ (negative, as compressive), it balances the tension force in the bottom bars. These latter bars have a cross-sectional area $A_{s,r2}$, and at plastic hinging they are forced by the stronger top flange to yield, so that

$$\sigma_{s,r} = -\frac{A_{s,r2}}{A_{s,r1}} f_y - \frac{F_{c,r}}{A_{s,r1}} = -\frac{\rho_2}{\rho_1} f_y \left(1 - \frac{\xi_{eff}}{\omega}\right) \quad \text{(D5.26)}$$

where $\rho_1$ and $\rho_2$ are the ratios of top and bottom reinforcement at the right face normalized to the product $bd$ of the beam, $\omega$ is defined as $\omega = \rho_1 f_y / f_c$ and $\xi_{eff}$ is the depth of a fictitious compression zone, normalized to $d$, such that $F_{c,r} = -bd\xi_{eff} f_c$. Therefore, at the top bars $\tau_b$ is equal to

$$\tau_b = \frac{d_{bL}}{4} \frac{f_y}{h_{co}} \left[1 + \frac{\rho_2}{\rho_1}\left(1 - \frac{\xi_{eff}}{\omega}\right)\right] \quad \text{(D5.27)}$$

and its value is lower than along the bottom bars for the same value of $d_{bL}$. However, the bond problem seems to be more acute along the top bars, because bond stresses are not uniformly distributed around the perimeter of the bar but are concentrated more on the side facing the joint core. At the top bars this is the underside of the bar, where bond conditions are considered 'poor' due to the effects of laitance and consolidation of concrete during compaction. At the bottom bars bond conditions are considered 'good'.

According to Eurocode 2 the design value of the ultimate bond stress is $2.25 f_{ctd}$ for 'good' bond conditions, and 70% of that value for 'poor' conditions. The design value of the concrete tensile strength is $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7 f_{ctm}/\gamma_c$. As the consequences of bar pull out from the joint core will not be catastrophic (it will increase the apparent flexibility of the frame and the interstorey drifts and it may prevent the beam from reaching its full flexural capacity at the joint face), basing the design bond strength on the 5% fractile of the tensile strength of concrete and – in addition – dividing it by the partial factor for concrete seems unduly conservative. So, this partial factor is not applied here. As bond outside the confined joint core is neglected, the positive effects of confinement by the joint stirrups, the top bars of the transverse beam and the large volume of the surrounding concrete are considered according to the CEB/FIP Model Code 90,[64] i.e. by doubling the design value of the ultimate bond stress instead of dividing it by 0.7 according to Eurocode 2. The result for the top bars ('poor' bond conditions), equal to $0.7 \times 2.25 \times 0.7 f_{ctm} \times 2 = 2.2 f_{ctm}$, may be increased by the friction due to the normal stress on the bar–concrete interface, $\sigma \cos^2 \varphi$, produced by the mean vertical compressive stress in the column above the joint, $\sigma = N_{Ed}/A_c = \nu_d f_{cd}$. Using the design value $\mu = 0.5$ specified in Eurocode 2 for the friction coefficient on an interface with the roughness characterizing that between the concrete and the bar and integrating the friction force $\mu\sigma \cos^2 \varphi$ around the bar (i.e. between $\varphi = 0$ and 180°), friction increases the design value of bond strength to $2.2 f_{ctm} + 0.5 \times 0.5 \nu_d f_{cd} \approx 2.2 f_{ctm}(1 + 0.8\nu_d)$. The factor 0.8 in parentheses incorporates a value of 10.5 for the ratio of $f_{ck} = 1.5 f_{cd}$ to $f_{ctm}$ (this ratio varies between 9 and 11.8 for C20/25 to C45/55, and the value of 10.5 corresponds to C30/37). Setting $\tau_b$ from equation (D5.27) equal to this design value of bond strength along the top bars, the following condition is derived for the diameter of beam longitudinal bars in beam–column joints, $d_{bL}$:

- in interior beam–column joints

$$\frac{d_{bL}}{h_c} \le \frac{7.5 f_{ctm}}{\gamma_{Rd} f_{yd}} \frac{1 + 0.8\nu_d}{1 + k\rho_2/\rho_{1,max}} \quad \text{(D5.28a)}$$

- in beam–column joints which are exterior in the direction of the beam

$$\frac{d_{bL}}{h_c} \le \frac{7.5 f_{ctm}}{\gamma_{Rd} f_{yd}} (1 + 0.8\nu_d) \quad \text{(D5.28b)}$$

where the overstrength coefficient for the beam bars, $\gamma_{Rd}$, is taken to be equal to 1.0 for DCM and to 1.2 for DCH. The coefficient $k$ represents $(1 - \xi_{eff}/\omega)$ in equation (D5.27); in equation (D5.28a) the coefficient is taken equal to $k = 0.5$ for DCM and to $k = 0.75$ for DCH. In exterior beam–column joints we have $\sigma_{s2} = 0$, which is equivalent to $k = 0$, giving equation (D5.28b). The value of $\nu_d = N_{Ed}/f_{cd} A_c$ should be computed from the minimum value of $N_{Ed}$ in

the seismic design situation; although no special instructions are given in Eurocode 8 for tensile net axial forces (as may occur in exterior columns of medium- or high-rise buildings), it is clear from the way equations (D5.28) are derived that in that case $\nu_d = 0$.

It is most convenient to apply equations (D5.28) at the stage of initial sizing of columns, on the basis of the desired maximum value of beam bar diameter. This can be done on the basis of a rough estimate of the minimum axial load ratio $\nu_d$ in the seismic design situation (corresponding to only the gravity loads in interior columns and gravity minus axial forces due to the overturning moment in exterior ones). At that stage the final value of the top reinforcement ratio $\rho_1$ in equation (D5.27) will not be known, so in equation (D5.28a) the value of $\rho_1$ in equation (D5.27) was taken equal to the maximum value allowed, $\rho_{1,\max}$, from equation (D5.24). At the same stage the bottom steel ratio $\rho_2$ may be taken to be equal to the minimum value from equation (D5.22), or to $0.5\rho_{1,\max}$. These convenient choices for $\rho_2$ and $\rho_{1,\max}$ are unconservative for $d_{bL}$. This should be viewed, though, bearing in mind that equation (D5.28a) is very demanding for the size of interior columns: a column size $h_c$ of over $40d_{bL}$ is required for DCH, common values of axial load ($\nu_d \sim 0.2$), steel with nominal yield stress of 500 MPa and relatively low concrete grade (C20/25) – i.e. $h_c$ over 0.6 m if $d_{bL} = 14$ mm and over 0.8 m when $d_{bL} = 20$ mm. The requirement is relaxed to about $30d_{bL}$ for medium-high axial loads and higher concrete grades. If DCM is chosen, the required column size is reduced by about 25%. Although onerous, such requirements are justified by tests: cyclic tests[65] on interior joints show that the cyclic behaviour of beam–column subassemblages with $h_c = 18.75d_{bL}$ is governed by bond slip of the beam bars within the joint and is characterized by low-energy dissipation and rapid stiffness degradation; a column size of $h_c = 37.5d_{bL}$ was needed for the cyclic behaviour of the subassemblage to be governed by flexure in the beam and to exhibit stable hysteresis loops with high energy dissipation (subassemblages with $h_c = 28d_{bL}$ gave intermediate results). According to Kitayama *et al.*,[66] the energy dissipated by subassemblages with $h_c = 20d_{bL}$ cycled to a storey drift ratio of 2% corresponds to an effective global damping ratio of only 8%.

Although equation (D5.28a) has been derived for the top bars, according to Eurocode 8 it applies to the bottom bars of the beam as well. For the bottom bars the denominator in the second term of equation (D5.28a) should be replaced by 2, and term $7.5f_{ctm}$ in the numerator should be divided by 0.7, to account for the 'good' bond conditions. The end result is about the same as that from equation (D5.28a), so for simplicity the same expression is used for bottom bars as well. It should be noted, though, that for the bottom bars of exterior joints, equation (D5.28b) is conservative by a factor of about 0.7 for the required column depth $h_c$ due to 'good' bond conditions.

For exterior joints, equation (D5.28b) is conservative for both the top and bottom bars for another reason: although at the exterior face of such joints, top beam bars are normally bent down and bottom bars up, equation (D5.28b) takes into account bond only along the horizontal part of these bars and discounts completely the contribution of the 90° hook or bend. Underpinning this are Table 8.2 and clause 8.4.4 of Eurocode 2, according to which only the straight part of the bar counts toward anchorage in compression. The potential of push-out of 90° hooks or bends, if the straight part of the bar is not sufficient to transfer the full bar yield force to the joint, was also behind the adoption of Eurocode 2 in this respect. However, 90° hooks or bends near the exterior face of such joints are protected from push-out – as well as from opening up and kicking out the concrete cover when in tension – by the dense stirrups placed in the joint between the 90° hook or bend and the external surface. Moreover, top bars are normally protected from yielding in compression by the overstrength of the top flange relative to the tensile capacity of the bottom flange. So, only the bottom bars may yield in compression at an exterior joint; but for them the margin of about 0.7 for $h_c$ noted above is available. The same margin of about 0.7 for $h_c$ is available according to the Eurocode 2 rules for anchorage in tension of top bars with a 90° standard hook or a bend near the exterior face of the joint. On these grounds, 70% of the value of $h_c$ required by equation (D5.28b) may be used at exterior joints, without reducing their safety against bond failure below that provided by equation (D5.28a) for interior ones. *Section*

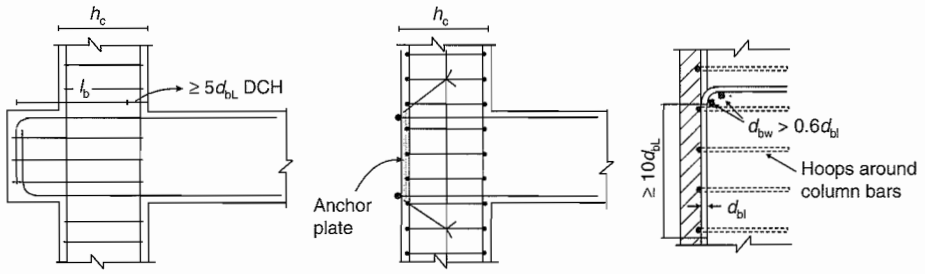



**Fig. 5.7.** Detailing arrangements in exterior beam–column joints proposed in *Section 5* as an alternative to straight anchorage of beam bars

*5* proposes the anchorage arrangements in Fig. 5.7 as an alternative to increasing the column size or reducing the diameter of beam bars in exterior beam–column joints to meet equation (D5.28b).

Equations (D5.28) lead to the use of square columns in two-way frames. Moreover, unless column sizes are large for other design reasons (drift control, strong column–weak beam design to satisfy equation (D4.23), etc.), equations (D5.28) lead also to small diameters of beam bars. To prevent them from buckling, such bars need to be restrained by closely spaced stirrups, especially at the bottom of the beam which lacks the lateral restraint provided at the top by the slab.

### 5.7.5. Verification of beam–column joints in shear

*Clauses 5.5.3.3(1), 5.5.3.3(2), 5.5.3.3(3)*

Assuming that bond strength along the beam and column bars framing the joint core is sufficient to transfer into the joint the full shear force demand, given by equation (D5.20) in terms of the horizontal shear force, $V_{jhd}$, the body of the joint then resists that shear. This shear force is translated into a shear stress, considered uniform within the joint volume, defined by the horizontal distance between the extreme layers of column reinforcement, $h_{jc}$, the net depth of the beam between its top and bottom reinforcement, $h_{jw}$, and the (horizontal) width, $b_j$, of the joint given by equation (D5.20):

$$v_j = \frac{V_{jhd}}{b_j h_{jc}} \tag{D5.29}$$

There is no universally accepted rational model for the mechanism through which the joint resists cyclic shear and ultimately fails. Experimental results on interior joints collected and compiled by Kitayama *et al.*[66] suggest that the joint shear resistance, expressed in terms of the shear stress, $v_j$, of equation (D5.29), increases about linearly with the ratio of horizontal reinforcement within the joint, $\rho_{jh}$, from $v_j \approx 0.15f_c$ for $\rho_{jh} = 0$ (unreinforced joint) to a limit value between $v_j \approx 0.24f_c$ and $v_j \approx 0.4f_c$ (mean value: $v_j \approx 0.32f_c$) at $\rho_{jh} = 0.4\%$. Above that value of the steel ratio and up to $\rho_{jh} = 2.4\%$, ultimate strength seems to always be attained by diagonal compression in the concrete and to be practically independent of the value of $\rho_{jh}$ and of the axial load ratio in the column, $\nu = N/f_c A_c$.

Guided by the test results mentioned above and in view of the lack of consensus on models, *Section 5* has adopted a very simple plane stress model for the verification of the shear strength of beam–column joints in DCH buildings. The model assumes homogeneous stresses in the body of the joint, consisting of:

(1) the shear stress, $v_j$, from equation (D5.29)
(2) the vertical normal stress, $-N/A_c = -\nu f_c = -\nu_d f_{cd}$ (compression), from the column
(3) a horizontal normal stress, $-\rho_{jh} f_{yw}$ (compression), as a reaction to the tensile force that develops in the horizontal reinforcement when the latter is driven to yielding by the dilatancy of the joint at imminent failure.

Joint strength criteria are based on the principal stresses, in tension, $\sigma_I$, and compression, $\sigma_{II}$, under the system of stresses 1–3 above. The required ratio of horizontal reinforcement, $\rho_{jh}$, is obtained from the condition that $\sigma_I$ does not exceed the concrete tensile strength, $f_{ct}$,

$$\rho_{jh} f_{yw} \geq \frac{v_j^2}{f_{ct} + \nu f_c} - f_{ct} \tag{D5.30a}$$

or, using the design values of the strengths, including $f_{ctd} = f_{ctk,\,0.05}/\gamma_c = 0.7 f_{ctm}/\gamma_c$,

$$\frac{A_{sh} f_{ywd}}{b_j h_{jw}} \geq \frac{(V_{jhd}/b_j h_{jc})^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd} \tag{D5.30b}$$

where $A_{sh}$ denotes the total area of the horizontal legs of hoops within the joint, between the top and bottom reinforcement of the beam. For a safe-sided (conservative) estimate of $A_{sh}$, $\nu_d$ in equation (D5.30b) is computed from the minimum value of the axial force of the column above the joint in the seismic design situation. It is noteworthy that for $\rho_{jh} = 0$ equation (D5.30a) gives values of $v_j$ ranging from $0.1 f_c$ to $0.2 f_c$ for values of $\nu$ between 0 and 0.3, in good agreement with the average value of $v_j \approx 0.15 f_c$ suggested for $\rho_{jh} = 0$ by the compilation of test results by Kitayama *et al.*[66]

The other verification condition is that $\sigma_{II}$ does not exceed the concrete compressive strength, as this is reduced due to the presence of tensile stresses and/or strains in the transverse direction (i.e. that of $\sigma_I$). The reduced compressive strength is taken to be equal to $\eta f_{cd} = 0.6(1 - f_{ck}(\text{MPa})/250) f_{cd}$ (the reduction factor $\eta$ is the same as factor $\nu$ applied on $f_{cd}$ in clause 6.2.3 of Eurocode 2 for the calculation of the shear resistance of concrete members, as this is controlled by diagonal compression in the concrete; the symbol $\eta$ was used in Eurocode 8, to avoid confusion with the frequently used normalized axial load $\nu$). The – adverse – effect of the horizontal normal stress, $-\rho_{jh} f_{yw}$, on the magnitude of $\sigma_{II}$, as well as its (more important) favourable effect on the compressive strength in the diagonal direction through confinement, are both neglected. So the condition $-\eta f_{cd} \leq \sigma_{II}$ gives

$$V_{jhd} \leq \eta f_{cd} \sqrt{1 - \frac{\nu_d}{\eta}}\, b_j h_{jc} \tag{D5.31}$$

Equation (D5.31) is the verification criterion of interior beam–column joints against diagonal compression failure. At exterior joints we rely on 80% of the value in equation (D5.31):

$$V_{jhd} \leq 0.8 \eta f_{cd} \sqrt{1 - \frac{\nu_d}{\eta}}\, b_j h_{jc} \tag{D5.32}$$

Unlike equations (D5.30), where for the verification to be safe-sided (conservative) the minimum value of the column axial force in the seismic design situation should be used, equations (D5.31) and (D5.32) should employ the maximum value of the column axial force in the seismic design situation (including the effect of the overturning moment in exterior joints). For common values of $\nu_d$ (~0.25), equation (D5.31) gives values of the shear stress, $v_j$, close to $0.4 f_{cd}$, which is at the upper limit of the strength values compiled by Kitayama *et al.*[66] for interior joints. Experimental results suggest that an ultimate value of the shear stress, $v_j$, close to $0.4 f_{cd}$ can be attained in columns with a slab at the level of the top of the beam and a transverse beam on both sides of the joint. For exterior joints, which are normally checked with a higher value of $\nu_d$ due to the effect of the overturning moment on column axial force, equation (D5.31) gives results close to the mean experimental value of $0.32 f_{cd}$ observed in interior joints without transverse beams and a top slab. The conclusion is that, unless the value of $f_{cd} = f_{ck}/\gamma_c$ uses a partial factor for concrete, $\gamma_c$, (significantly) higher than 1.0, equations (D5.31) and (D5.32) do not provide a safety margin against failure of the joint by diagonal compression.

As an alternative to equations (D5.30), *Section 5* derives the joint horizontal reinforcement from a physical model proposed by Park and Paulay.[67] According to that model, a joint resists shear via a combination of two mechanisms: *Clause 5.5.3.3(4)*

(1) a diagonal concrete strut between the compressive zones of the beams and columns at opposite corners of the joint
(2) a truss extending over the entire core of the joint, consisting of
   - (any) horizontal hoops in the joint
   - (any) vertical bars between the corner bars of the column (including column longitudinal bars contributing to the flexural capacity of the end sections of the column above and below the joint)
   - a diagonal compression field in the concrete.

The force in the strut is assumed to develop from:

- the concrete forces in the beam and column compression zones at the two ends of the strut
- the bond stresses transferred to the joint core within the width of the strut itself.

The truss resists the rest of the joint shear force. Then, for the dimensioning of the horizontal joint reinforcement to be safe-sided (conservative), the horizontal component of the strut force should not be overestimated. With this in mind, the assumption in Paulay and Priestley[68] is adopted, namely that at the face of the joint where the beam is in positive bending (tension at the bottom) the crack cannot close at the top flange, due to accumulation of plastic strains in the top reinforcement. This is very conservative for the truss and its horizontal joint reinforcement, because the compression zone of the beam does not deliver a horizontal force to the concrete strut, but only a compressive force to the beam top reinforcement to be transferred (together with the tension force at the opposite face of the joint) to the truss and the strut, in proportion to their share in the joint width at the level of the top reinforcement. As the horizontal width of the strut at that level is equal to the depth of the compression zone of the column above the joint, $x_c$, and assuming – for simplicity – that the transfer of the total force $(A_{sb1}+A_{sb2})f_y$ by bond takes place uniformly along the total length, $h_c$, of the top bars within the joint, a fraction of this force equal to $x_c/h_c$ goes to the horizontal force of the strut, and the rest, $(1-x_c/h_c)$, to the truss. It is both realistic and safe-sided for the truss horizontal reinforcement to consider that the column shear force, $V_c$, appearing as the last term in equation (D5.21) for $V_{jhd}$, is applied directly to the strut through the compression zone of the column above and affects only its horizontal shear force, not that of the truss. So, as the whole depth of the vertical faces of the joint are taken up by the truss, the total area, $A_{sh}$, of the horizontal legs of hoops within the joint should be dimensioned for the force $(1-x_c/h_c)(A_{sb1}+A_{sb2})f_y$. The value of $x_c/h_c$ may be computed from equation (D5.3), using $\omega_1=\omega_2$, $\omega_\nu=0$ (for convenience), $\varepsilon_{co}=0.002$ and $\varepsilon_{cu}=0.0035$ (for spalling of the extreme concrete fibres at the end section of the column). Then, $\xi_c\approx\nu_d/0.809=\nu_d/(1.5\times0.809)\approx0.8\nu_d$, with both $\nu_d$ and $\xi_c$ normalized to $h_c$. So, the following total area of horizontal hoops should be provided:

- At interior joints,

$$A_{sh}f_{ywd}\geq\gamma_{Rd}(A_{sb1}+A_{sb2})f_{yd}(1-0.8\nu_d) \qquad \text{(D5.33)}$$

  where $\gamma_{Rd}$ is taken equal to 1.2 (as in equation (D5.21) for DCH) and the normalized axial force $\nu_d$ is the minimum value in the column above the joint in the seismic design situation.

Reinforcement requirements at exterior joints cannot be obtained by setting $A_{sb2}=0$ in equation (D5.33). The underlying reason is that the beam top reinforcement is bent down at the far face of the joint, and when it is in tension it delivers at the bend to the diagonal strut, starting there the full diagonal compression force of the strut. The horizontal component of that force is close to $f_yA_{sb1}-V_c$, and so very little force is transferred by bond along the part of the top bars outside the strut, to be resisted as horizontal shear by the truss between the strut and the face of the joint towards the beam. What governs the horizontal shear force of the

truss is the force transferred by bond along the part of the bottom bars outside the strut (the upward bend of bottom bars at the far face of the joint does not deliver forces to the joint core when these bars are in compression). The compression zone at the bottom of the beam delivers to the bottom end of the strut a horizontal force equal to the compression force in the concrete, i.e. equal to the difference between the tension in the top reinforcement, $A_{sb1}f_y$, and the force in the bottom reinforcement, which yields in compression, $A_{sb2}f_y$. The difference between the horizontal component of the strut force at its top end, $A_{sb1}f_y - V_c$, and the horizontal forces delivered at its bottom end from the beam and the column below, $(A_{sb1} - A_{sb2})f_y - V_c$, and by bond within the strut width at the level of the bottom reinforcement, $(1 - x_c/h_c)A_{sb2}f_y$, is the force transferred by bond along the part of the bottom bars outside the strut width and to be to be resisted by the truss as horizontal shear between the strut and the external face of the joint. This gives:

- At exterior joints,

$$A_{sh}f_{ywd} \geq \gamma_{Rd}A_{sb2}f_{yd}(1 - 0.8\nu_d) \qquad \text{(D5.34)}$$

  where again $\gamma_{Rd} = 1.2$, but $\nu_d$ is the minimum value of the normalized axial force in the column below the joint in the seismic design situation.

The two alternative models, equations (D5.21) and (D5.30) and equations (D5.33) and (D5.34), give quite dissimilar results. The amount of reinforcement required according to equations (D5.21) and (D5.30) is very sensitive to the values of $\nu_d$ and $v_j$ (suggesting that according to this model the shear resisted by means of the diagonal tension mechanism is insensitive to the amount of horizontal reinforcement), whereas the joint reinforcement required according to equations (D5.33) and (D5.34) is rather insensitive to the value of $\nu_d$ and proportional to $v_j$. For medium-high values of $\nu_d$ (around 0.3) equations (D5.21) and (D5.30) require much less joint reinforcement than equations (D5.33) and (D5.34), whilst for low values of $\nu_d$ (around 0.15) equations (D5.21) and (D5.30) require less joint reinforcement than equations (D5.33) and (D5.34) for $v_j < 0.3f_{cd}$, and the opposite if $v_j > 0.3f_{cd}$. For near-zero values of $\nu_d$, equations (D5.21) and (D5.30) require much more joint reinforcement than equations (D5.33) and (D5.34), especially for high values of $v_j$. If this discrepancy is disturbing, even less reassuring is the difference between the predictions of either model and the experimental strength values compiled by Kitayama *et al.*[66] for interior joints: for a given shear stress demand, $v_j$, the experimental evidence is that much less joint reinforcement is needed than given by either of the two models. The only case of acceptable agreement with the test results is that of equations (D5.21) and (D5.30) for medium-high values of $\nu_d$ (around 0.3). The conclusion of these comparisons is that the designer may use with confidence the minimum of equations (D5.21) and (D5.30) and equations (D5.33) and (D5.34) for the steel requirements.

*Clauses 5.4.3.2.2(2), 5.4.3.2.2(11)(b), 5.4.3.3(3), 5.5.3.2.2(2), 5.5.3.2.2(12)(c), 5.5.3.3(9), 5.5.3.3(5)*

The truss mechanism underlying equations (D5.33) and (D5.34) includes as one of its components vertical reinforcement that provides the vertical tensile field equilibrating the vertical component of the diagonal compression field in the concrete. Intermediate bars between the corner ones, arranged along the sides of the column with depth $h_c$, can play that role, along with contributing to the flexural capacity of the end sections of the column above and below the joint. Such bars are provided along the perimeter at a spacing of not more than 150 mm for DCH or 200 mm for DCM, to improve the effectiveness of concrete confinement. For the present purposes, *Section 5* requires at least one intermediate vertical bar between the corner ones, even on short column sides (less than 250 mm for DCH or 300 mm for DCM).

For the joints of DCH buildings, where the horizontal joint reinforcement area, $A_{sh}$, needs to be calculated through equations (D5.21) and (D5.30) or equations (D5.33) and (D5.34), the total area of column intermediate bars between the corner ones, $A_{sv,i}$, should be determined from $A_{sh}$ as follows:

$$A_{sv,i} \geq \tfrac{2}{3}A_{sh}(h_{jc}/h_{jw}) \qquad \text{(D5.35)}$$

The coefficient $\frac{2}{3}$ accounts for the normally smaller inclination of the strut and the truss compression field to the vertical, compared with the diagonal of the joint core. It also limits the effect of the overestimation of $A_{sh}$ by equations (D5.21) and (D5.30) or equations (D5.33) and (D5.34) in affecting the vertical reinforcement as well.

*Clauses 5.4.3.3(1), 5.4.3.3(2), 5.5.3.3(7), 5.5.3.3(8)*

The computational verification of beam–column joints according to equations (D5.30)–(D5.35) is required only in DCH buildings. For DCM, the detailing measures prescribed by *Section 5* for both DCH and DCM joints without any calculation suffice. According to these measures, the transverse reinforcement placed in the critical regions of the column above or below (whichever is the greatest) should also be placed within the joint, except if beams frame into all four sides of the joint and their width is at least 75% of the parallel cross-sectional dimension of the column. In that case the horizontal reinforcement in the joint is placed at a spacing which may be double of that at the columns above and below, but not more than 150 mm.

To see what the prescriptive rules above imply for the minimum horizontal reinforcement in the joint, it is recalled that for DCH the critical regions of columns above the base of the building should be provided with a minimum design value of 0.08 for the mechanical volumetric ratio of transverse reinforcement, $\omega_{wd}$. For S500 steel and concrete grade C30/37, this value corresponds to $\rho_{jh} = 0.185\%$ per horizontal direction if the partial factors for steel and concrete are equal to their recommended values for the persistent and transient design situations, or to $\rho_{jh} = 0.24\%$ if they are set equal to the recommended value of 1.0 for the accidental design situation (for other concrete grades the minimum value of $\rho_{jh}$ is proportional to $f_c$). Although other constraints on the column transverse reinforcement in critical regions (e.g. that on the diameter and spacing of transverse reinforcement: $d_{bh} \geq \max(6\text{ mm}; 0.4d_{bL})$, $s_w \leq \min(6d_{bL}; b_o/3; 125\text{ mm})$, or on the minimum value of $\mu_\phi$ it ensures) may govern, it is indicative that the values quoted above for $\rho_{jh}$ are well below the value of 0.4% that marks the limit of the contribution of horizontal reinforcement to the shear resistance of the joint according to Kitayama *et al.*[66] For DCM, *Section 5* has no lower limit on $\omega_{wd}$ in the critical regions of columns, only a limiting hoop diameter ($d_{bh} \geq \max(6\text{ mm}; d_{bL}/4)$) and spacing ($s_w \leq \min(8d_{bL}; b_o/2; 175\text{ mm})$). These limit values give a low horizontal reinforcement ratio in the joint. Considering that the practical minimum for DCM is 8 mm hoops, with a horizontal spacing for the legs of 200 mm, at a hoop spacing of 125 mm, the resulting steel ratio in the joint is $\rho_{jh} = 0.2\%$ per horizontal direction.

### 5.7.6. Dimensioning of shear reinforcement in critical regions of beams and columns

*Clauses 5.4.3.1.1(1), 5.4.3.2.1(1), 5.5.3.2.1(1)*

The design value of the shear resistance of beams or columns is computed according to the rules of Eurocode 2 for monotonic loading, both when it is controlled by the transverse reinforcement, $V_{Rd,s}$, and when it is controlled by diagonal compression in the web of the member, $V_{Rd,max}$. There is one exception to this: the value of $V_{Rd,s}$ in the critical regions of beams of DCH. The special rules for $V_{Rd,s}$ in this particular case are described below.

*Clause 5.5.3.1.2(2)*

In the critical regions of beams of DCH the strut inclination, $\theta$, is taken equal to 45° ($\cot\theta = 1$). This is equivalent to a classical Mörsch–Ritter 45° truss with no concrete contribution term ($V_{cd} = 0$). The underlying reason is the experimentally observed reduction of $V_{Rd,s}$ in plastic hinges (i.e. after flexural yielding) with the magnitude of inelastic cyclic deformations. In members that have initially yielded in bending, this reduction manifests itself by a rapid increase of shear deformations with load cycling, leading ultimately to shear failure. This phenomenon is described conveniently and fairly accurately on the basis of a classical Mörsch–Ritter 45° truss model for shear resistance under cyclic loading with non-zero concrete contribution term, $V_c$, considering that either the $V_c$ or the sum of $V_c$ and the contribution of transverse reinforcement, $V_w$, decrease with the plastic part of the imposed displacement ductility factor, $\mu_\theta^{pl} = \mu_\theta - 1$. The models developed for concrete beams, columns (rectangular or circular) and walls in Biskinis *et al.*[69] and adopted in Annex A of EN 1998-3[52] are of this type (in units of meganewtons and metres):

$$V_{R,s} = \frac{h-x}{2L_s}\min(N;\ 0.55A_c f_c) + (1-0.05\min(5;\ \mu_\theta^{pl}))[V_w + V_c] \quad \text{(D5.36a)}$$

$$V_{R,s} = \frac{h-x}{2L_s}\min(N;\ 0.55A_c f_c) + V_w + (1-0.095\min(4.5;\ \mu_\theta^{pl}))V_c \quad \text{(D5.36b)}$$

where:

- $x$ is the compression zone depth
- $N$ is the compressive axial force in the seismic design situation (positive, zero for tension)
- $L_s$, equal to $M/V$, is the shear span at the end of the member end
- $A_c$ is the cross-section area, equal to $b_w d$ for a cross-section with a rectangular web of thickness $b_w$ and structural depth $d$, or to $\pi D_c^2/4$ (where $D_c$ is the diameter of the concrete core to the inside of the hoops) for circular sections
- the concrete contribution term is equal to

$$V_c = 0.16\max(0.5;\ 100\rho_{tot})\left(1-0.16\min\left(5;\ \frac{L_s}{h}\right)\right)\sqrt{f_c}A_c \quad \text{(D5.37)}$$

  where $\rho_{tot}$ is the total longitudinal reinforcement ratio
- the contribution of transverse reinforcement to shear resistance is equal to:
  (a) for cross-sections with rectangular web of width (thickness) $b_w$:

$$V_w = \rho_w b_w z f_{yw} \quad \text{(D5.38a)}$$

  where:
  - $\rho_w$ is the transverse reinforcement ratio
  - $z$ is the length of the internal lever arm ($z \approx d\text{-}d'$ in beams, columns, or walls with a barbelled or T section, $z \approx 0.8l_w$ in rectangular walls)
  - $f_{yw}$ is the yield stress of transverse reinforcement

  (b) for circular cross-sections

$$V_w = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}(D-2c) \quad \text{(D5.38b)}$$

  where:
  - $D$ is the diameter of the section
  - $A_{sw}$ is the cross-sectional area of a circular stirrup
  - $s$ is the centreline spacing of stirrups
  - $c$ is the concrete cover.

In buildings designed for plastic hinging in the beams, the value of $\mu_\theta$ in these beams is normally equal to the global displacement ductility factor, $\mu_\delta$, that corresponds to the value of $q$ used in the design via equations (D2.1) and (D2.2). Therefore, depending on the value of $\alpha_u/\alpha_1$ and the regularity classification of the building, the value of $\mu_\theta^{pl}$ ranges from 1.5 to 3.5 in DCM beams and from 2.5 to 5.5 in DCH ones. According to equation (D5.38a), the ensuing reduction of $V_{R,s}$ is small for DCM beams, but may be significant in DCH ones. For simplicity, in the case of DCM beams the reduction is neglected, and the normal expression for $V_{Rd,s}$ from Eurocode 2 is applied (that expression employs only the $V_w$ term from equations (D5.38), multiplied by $\cot\theta$, with $\cot\theta$ between 1 and 2.5). For DCH beams, where the reduction of $V_{R,s}$ with $\mu_\theta^{pl}$ in plastic hinges cannot be neglected, and as in the context of Eurocode 2 no $V_c$ term is used in the expression for $V_{Rd,s}$, $\cot\theta = 1$ is taken (cf. equations (D5.38)), which is equivalent to a reduction of $V_c$ to zero, instead of the reduction by about half suggested by equation (D5.36b). Figure 5.8, in which the test data used to fit equations (D5.36) and (D5.37) have been cast in the format of a model with variable strut inclination, $\theta$, shows by how much this approximation is conservative.

Plastic hinging is not expected in the columns of dissipative buildings designed to Eurocode 8. If it does take place, it will normally lead to lower chord rotation ductility demands and less

*Clause 5.5.3.2.1(1)*

ensuing reduction of the value of $V_{R,s}$ than in beams. It is expected that, if such a reduction occurs, its effects will be offset by the $\gamma_{Rd}$ factor of 1.1 for DCM and of 1.3 for DCH employed in the capacity design calculation of shear force demands (cf. equation (D5.12)). So, for columns the reduction of shear resistance in plastic hinges is neglected, and the normal expression for $V_{Rd,s}$ from Eurocode 2 is applied. That expression employs the $V_w$ term from equations (D5.38), multiplied by cot $\theta$, with cot $\theta$ between 1 and 2.5, as well as the contribution of the inclined compression chord given by the first term in equations (D5.36) without the $0.55A_cf_c$ upper limit. For simplicity, that term may be taken as equal to $(d - d_1)/l_{cl}$.

*Clause 5.5.3.1.2(2)*

The second point where the shear verification of plastic hinges in DCH beams deviates from the Eurocode 2 rules refers to the use of inclined bars at an angle $\pm\alpha$ to the beam axis against sliding in shear at the end section of the beam. Such sliding may occur in an instance when the crack is open throughout the depth of the end section and the shear force is relatively high. For this to happen, a significant reversal of the shear force is necessary, as well as a high value of the peak shear force. A value of $\zeta$ from equation (D5.14), which is algebraically less than –0.5, is the criterion adopted in *Section 5* for a significant reversal of the shear, and a value of the maximum shear from equation (D5.13a) greater than $(2 + \zeta)f_{ctd}b_wd$ is the limit for a peak shear capable of causing sliding for $\zeta < -0.5$. This limit shear is between one-third to one-half of the value of $V_{Rd,max}$ for cot $\theta = 1$. As the surface susceptible to shear sliding is not crossed by stirrups, if these limits are exceeded, inclined bars crossing this surface should be dimensioned to resist through the vertical components $A_sf_{yd}$ sin $\alpha$ of their yield force – in tension and compression – at least 50% of the peak shear from equation (D5.13a). The 50% value corresponds to the limit value $\zeta = -0.5$, and respects the recommendation of clause 9.2.2(3) in Eurocode 2 to resist at least 50% of the design shear through links. If the beam is short, the inclined bars are most conveniently placed along its two diagonals, as in coupling beams; then, tan $\alpha \approx (d - d')/l_{cl}$. If the beam is not short, then the angle $\alpha$ of the diagonals to the beam axis is small, and the effectiveness of inclined bars placed along them is also low; two series of shear links, one at an angle $\alpha = 45°$ to the beam axis and the other at $\alpha = -45°$, would be effective then. The construction difficulties and reinforcement congestion associated with such a choice are obvious, though. Normally there is neither risk from sliding shear nor a need for inclined reinforcement, if the configuration of the framing is selected to avoid beams that are relatively short and are not loaded with significant gravity loads in the seismic design situation (i.e. having a high value for the first term and a low value for the second term on the right-hand side of equations (D5.13)).

*Clause 5.5.3.2.1(1)*

Plastic hinges in columns are subjected to an almost full reversal of shear ($\zeta \approx -1$), and the peak value of the shear force from equation (D5.12) is normally high. However, no inclined

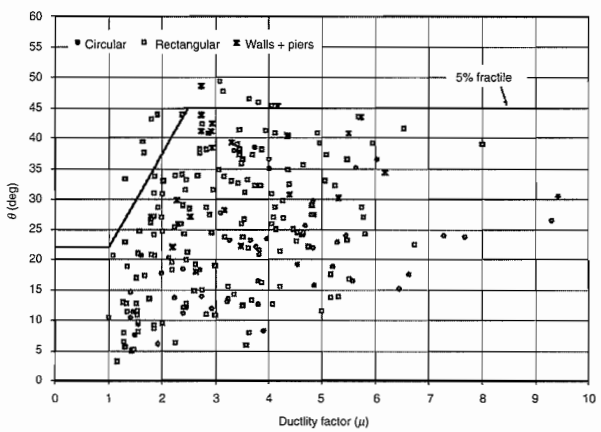



**Fig. 5.8.** Experimental data on the dependence of the strut inclination $\theta$ on the imposed chord rotation ductility ratio, for cyclic loading after flexural yielding[69]

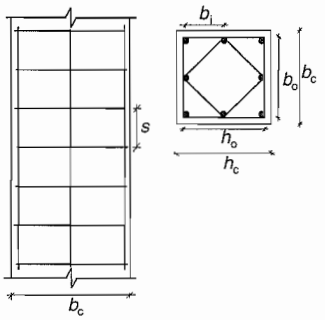



**Fig. 5.9.** Definition of geometric terms for concrete confinement in columns

bars are required for them, as, due to the axial force and the small magnitude of plastic strains in vertical bars, the crack is expected to always be closed over part of the depth of the end section. Moreover, sliding is resisted through clamping and dowel action in the large-diameter vertical bars, which are normally available between the extreme column reinforcement in the section and remain elastic at the instance of peak positive or negative response of the column. Verification against sliding shear and placement of inclined bars to resist it is required, though, in ductile walls of DCH (see Section 5.7.9), as in walls the axial load level is lower and the web bars are of smaller diameter and more sparse than in columns. An important practical difference between columns and walls in this respect is that, due to the size, density of transverse and longitudinal reinforcement and one-directional nature of the cross-sectional shape and function of the walls, inclined bars can be easily placed and are quite effective in shear; this is not the case in columns, on exactly the same grounds.

### 5.7.7. Confinement reinforcement in the critical regions of columns and ductile walls

*Clauses 5.4.3.2.2(7), 5.4.3.2.2(8), 5.4.3.4.2(2), 5.4.3.4.2(3), 5.4.3.4.2(4), 5.5.3.2.2(8), 5.5.3.2.2(9), 5.5.3.4.5(2), 5.5.3.4.5(3), 5.5.3.4.5(4)*

The longitudinal reinforcement of columns and walls is normally symmetric, $\rho_1 = \rho_2$. So, the value of $\mu_\phi$ specified via equations (D5.11) for the plastic hinges cannot be provided as in beams, i.e. by keeping the extreme concrete fibres below their ultimate strain through a low difference between the tension and compression reinforcement ratios, $\rho_1 - \rho_2$ (cf. equation (D5.23)). In columns and walls we let, instead, the extreme concrete fibres reach their ultimate strain and spall, but rely thereafter on the enhanced ultimate strain of the confined concrete core to the centreline of the hoops. In other words, the necessary value of $\mu_\phi$ is provided through confinement. The necessary amount of confinement reinforcement is derived as follows.

With the same reasoning as in Section 5.7.3, $\phi_u$ is given by the second term in parentheses in equation (D5.8), but this time applied to the reduced section of the confined core to the centreline of the hoops, which has depth $h_o = h_c - 2(c + d_{bh}/2)$, width $b_o = b_c - 2(c + d_{bh}/2)$ and effective depth $d_o = d - 2(c + d_{bh}/2)$, where $c$ denotes the concrete cover to the outside of the hoops, $h_c$ and $b_c$ are respectively the external dimensions of the original unspalled concrete section and $d_{bh}$ is the hoop diameter (Fig. 5.9). The strain at the extreme fibres of the confined core, $\varepsilon^*_{cu}$, is taken equal to the ultimate strain for confined concrete, $\varepsilon_{cu2,c}$, according to Eurocode 2 (equation (D5.6)). It should also be recalled that according to Eurocode 2 confinement enhances the strength of concrete and the corresponding strain to

$$f_{c,c} = \beta f_c \tag{D5.39}$$

$$\varepsilon_{c2,c} = \beta^2 \varepsilon_{c2} \tag{D5.40}$$

where

$$\beta = \min(1 + 2.5\alpha\omega_w;\ 1.125 + 1.25\alpha\omega_w) \tag{D5.41}$$

Using in $\mu_\phi = \phi_u/\phi_y$ the semi-empirical value $\phi_y = \lambda\varepsilon_y/h$, with $\lambda = 1.85$ for columns and $\lambda = 1.45$ for walls, derived from test results of columns or walls at yielding, the value of strain at the extreme fibres of the confined core, $\varepsilon_{cu}^*$, required for the target value of $\mu_\phi$ is

$$\varepsilon_{cu}^* = \lambda\mu_\phi\varepsilon_y\xi_{cu}^*\frac{h_o}{h_c} \tag{D5.42}$$

Variables denoted by an asterisk refer to the confined core, rather than to the original unspalled section. The value of the compression zone depth of the confined core, normalized to $h_o$ as $\xi_{cu}^*$, is given by equation (D5.3) with $\omega_1^* = \omega_2^*$, $\delta_1 = (h_o - d_o)/h_o = (d_{bL} + d_{bh})/2h_o \ll 1$ (hence $\delta_1 \approx 0$), $\omega_\nu^* = A_{s\nu}f_y/b_oh_of_{c,c}$, $\nu^* = N/b_oh_of_{c,c}$:

$$\xi_{cu}^* \approx \frac{\nu^* + \omega_\nu^*}{(1-\varepsilon_{c2,c}/3\varepsilon_{cu2,c}) + 2\omega_\nu^*} \approx \frac{\nu + \omega_\nu}{(1-\varepsilon_{c2,c}/3\varepsilon_{cu2,c})(f_{c,c}/f_c)(b_oh_o/b_ch_c) + 2\omega_\nu} \tag{D5.43}$$

where $\omega_\nu = A_{s\nu}f_y/h_cb_cf_c$, and $\nu = N/h_cb_cf_c$ is the mechanical reinforcement ratio of intermediate vertical bars (between the extreme tension and compression bars) and the axial load ratio in the unspalled section, respectively. After substitution of this expression for $\xi_{cu}^*$ into equation (D5.45), setting the resulting expression for $\varepsilon_{cu}^*$ equal to $\varepsilon_{cu2,c}$ from equation (D5.6), substitution of the values of $f_{c,c}$, $\varepsilon_{c2,c}$ from equations (D5.39) to (D5.41), and neglecting some terms as small (i.e. of second order), then for normal – i.e. low – values of $\alpha\omega_w$ the final result is

$$\alpha\omega_w \approx 10\lambda\mu_\phi\varepsilon_y(\nu + \omega_\nu)\frac{b_c}{b_o} - 0.0285 \tag{D5.44a}$$

or, after multiplying both sides of equation (D5.44a) by $(f_{yd}/f_y)(f_c/f_{cd}) = \gamma_c/\gamma_s$,

$$\alpha\omega_{wd} \approx 10\lambda\mu_\phi\varepsilon_{yd}(\nu_d + \gamma_s\omega_{\nu d})\frac{b_c}{b_o} - 0.0285\gamma_c/\gamma_s \tag{D5.44b}$$

Instead of equation (D5.44b) *Section 5* adopts the following expression:

$$\alpha\omega_{wd} = 30\mu_\phi\varepsilon_{yd}(\nu_d + \omega_{\nu d})\frac{b_c}{b_o} - 0.035 \tag{D5.45}$$

with $\omega_{\nu d}$ neglected in columns, as small in comparison with $\nu_d$. The last term is lower (more conservative) than the value $0.0285\gamma_c/\gamma_s = 0.037$ that results from the values of $\gamma_c$ and $\gamma_s$ recommended for the persistent and transient design situations and higher (less conservative) than the value of 0.0285 obtained from the values recommended for the accidental design situation. For the usual values of confinement reinforcement, the difference in the final confinement requirements corresponds to the difference in the $\gamma_s$ values ($\gamma_s = 1.15$ versus $\gamma_s = 1.0$). The difference between $10\lambda$ and the adopted value of 30 for the coefficient provides a safety factor for the average value of $\mu_\phi$ achieved for given value of $\alpha\omega_{wd}$. It should be recalled that, according to Section 5.6.3.2 (see p. 104):

- for the value of 0.4 of the ratio $L_{pl}/L_s$ representative of typical building columns, application of equation (D5.10) gives a safety factor of about 1.65 with respect to the more realistic values obtained by inverting equation (D5.5), or of 2.45 if it is recognized that only $q/1.5$ produces inelastic deformation and ductility demands
- in walls, for the value of 0.21 of $L_{pl}/L_s$ representative of typical ductile walls in buildings, equation (D5.10) gives a safety factor of about 1.1 with respect to the values obtained by inverting equation (D5.5), or of 1.2 when it is taken into account that only $q/1.5$ produces ductility demands.

The end result is an average safety factor for $\mu_\phi$ of

- $1.65 \times 30/(10 \times 1.85) \approx 2.65$ for columns or

- $1.1 \times 30/(10 \times 1.45) \approx 2.25$ for walls.

A larger safety factor is appropriate for columns, as (1) for them $\omega_{vd}$ is neglected compared with $\nu_d$, and (2) due to the large stiffness and resistance of walls relative to the foundation system and soil, part of the inelastic deformation demand at their base may be absorbed there, rather than at the plastic hinge of the wall. Values of the safety factor for $\mu_\phi$ of around 2.5 are fully justified in view of (1) the crucial importance of vertical elements for the integrity of the whole structural system, and (2) the large scatter and uncertainty in the correspondence between $\mu_\phi$ and $\mu_\theta$ evident from the experimental results. In fact, in view of this uncertainty, the 'theoretical' safety factor has been compared with the ratio of (1) the value of $\alpha\omega_{wd} + 0.035$ required from equations (D5.45) and (D5.10) in columns or walls cyclically tested to flexural failure for the value of $\mu_\theta$ at member ultimate deflection to (2) the value of $\alpha\omega_{wd} + 0.035$ provided in the tested member (which should be proportional to the available value of $\mu_\phi$ according to equation (D5.45)).[70] The median value of the ratio in 626 cyclic tests of columns with non-zero $\nu_d$ is 0.88 for $\gamma_c = 1$ and $\gamma_s = 1$, or 0.92 if $\gamma_c = 1.5$ and $\gamma_s = 1.15$ is used. The corresponding median values in 49 cyclic tests on flexure-controlled walls is 0.93 for $\gamma_c = 1$ and $\gamma_s = 1$, or 0.96 for $\gamma_c = 1.5$ and $\gamma_s = 1.15$. Values less than 1.0 suggest that equation (D5.45) is unconservative. However, if the value of $\mu_\theta$ is determined as the ratio of the member ultimate drift not to the experimental yield drift but to the value $M_y L_s/3(0.5EI)$ corresponding to the effective elastic stiffness of $0.5EI$ suggested by Eurocode 8 for the analysis of concrete or masonry buildings, the median ratio becomes 2.08 for $\gamma_c = 1$ and $\gamma_s = 1$ or 2.26 for $\gamma_c = 1.5$ and $\gamma_s = 1.15$ in the 626 column tests, and 2.69 for $\gamma_c = 1$ and $\gamma_s = 1$ or 3.13 for $\gamma_c = 1.5$ and $\gamma_s = 1.15$ in the 49 wall tests, i.e. not far from the 'theoretical' safety factors of 2.25 or 2.65 quoted above.

If equation (D5.45), applied with $b_o = b_c$, gives a negative result, the target value of $\mu_\phi$ can be achieved by the unspalled section without any confinement. Then in the critical regions considered, stirrups need to follow just the relevant prescriptive rules of the corresponding ductility class.

*Clauses 5.4.3.2.2(6), 5.5.3.2.2(6), 5.5.3.2.2(7)*

The confinement reinforcement computed from equation (D5.45) is not placed in all column critical regions indiscriminately, but only where plastic hinges will develop by design. These are only the critical regions at the base of DCM or DCH columns (i.e. at the connection to the foundation). In all other critical regions of DCM columns, only the prescriptive detailing rules apply – e.g. against buckling of rebars, etc. However, in DCH buildings the confinement reinforcement from equation (D5.45) should also be placed in critical regions at the ends of those columns which are not checked for fulfilment of equation (D4.23), as falling within the exemptions from this rule listed in Section 5.6.2.3. Moreover, in the critical regions of the ends of DCH columns which are protected from plastic hinging through fulfilment of equation (D4.23) in both horizontal directions, confining reinforcement should be placed, given from equation (D5.45) for the value of $\mu_\phi$ obtained from equations (D5.11) for two-thirds of the value of the basic $q$ factor value used in the design, and not for the full value.

Implicit in the derivations and rules above is the assumption that the section of the column or wall is rectangular. For such a section, equation (D5.45) should be applied, taking as the width $b_c$ the shorter side of the cross-section. In a rectangular column the outcome of equation (D5.45) for $\omega_{wd}$ should be implemented as the sum of the mechanical reinforcement ratios in both transverse directions, $(\rho_x + \rho_y)f_{ywd}/f_{cd}$, taking special care, though, to provide about equal transverse reinforcement ratios in both directions: $\rho_x \approx \rho_y$. The arrangement of confining reinforcement in walls, rectangular or not, is the subject of the next section.

For circular columns, the only change is in the confinement effectiveness factor $\alpha$. This factor is defined as the ratio of the minimum confined area of the core to the total core area. For circular columns the $\alpha$ factor is calculated via a variant of equation (D5.6) without the third factor and with the core dimensions $h_o$ and $b_o$ replaced by the diameter of the centreline of the circular hoops, $D_o$. If spiral reinforcement is used instead of individual hoops, the minimum confined area within the spiral gives $\alpha = 1 - s_h/2D_o$.

*Clauses 5.4.3.4.2(5), 5.5.3.4.5(5)*

Wall or column sections may consist of several rectangular parts orthogonal to each other (hollow rectangular sections, walls with barbells at the end of the cross-section, flanged sections with a T, L, double-T, U or even a Z shape with web perpendicular to the flanges, etc.). In these, the mechanical volumetric ratio of confining reinforcement should be determined separately for each rectangular part of the section that may act as a compression flange. In that case, equation (D5.45) should first be applied, taking as the width $b_c$ the external width of the section at the extreme compression fibres; that value of $b_c$ should also be used in the normalization of the axial force, $N_{Ed}$, and of the area of the vertical reinforcement between the tension and compression flanges, $A_{sv}$, as $\nu_d = N_{Ed}/h_c b_c f_{cd}$, $\omega_{\nu d} = (A_{sv}/h_c b_c) f_{yd}/f_{cd}$, with $h_c$ being the maximum dimension of the spalled section normal to $b_c$. In other words, in this calculation the section is taken as rectangular, with width $b_c$ and depth $h_c$. For this consideration to be representative of the conditions in the compression zone, the latter has to be limited to the compression flange of width $b_c$. To check this, the neutral axis depth at ultimate curvature after spalling of the concrete outside the confined core of the compression flange is calculated on the basis of the above considerations as

$$x_u = (\nu_d + \omega_{\nu d}) \frac{h_c b_c}{b_o} \tag{D5.46}$$

and is compared with the dimension of the rectangular compression flange normal to $b_c$ (i.e. parallel to $h_c$) after its reduction by $(c + d_{bh}/2)$ due to spalling of the cover concrete. If this latter value exceeds $x_u$, then the outcome of equation (D5.45) for $\omega_{wd}$ should be implemented through stirrups arranged in the compression flange considered. Although again about equal transverse reinforcement ratios should preferably be provided in both directions of this compression flange as $\rho_x \approx \rho_y$, what mainly counts in this case is the steel ratio of the stirrup legs which are normal to $b_c$.

If the value of $x_u$ from equation (D5.46) appreciably exceeds in size the dimension of the compression flange normal to $b_c$ after spalling of the cover concrete, there are three alternatives:

(1) The difficult option: *Section 5* recommends the computationally cumbersome and tricky option of generalizing the theoretically sound approach outlined above for the derivation of equations (D5.44) and (D5.45), on the basis of:
   - the definition of $\mu_\phi$ as $\mu_\phi = \phi_u/\phi_y$
   - the calculation of $\phi_u$ from the second term in equation (D5.8) as $\phi_u = \varepsilon_{cu2,c}/x_{cu}$ and of $\phi_y$ as $\phi_y = \varepsilon_{sy}/(d - x_y)$
   - estimation of the neutral axis depths $x_u$ and $x_y$ from the equilibrium of stresses over the section
   - equations (D5.6) and (D5.39)–(D5.41) for the properties of the confined concrete.

   The necessary amount of confinement reinforcement should be derived both for the compression flange of width $b_c$ and for the adjoining rectangular part of the section orthogonal to it (the 'web'). This derivation should provide the same safety margin for the value of $\mu_\phi$ as that given by the use of equation (D5.45) instead of equation (D5.46) (in other words, it should approximate the result of equation (D5.45) when applied to a rectangular section).

(2) The easy option: to increase the dimension of the rectangular compression flange normal to $b_c$, so that, after being reduced by $(c + d_{bh}/2)$ due to spalling, it exceeds the value of $x_u$ from equation (D5.46).

(3) The intermediate option: providing confinement only over the rectangular part of the section that is normal to the compression flange (the 'web'). This option is meaningful only when the compression flange for which the neutral axis depth has first been calculated via equation (D5.46) is shallow and not much wider than the web. Equation (D5.45) should then be applied, taking as the width $b_c$ the thickness of the web (also for the normalization of $N_{Ed}$ and $A_{sv}$ as $\nu_d$ and $\omega_{\nu d}$). The outcome of equation (D5.45) for $\omega_{wd}$

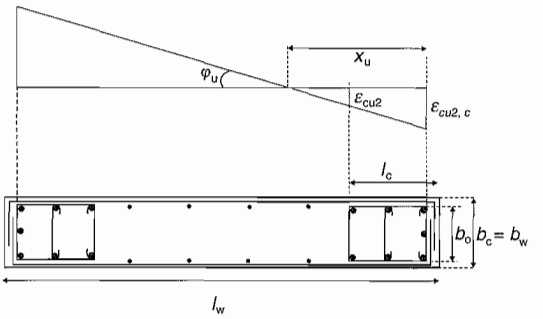



**Fig. 5.10.** Boundary elements in a rectangular wall and the strain distribution along the section at ultimate curvature

should be implemented through stirrups arranged in the web. It would be consistent with this approach to sacrifice the compression flange by placing, in its parts that protrude from the web, transverse reinforcement that meets only the prescriptive rules on stirrup spacing and diameter of the corresponding ductility class, irrespective of any confinement requirements; it is more prudent, though, to place in the flange the same confining reinforcement as in the web.

Although the approach above can be applied both to walls and to columns of composite section, it is specified for walls alone in *Section 5* (then $h_c$ is the wall length, $l_w$). The only differentiation of walls from columns in this respect is the extent of the confinement in the direction of the length, $l_w$, as described in the next section.

### 5.7.8. Boundary elements at section ends in the critical region of ductile walls

*Clauses 5.4.3.4.2(6), 5.5.3.4.5(6)*

As noted in the definition of walls in Section 5.2.2, what mainly differentiates the design and detailing of a wall as a concrete member from that of a column is that for a wall, flexural resistance is assigned to the opposite ends of the section (flanges, or tension and compression chords) and shear resistance to the web in between. This is accomplished by concentrating the vertical reinforcement and limiting the confinement of the concrete only at the two ends of the section, in the form of boundary elements (Fig. 5.10).

Confined boundary elements need to extend only over the part of the section where at ultimate curvature conditions the concrete strain exceeds the ultimate strain of unconfined concrete $\varepsilon_{cu2} = 0.0035$. This means that the centreline of the hoop enclosing a boundary element should have a length of $x_u(1 - \varepsilon_{cu2}/\varepsilon_{cu2,c})$ in the direction of the wall length, $l_w$, with the neutral axis depth after concrete spalling, $x_u$, estimated from equation (D5.46) for the value of $\alpha\omega_{wd}$ provided in the boundary element. The length of the confined boundary element from the extreme compression fibres, $l_c \geq x_u(1 - \varepsilon_{cu2}/\varepsilon_{cu2,c}) + 2(c + d_{bh}/2)$, should respect the prescriptive minimum value of $0.15l_w$ and $1.5b_w$.

Boundary elements with the confinement specified above are required only in the critical region at the base of DCM and DCH walls. In DCH walls they should be continued for one more storey with half of the confining reinforcement required in the critical region. Although not required by Eurocode 8, it is advisable to extend boundary elements to the top of the wall, with their minimum length and reinforcement. This is particularly so in barbelled walls, in which the barbells have to be detailed anyway as column-like elements.

### 5.7.9. Shear verification in the critical region of ductile walls

*Clause 5.4.3.4.1(1)*

Similarly to beams and columns, the design value of the shear resistance of ductile walls, as controlled by the transverse reinforcement, $V_{Rd,s}$, or by diagonal compression in the web, $V_{Rd,max}$, is computed according to the rules of Eurocode 2 for monotonic loading, except for

DCH walls and especially in their critical region. The special rules applicable for DCH walls are detailed below.

*Clause 5.5.3.4.2(1)*

In the critical region of DCH walls, the design value of the cyclic shear resistance, as controlled by diagonal compression in the web, $V_{\mathrm{Rd,\,max}}$, is taken as just 40% of the value given by Eurocode 2 for monotonic loading. It was found by Biskinis *et al.*[69] that cyclic loading drastically reduces this particular shear resistance of walls, and they fitted the following expression to it, adopted in Annex A of EN 1998-3[52] (with units of meganewtons and metres):

$$V_{\mathrm{R,\,max}} = 0.85(1-0.06\min(5;\,\mu_\theta^{pl}))\left(1+1.8\min(0.15;\,\frac{N}{A_c f_c}\right)\times \\ (1+0.25\max(1.75;\,100\rho_{\mathrm{tot}}))\left(1-0.2\min(2;\,\frac{L_s}{h}\right)\sqrt{\min(100;\,f_c)}b_w z \qquad \text{(D5.47)}$$

The variables in equation (D5.47), including the plastic part of the chord rotation ductility factor, $\mu_\theta^{\mathrm{pl}} = \mu_\theta - 1$, are as defined for equations (D5.36)–(D5.38). The limited test results for shear failure under cyclic loading by diagonal compression in the web prior to flexural yielding suggest that equation (D5.47) holds in that case as well, with $\mu_\theta^{\mathrm{pl}} = 0$. The test data to which equation (D5.47) was fitted show that, for values of $\mu_\theta$ representative of ductility demands in DCH walls, on average, the Eurocode 2 value of $V_{\mathrm{R,\,max}}$ (using the actual value of $f_c$ in lieu of $f_{cd}$) gives 40% of the experimental cyclic shear resistance. Hence the relevant rule of *Section 5* for the critical region of DCH walls. The difference is very large, and normally it should have been taken into account in the Eurocode 8 rules for the shear design of ductile walls of DCM as well. It was feared, though, that a large reduction of the design shear resistance, when applied together with the magnification of shears by the factor of equation (D5.19), might be prohibitive for the use of ductile concrete walls in earthquake-resistant buildings. So, it was decided to leave the design rules for DCM walls unaffected, at least until the reduction demonstrated by the currently available data is supported by more test results. For the time being, the designer is cautioned to avoid exhausting the present liberal limits for DCM ductile walls against diagonal compression in the web.

*Clauses 5.5.3.4.3(1), 5.5.3.4.3(3)*

The second point where shear design of DCH walls deviates from the general Eurocode 2 rules is in the calculation of the web reinforcement ratios, horizontal $\rho_h$ and vertical $\rho_v$, in those storeys of DCH walls where the shear span ratio, $\alpha_s = M_{\mathrm{Ed}}/V_{\mathrm{Ed}}l_w$, is less than 2. The maximum value of $M_{\mathrm{Ed}}$ in the storey (normally at its base) is used in the calculation of $\alpha_s$. Significant uncertainty exists regarding the cyclic behaviour of walls with $\alpha_s < 2$ that ultimately fail by diagonal tension (and hence are controlled by the web reinforcement), as most of the walls with $\alpha_s < 2$ which have been cyclically tested in the laboratory have failed by diagonal compression (and hence are included in the data that support equation (D5.47)). Unlike the relative abundance of data on this latter type of wall, only four out of the 26 laboratory walls that failed in shear by diagonal tension after flexural yielding and support equations (D5.36)–(D5.38) have $\alpha_s < 2$. In view of the lack of information specific to cyclic loading, the following modification of the rule given in clause 6.2.3(8) of Eurocode 2 for the calculation of the transverse reinforcement in members with $0.5 < \alpha_s < 2$ under monotonic loading has also been adopted for the determination of $\rho_h$ in those storeys where $\alpha_s < 2$:

$$V_{\mathrm{Rd,\,s}} = V_{\mathrm{Rd,\,c}} + \rho_h b_{\mathrm{wo}}(0.75 l_w \alpha_s) f_{\mathrm{yhd}} = V_{\mathrm{Rd,\,c}} + \rho_h b_{\mathrm{wo}}\left(0.75\frac{M_{\mathrm{Ed}}}{V_{\mathrm{Ed}}}\right) f_{\mathrm{yh,d}} \qquad \text{(D5.48)}$$

where $\rho_h$ is the ratio of horizontal reinforcement, normalized to the thickness of the web, $b_{\mathrm{wo}}$, and $f_{\mathrm{yh,\,d}}$ is its design yield strength. A $V_c$ term has been included, equal to the design shear resistance of concrete members without shear reinforcement according to Eurocode 2, $V_{\mathrm{Rd,\,c}}$. If $b_{\mathrm{wo}}$ and the effective depth, $d$, of the wall are expressed in metres, the wall gross cross-sectional area, $A_c$, in square metres, $V_{\mathrm{Rd,\,c}}$ and the wall axial force in the seismic design situation, $N_{Ed}$, in kilonewtons and if $f_{ck}$ is in megapascals, $V_{\mathrm{Rd,\,c}}$, as given in Eurocode 2, is

$$V_{\mathrm{Rd,c}} = \left\{ \min\left[ \frac{180}{\gamma_c}(100\rho_L)^{1/3},\ 35\sqrt{1+\sqrt{\frac{0.2}{d}}}\, f_{ck}^{1/6} \right] \left(1+\sqrt{\frac{0.2}{d}}\right) f_{ck}^{1/3} + 0.15\min\left(\frac{N_{Ed}}{A_c},\ 0.2\frac{f_{ck}}{\gamma_c}\right) \right\} b_{wo} d \qquad \text{(D5.49)}$$

where $\rho_L$ denotes the tensile reinforcement ratio, and $\gamma_c$ is the partial factor for concrete. However, in the critical region of walls, $V_{\mathrm{Rd,c}} = 0$ if $N_{Ed}$ is tensile (negative). The ratio of vertical web reinforcement, $\rho_v$, is then dimensioned to provide a 45° inclination of the concrete compression field in the web, together with the horizontal reinforcement and the vertical compression in the web due to minimum axial force in the seismic design situation, $\min N_{Ed}$. There is certainly room for future improvement of these rules, once more data become available on the cyclic behaviour and failure of low-shear-span-ratio walls by diagonal tension.

As mentioned in the closure to Section 5.7.6, sections of a DCH wall within its critical region should be verified against sliding shear. Verification may be limited to the storey end section(s) within the wall critical region, normally coinciding with a construction joint. If the critical region of the wall is limited to its bottom storey, only the base section needs to be verified. *Clause 5.5.3.4.4(1)*

The design resistance against sliding shear comprises three components: *Clause 5.5.3.4.4(2)*

(1) A dowel action term, equal to the minimum of the following:
   - The resistance of vertical bars in pure shear, taken as $0.25A_{sv}f_{yd}$, where $A_{sv}$ is the total area of the vertical bars in the web plus any additional vertical bars placed in the boundary elements specifically for the purpose of resistance to shear sliding without counting in the flexural reinforcement. The safety factor with respect to the yield force of a bar in pure shear (i.e. without axial force), which is equal to $A_{sv}f_{yd}/\sqrt{3}$, has a value of 2.3.
   - The dowel action resistance, as determined by the interaction between the bar and the surrounding concrete, taken to be equal to $1.3A_{sv}(f_{yd}f_{cd})^{1/2}$ with $A_{sv}$ as defined above. The safety factor with respect to the monotonic dowel action resistance of stress-free bars deeply embedded in concrete, which is equal to $1.3d_{bL}^2(f_{yd}f_{cd})^{1/2}$, is then $4/\pi = 1.275$.

   For lower concrete classes, e.g. below C25/30, the term $0.25A_{sv}f_{yd}$ governs. For the contribution of a bar to these sources of resistance to be fully available, its concrete cover should be at least $3d_{bL}$ in the direction of the thickness of the wall, at least $8d_{bL}$ along the wall length ahead of the bar (i.e. towards the compression zone of the section) and at least $5d_{bL}$ behind it. These fairly restrictive conditions and the reduction of dowel action resistance with the axial stress level in the bar are behind the large hidden safety factors mentioned above and the exclusion from $A_{sv}$ of those vertical bars in the boundary elements that count as flexural reinforcement.

(2) The contribution of the compression zone, taken to be equal to the minimum of the following:
   - The shear resistance as controlled by diagonal compression over the compression zone, computed as if the latter were a beam of rectangular section with effective depth that of the compression zone, $x$, and thickness that of the web, $b_{wo}$. This calculation employs an inclination $\theta$ of the compression struts equal to 45° and the reduction factor $0.6(1-f_{ck}(\mathrm{MPa})/250)$ on $f_{cd}$ (the factor $\nu$ of clause 6.2.3 in Eurocode 2, or $\eta$ of Section 5.7.5 and equations (D5.31) and (D5.32) above).
   - The frictional resistance, taken to be equal to the friction coefficient $\mu$ multiplied by the normal force on the compression zone. This latter force is taken to be equal to the compression force, $M_{Ed}/z$, delivered to the compression zone by the bending moment from the analysis in the seismic design situation, $M_{Ed}$, plus the share of the compression zone to the total clamping force developing over the cross-section at

imminent sliding, $A_{sv}f_{yd} + N_{Ed}$. Considering this force as uniformly distributed along the length of the wall, $l_w$, the share of the compression zone is equal to its depth, $x$, normalized to $l_w$. The values provided for $\mu$ in Eurocode 2 may be used, with $\mu = 0.6$ – applicable to smooth interfaces – being more appropriate at construction joints and $\mu = 0.7$ – for rough ones – at cracks that may develop during the response in monolithic concrete.

Normally the former term (that due to diagonal compression) governs.

*Clauses 5.5.3.4.4(4), 5.5.3.4.4(5)*

(3) The horizontal components $A_s f_{yd}$ sin $\alpha$ of the yield force – in tension and compression – of bars placed, at an angle $\pm\alpha$ to the vertical and with cross-sectional area $A_s$ per direction, specifically to resist sliding shear. It is recommended that inclined bars are placed so that they cross the base section of the wall at its mid-length, to avoid affecting – through the couple of the vertical components of their tension and compression forces – neither its flexural capacity, $M_{Rdo}$, used for the calculation of the design shear, $V_{Ed}$, according to equations (D5.17) and (D5.18), nor the location of the plastic hinge. A value of the inclination $\alpha = 45°$ is not only convenient but also the most cost-effective, in view of the requirement of *Section 5* that inclined bars extend up to a distance of at least $0.5l_w$ above the base section.

*Clause 5.5.3.4.4(3)*

Inclined bars should normally be placed only if the two other components of the resistance against sliding shear (listed under 1 and 2 above) are not sufficient. However, *Section 5* requires that they are always placed at the base of squat DCH walls – i.e. of those with a height-to-length ratio less than 2 – in a quantity sufficient to resist at least 50% of the design shear there, $V_{Ed}$; moreover, in such walls inclined bars are required at the base of all storeys in a quantity sufficient to resist at least 25% of the storey design shear.

### 5.7.10. Minimum clamping reinforcement across construction joints in walls of DCH

*Clause 5.5.3.4.5(16)*

An additional requirement for DCH walls is to provide across all construction joints clamping reinforcement at a minimum ratio:

$$\rho_{v,\,\min} = \min\left(0.0025;\ \frac{1.3f_{ctd} - N_{Ed}/A_c}{f_{yd} + 1.5\sqrt{f_{cd}f_{yd}}}\right) \tag{D5.50}$$

where $N_{Ed}$ is the minimum axial force from the analysis in the seismic design situation (positive when compressive). Equation (D5.50) is derived from the requirement that the combination of cohesion, friction and dowel action at such a joint is not less than the shear stress that may cause shear cracking at a cross-section nearby. According to Eurocode 2, cohesion and friction provide at a naturally rough, untreated interface between concretes cast at different times a design shear resistance equal to

$$v_{Rdi} = 0.35f_{ctd} + 0.6\left(\frac{N_{Ed}}{A_c} + \rho_v f_{yd}\right) \tag{D5.51}$$

where $f_{ctd} = f_{ctk,\,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c$ is the design value of the tensile strength of concrete and $\rho_v$ the ratio of wall vertical reinforcement providing clamping at the interface. It may be assumed that at the displacements associated with the shear resistance given by equation (D5.51), 50–60% of the design shear resistance due to dowel action may be mobilized as well. As for a single bar of diameter $d_{bL}$, this latter shear resistance is equal to $1.3d_{bL}^2(f_{yd}f_{cd})^{1/2}$, dowel action may be considered to add to the right-hand side of equation (D5.51) the term $0.9\rho_v(f_{yd}f_{cd})^{1/2}$. The so enhanced design shear resistance of the interface should not be less than the shear stress causing concrete cracking, which, for pure shear conditions, $\sigma_I = -\sigma_{II} = \tau$, and the linear biaxial strength envelope for concrete between $\sigma_I = f_{ct}$ and $\sigma_{II} = -f_{ck} \approx -10f_{ct}$, is equal to $\tau_{cr} \approx 0.9f_{ct}$. Taking, for simplicity, $\tau_{cr} = 0.9f_{ctd}$, gives equation (D5.50) for the minimum ratio of clamping reinforcement across construction joints.

## 5.8. Special rules for large walls in structural systems of large lightly reinforced walls

### 5.8.1. Introduction

*Clauses 5.4.2.5, 5.4.3.5*

Eurocode 8 is unique among all regional (as opposed to national) seismic design codes in that it includes special design provisions for structural systems consisting of large walls that cannot be meaningfully designed and detailed for ductile response based on development of a single flexural hinge at the base. Because of this peculiarity, the special dimensioning and detailing provisions given in *Section 5* for the large walls of such systems are described in more detail. They are based on the experience of the application of similar rules in the seismic region of the south of France. They apply only to walls that qualify as large and belong in a structural system of large lightly reinforced walls.

### 5.8.2. Dimensioning for the ULS in bending with axial force

*Clauses 5.4.3.5.1(1), 5.4.3.5.3(3)*

Large walls should be dimensioned for the ULS in flexure without any increase of the design moments above the base over those obtained from the analysis for the seismic design situation. Moreover, the vertical reinforcement placed in the cross-section should be tailored to the requirements of the ULS in flexure with axial force – e.g. without excess reinforcement and with less minimum web vertical reinforcement than required in ductile walls. The objective is to spread flexural yielding at several floor levels and not just at the base of the wall. This will increase the overall lateral deflections of the wall and will mobilize better, through uplift, the contribution to earthquake resistance of masses and transverse beams supported by the wall at intermediate floors. Moreover, the minimization of flexural overstrengths reduces shear force demands and helps in avoiding pre-emptive shear distress.

*Clauses 5.4.3.5.1(2), 5.4.3.5.1(3)*

Due to their small thickness relative to the in-plane dimensions, large walls may be susceptible to out-of-plane instability. *Section 5* requires limiting the magnitude of compression stresses due to bending with axial force, to avoid such out-of-plane instability, without giving detailed guidance for the implementation of this requirement. It opens the door, though, for complementary guidance provided via the National Annex. It refers also to the rules of Eurocode 2 on second-order effects. The rules in Eurocode 2 pertinent to out-of-plane instability are:

- the rules against lateral instability of the laterally unrestrained compression flange of beams (clause 5.9 in EN 1992-1-1)
- the rules for second-order effects in plain (i.e. unreinforced) or lightly reinforced walls (clause 12.6.5 in 1992-1-1).

Deemed-to-satisfy rules in Eurocode 2 against lateral instability of the compression flange of beams include a condition that the product $(h_{st}/b_{wo})(l_w/b_{wo})^{1/3}$ is less than 70, plus another one that $l_w/b_{wo}$ is less than 3.5. This second condition is not meaningful in walls.

The rules for second-order effects in plain or lightly reinforced walls comprise:

- Reduction of the compressive strength of concrete by a factor $\varphi < 1$ equal to $\varphi = \min[1.14(1 - 2e/b_{wo}) - 0.02l_o/b_{wo}, (1 - 2e/b_{wo})]$, where $l_o$ is the unbraced length of the wall and $e$ is the eccentricity of loading in the direction of the thickness of the wall, with a default value of $e = l_o/400$. The unbraced length $l_o$ is taken as equal to the clear storey height, $h_{st}$, divided by $[1 + (h_{st}/3l_w)^2]$ or by $[1 + (h_{st}/l_w)^2]$, if the wall is connected at one or at both ends of its length $l_w$, respectively, to a transverse wall with a length of at least $h_{st}/5$ and thickness of at least $b_{wo}/2$.
- (Only for cast-*in-situ* walls of plain concrete) a lower limit of $l_o/25$ on $b_{wo}$, with $l_o$ being the unbraced length of the wall defined above.

*Clauses 5.4.2.5(3), 5.4.2.5(4), 5.4.2.5(5)*

A characteristic feature of the seismic response of large lightly reinforced walls is their rigid-body rocking with respect to the ground (if they are on footings), or their flexural response as a system of storey-high rigid blocks. This type of response entails hard impact(s) either upon closing of horizontal cracks at floor levels, or of the uplifting footing to the

ground. Such hard impacts excite high-frequency vertical vibrations of the whole of the large wall, or of certain storeys of it. Being of high frequency, these vibrations die out fast and do not have significant global effects. However, they may induce significant fluctuation of the axial force in each individual wall. In view of the inherent uncertainty and the complexity of the local phenomena, *Section 5* allows taking into account this fluctuation in a simplified and safe-sided way, namely by increasing or decreasing the design axial force of each individual wall by half its axial force due to the gravity loads present in the seismic design situation. It also allows neglecting this additional force if the value of the $q$ factor used in the design does not exceed the value $q = 2$. The vertical reinforcement is normally conditioned by the case in which the additional axial force is taken in the ULS verification for flexure with axial load as tensile, whilst a compressive additional axial force is more critical for the concrete and for wall lateral instability.

*Clause 5.4.3.5.1(4)*

Due to the high frequency of these vertical vibrations, the ULS verification for flexure with axial load may be performed with a value of the ultimate strain of concrete increased to $\varepsilon_{cu2} = 0.005$ for unconfined concrete. The beneficial effect of confinement on $\varepsilon_{cu2}$ may be taken into account according to equation (D5.6). If the positive effect of confinement is considered, the unconfined concrete should be neglected if its strain exceeds 0.005. Due to this, and as in thin walls, the (effectively) confined part of the section is normally quite small, taking into account the beneficial effect of confinement on the value of $\varepsilon_{cu2}$ in the confined part of the section will normally not increase the flexural capacity of the wall and is not worth doing.

### 5.8.3. Dimensioning for the ULS in shear

*Clauses 5.4.2.5(1), 5.4.2.5(2)*

To preclude shear failure, each large wall is dimensioned for a shear force, $V_{Ed}$, obtained by multiplying the shear force from the analysis for the design seismic action, $V'_{Ed}$, by a magnification factor $\varepsilon$:

$$\varepsilon = \frac{V_{Ed}}{V'_{Ed}} = \frac{q+1}{2} \tag{D5.52}$$

For the usual value of $q = 3$ applying to systems of large lightly reinforced walls, the value of $\varepsilon$ is equal to 2, and exceeds that given by equation (D5.19) for ductile walls of the same ductility class (M). Moreover, as

- the rules for dimensioning the vertical reinforcement explicitly request minimization of the flexural overstrength, $M_{Rd}/M_{Ed}$
- the period of the fundamental mode in the direction of the length of the wall, $T_1$, is normally not (much) longer than the corner period of the spectrum, $T_C$,

the value of $\varepsilon$ from equation (D5.52) is of the order of that given by equation (D5.18) for slender ductile walls of DCH, and exceeds those given by equation (D5.17) for squat ductile walls of DCH.

*Clause 5.4.3.5.2(1)*

As the magnification factor $\varepsilon$ provides a large margin between the design shear force, $V_{Ed} = \varepsilon V'_{Ed}$, and the value from the analysis, $V'_{Ed}$, and, moreover, the vertical reinforcement is dimensioned for minimum flexural overstrength, it is allowed not to place in large lightly reinforced walls the minimum amount of smeared horizontal reinforcement, if the design shear force, $V_{Ed} = \varepsilon V'_{Ed}$, is less than the design shear resistance of concrete members without shear reinforcement, $V_{Rd,c}$, according to Eurocode 2, given by equation (D5.49). The requirement for horizontal reinforcement is more relaxed than for non-seismic actions because, if inclined cracks form despite fulfilment of the verification $V_{Ed} \le V_{Rd,c}$, their width will not grow uncontrolled as in walls without horizontal reinforcement under force-controlled actions (e.g. wind), but will soon close due to the transient and deformation-controlled nature of the seismic action. Moreover, due to the large horizontal dimension of the wall, $l_w$, any inclined cracks will intersect a floor and mobilize the horizontal ties required to be placed at its intersection with the wall, as well as part of the slab reinforcement in the immediate vicinity of the wall that runs parallel to $l_w$.

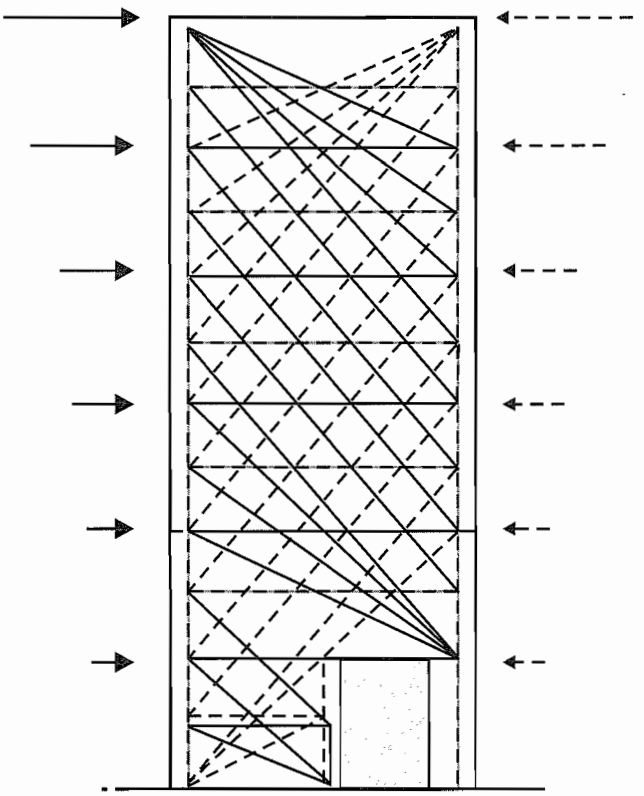


**Fig. 5.11.** Design of a large wall with openings, using the strut-and-tie model

*Clauses 5.4.3.5.2(2), 5.4.3.5.2(3)*

If $V_{Ed} > V_{Rd,c}$, horizontal reinforcement should be calculated according to Eurocode 2, on the basis of either a variable strut inclination model for shear resistance, or a strut-and-tie model, depending on the geometry of the wall. The first type of model is appropriate for walls without openings. Eurocode 2 provides for a strut inclination with respect to the vertical, $\theta$, between 22 and 45° and allows calculating the required horizontal reinforcement on the basis of the minimum value of the shear force within lengths of $z \cot \theta$, where $z$ is the internal lever arm, normally taken equal to $0.8l_w$. Experimental and field evidence suggests that in large walls under lateral loading the struts follow a fan pattern up to a distance $z$ from the base of the wall; from then up, they are at an angle $\theta$ of 45°, intersecting the floors and mobilizing them as ties. The implication for design is that wall horizontal reinforcement should be calculated for $\theta = 45°$, starting with the value of the shear force at $z = 0.8l_w$ from the base and taking into account as part of the shear reinforcement the cross-section of the ties placed at the intersection of the wall with the floors. The floors should be included as ties in any strut-and-tie model due to be used in the presence of significant openings in the wall (see Fig. 5.11). If the geometry of the wall and its openings is not symmetric with respect to the centreline, a different strut-and-tie model should be constructed for each sense of the seismic action parallel to the plane of the wall (positive or negative). Struts should avoid intersecting the openings, and their width should not be chosen to be more than $0.25l_w$ or $4b_{wo}$, whichever is smaller.

If $V_{Ed} > V_{Rd,c}$, and horizontal reinforcement needs to be calculated according to Eurocode 2, then a minimum amount of smear horizontal reinforcement should be placed. For large lightly reinforced walls this minimum amount is a Nationally Determined Parameter with a recommended value equal to the minimum horizontal reinforcement required by Eurocode 2 in walls subjected to non-seismic actions. It should be recalled that, according to Eurocode 2, wall horizontal reinforcement should be placed at a maximum bar spacing of 0.4 m and at a

minimum ratio which is a Nationally Determined Parameter, with a recommended value of 0.1% or of the ratio of web vertical reinforcement, whichever is greater.

*Clause 5.4.3.5.2(4)*

The shear force $V_{Ed}$ computed at construction joints at floor levels from equation (D5.52) should be verified against the design resistance of the interface in sliding, $V_{Rdi}$, taken according to Eurocode 2. This latter is equal to the shear stress given by equation (D5.51), multiplied by $b_{wo}z$. The values of the coefficients 0.35 and 0.6 for cohesion and friction, respectively, apply for a naturally rough free concrete surface without treatment. If the surface is artificially roughened through raking and exposure of aggregates to an average of 3 mm of roughness about every 40 mm, these values may be increased to 0.45 and 0.7, respectively. An additional requirement with respect to Eurocode 2 is that the anchorage length of the clamping bars included in $\rho_v$ should be increased by 50% over the normal value required in Eurocode 2. This does not mean that all vertical bars crossing the interface need to have their anchorage length increased: the requirement applies only to those bars that need to be included in $\rho_v$ so that $V_{Ed} \leq V_{Rdi}$.

### 5.8.4. Detailing of the reinforcement

*Clauses 5.4.3.5.3(1), 5.4.3.5.3(2)*

As stated above, wherever the large wall can resist the design shear force $V_{Ed}$ without horizontal reinforcement, then it can be constructed without such reinforcement. The minimum horizontal reinforcement (at a recommended amount given in Eurocode 2 for walls subjected to non-seismic actions) has to be placed only wherever the wall needs horizontal reinforcement to resist the design shear force. As there is no specific mention of minimum vertical reinforcement in *Section 5*, the pertinent rules of Eurocode 2 apply. These rules call for smeared vertical reinforcement at a bar spacing not more than 0.4 m or three times the web thickness, $b_{wo}$. If this minimum reinforcement suffices for the ULS verification of the section in flexure with axial force, then it should be placed in two layers, one near each face of the wall, both fulfilling the maximum bar spacing requirement. The minimum value of the ratio of the total vertical reinforcement in the cross-section is a Nationally Determined Parameter with a recommended value of 0.002. Both the smeared web reinforcement at the above-mentioned maximum spacing, as well as the vertical bars concentrated near the edges of the cross-section as described below for ULS resistance in flexure with axial force, are included in the total vertical reinforcement that has to meet the minimum ratio requirement.

The vertical reinforcement that is needed, in addition to the minimum smeared reinforcement just described, to provide the ULS resistance in flexure with axial force should be concentrated in boundary elements, one near each of the two far ends of the cross-section (Fig. 5.12). The length, $l_c$, of each boundary element in the direction of the length dimension $l_w$ of the wall should be at least equal to $b_{wo}$ multiplied by the maximum of 1.0 or $3\sigma_{cm}/f_{cd}$, where $\sigma_{cm}$ is the mean value of the concrete stress in the compression zone at the ULS in flexure with axial force and $f_{cd}$ is the design compressive strength of concrete. For the parabola–rectangle $\sigma$–$\varepsilon$ diagram normally used in this ULS verification, the ratio $\sigma_{cm}/f_{cd}$ is equal to $\varphi(1 - \varepsilon_{c2}/3\varepsilon_{cu2})$, with $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.005$ when the additional force due to the vertical vibration of the wall is considered to act as compressive or $\varepsilon_{cu2} = 0.0035$ otherwise and with $\varphi < 1$ denoting the above-mentioned reduction factor for second-order effects in the out-of-plane direction. In the bottom storey of the wall and in any storey where the wall length $l_w$ is reduced with respect to that of the storey below by more than one-third of the storey height, $h_{st}$, the vertical bars in the boundary elements should have a diameter of at least 12 mm. In all other storeys, the minimum diameter of these vertical bars is 10 mm.

All vertical bars should be laterally restrained at the corner of a hoop or by the hook of a cross-tie. The boundary elements at the two ends of the section should be enclosed by hoops

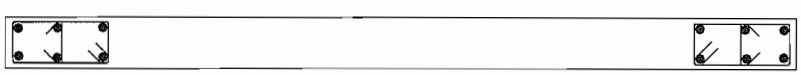


**Fig. 5.12.** Hoops around boundary elements and cross-ties engaging vertical bars in large lightly reinforced walls

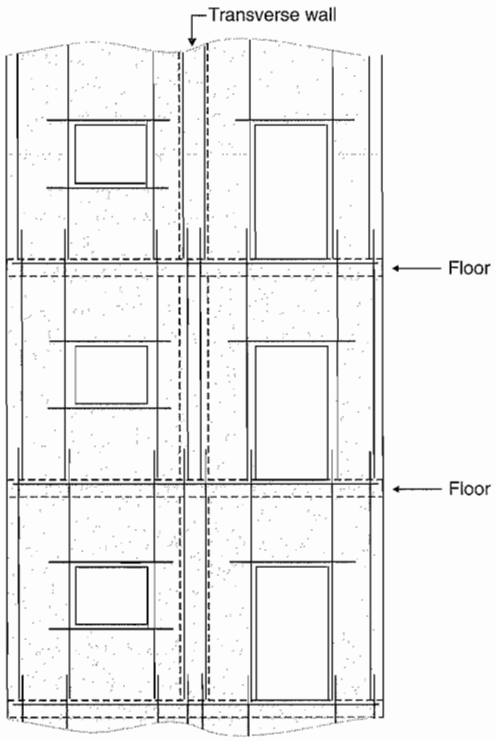



**Fig. 5.13.** Horizontal and vertical steel ties in large lightly reinforced walls with openings

engaging the four corner bars, but intermediate vertical bars in the boundary elements, as well as any vertical bars placed between the two boundary elements to satisfy minimum vertical reinforcement requirements, may simply be engaged by cross-ties across the thickness of the wall (see Fig. 5.12). These hoops and cross-ties should have a minimum diameter of 6 mm or one-third of the vertical bar diameter, $d_{bL}$, whichever is greater, and a maximum spacing in the vertical direction of 100 mm or $8d_{bL}$, whichever is less.

A continuous horizontal steel tie is required along each intersection of a large wall with a floor. This tie should extend into the floor beyond the ends of the wall, to a sufficient length not only for anchorage of the tie, but also for the collection of inertia forces from the floor diaphragm and their transfer to the wall. Vertical steel ties are also required at all intersections of a large wall with transverse walls or with wall flanges, as well as along the vertical edges of openings in the wall. These vertical ties should be made continuous from storey to storey through the floor, by means of appropriate lapping. When openings are not staggered at different storeys but have the same horizontal size and location, vertical steel ties along their edges should also be made continuous through lap splicing (Fig. 5.13). Horizontal ties should also be placed at the level of the lintels above openings, but they do not need to be continuous from one opening to the other. Specific rules for the dimensions and the capacity of the ties are not given, but reference to the clauses of Eurocode 2 is made. Countries may include in the National Annex reference to complementary sources of information for these ties.

*Clause 5.4.3.5.3(4)*

## 5.9. Special rules for concrete systems with masonry or concrete infills

*Section 4* of EN 1998-1 contains special rules for the analysis and design of frame (or frame-equivalent) concrete buildings and of unbraced steel or composite buildings with non-engineered masonry infills (see Section 4.12 of this guide). These rules are mandatory

only for buildings designed for DCH. If the building is designed for DCM or DCL, the rules of *Section 4* are considered to serve only as a guide to good practice.

*Section 5* contains additional rules for concrete buildings with infills, which apply to buildings designed for either DCH or DCM, (but not for DCL), irrespective of the structural system. The objective of these rules is to protect concrete buildings from the adverse local effects of infills.

The potential adverse local effects of infills are mainly from two sources:

- damage or even failure of columns in contact with strong infills over their full height, due to non-uniform and/or unbalanced contact conditions, or
- a reduction in the clear height (and hence in the effective shear span) of columns due to contact with (and restraint by) infills over part of the full height; the resulting 'short' or 'captive' column is prone to flexural/shear failure or a pure shear failure dominated by diagonal compression.

*Clause 5.9(1)* Part of an infill panel may be dislocated by failure or heavy damage, exerting a concentrated force on the adjacent column. The stronger the infill, the larger the magnitude of this force and the higher the likelihood of local column failure. Infill panels are more likely to fail or suffer heavy damage at the ground storey, as there the shear force demand is largest. For this reason, in buildings with masonry or concrete infills, the entire length of the columns of the ground storey is considered a critical region and subject to the corresponding special detailing and confinement requirements, to be prepared for local overloading by the failed infill panel at any point along its height.

*Clause 5.9(3)* Unbalanced contact conditions may take place in columns with a masonry infill on only one side (e.g. corner columns). The entire length of such columns is considered a critical region and subject to the associated special detailing and confinement requirements.

*Clause 5.9(2)* The lateral restraint of a column due to the contact with the infill over part of its full height is normally sufficient to cause the plastic hinge to develop in the column at the elevation where the infill is terminated, instead of the end of the column beyond the contact with the infill. This may be the case even when at this latter end the beams are weaker than the column and equation (D4.23) is satisfied. So, *Section 5* requires calculation of the design shear force of the 'short' or 'captive' column through equation (D5.12), with:

(1) the clear length of the column, $l_{cl}$, taken equal to the length of the column not in contact with the infills
(2) the term min(...) taken equal to 1.0, at the column section at the termination of the contact with the infill wall.

Moreover, as (1) the clear length of the column may be short and (2) the exact location and extent of the potential plastic hinge region near the termination of the contact with the infill wall is not clear and may well extend into the region of the column in contact with the infill, it is a requirement to:

- place the transverse reinforcement necessary to resist the design shear force not just along the clear length of the column, $l_{cl}$, but also along a length into the column part in contact with the infills equal to the column depth, $h_c$, within the plane of the infill
- consider the entire length of the column as a critical region and provide it with the amount and pattern of stirrups specified for critical regions.

This additional transverse reinforcement will increase the nominal shear resistance of the 'captive' column over its full length, beyond the design shear force for which it has been verified, and will enhance its deformation capacity for any potential location of the plastic hinging. This may partly compensate for the lack of a special rule in Eurocode 8 for the calculation of the nominal shear resistance of columns with a low shear span ratio ('squat columns'), regardless of their reduced cyclic shear resistance as controlled by failure of the concrete along the diagonal(s) of the column in elevation. In fact, cyclic test data from 44 columns with shear span ratio, $L_s/h_c$, less than or equal to 2 that have failed by shear

compression suggest the following expression for their shear resistance as controlled by failure of the concrete (units: meganewtons and metres):

$$V_{R,\max} = \frac{4}{7}(1-0.02\min(5;\mu_\theta^{pl}))\left(1+1.35\frac{N}{A_c f_c}\right)[1+0.45(100\rho_{tot})]\times \sqrt{\min(f_c,40)}\,b_w z\sin 2\vartheta \quad \text{(D5.53)}$$

Equation (D5.53) is the counterpart of equation (D5.47) for squat columns; all variables in it are defined as in equation (D5.47), except (1) the internal lever arm $z$, which is taken here to be equal to $z = d–d'$ and (2) $\theta$ in the last term, which is the angle between the axis of the column and its diagonal in elevation ($\tan\theta = h_c/2L_s$). Equation (D5.53), proposed by Biskinis *et al.*,[69] has been adopted in Annex A of EN 1998-3[52].

If the clear length of the column, $l_{cl}$, as specified in point 1 above, is short, then the design shear force may be so large that it may be difficult to verify the column for it, especially as the critical shear resistance may be controlled by shear compression (cf. equation (D5.53)) and cannot be increased through transverse reinforcement. Although designation of such a column as 'secondary seismic' (cf. Section 4.10) may seem a convenient way out of this predicament, it is far more sensible to attempt to solve the problem through a change of the geometric conditions by either:

(1) changing the configuration of the infills and their openings to remove the partial-height contact of the column with the infill or increase the clear length of the column, $l_{cl}$, beyond this contact or
(2) changing the cross-sectional dimensions of the column.

Option 2 should be exercised to reduce the size of the column, rather than increasing it:

- if the shear span ratio, $L_s/h_c$, of the column increases above 2 (or, preferably, 2.5) its behaviour in cyclic shear will not exhibit the special vulnerability and low dissipation capacity which characterizes short columns
- the decrease in the cross-sectional dimensions will reduce the design shear force from equation (D5.12) (by reducing the design values of the flexural resistance of the column, $M_{Rdc,\,i}$, $i = 1, 2$) more than it will reduce the nominal shear resistance, helping both the verification as well as the physical problem.

Reinforcement placed along both diagonals of the clear length of the short column within the plane of the infill is very effective in increasing its energy dissipation and deformation capacity. Placement of such reinforcement, in addition to or instead of the conventional transverse reinforcement of the column, is another viable option. This reinforcement may be dimensioned to resist at the same time the design shear force from equation (D5.12) as well as the design bending moments at the end sections of the short column, in accordance with the relevant rules for coupling beams in coupled walls. Placement of such reinforcement and its dimensioning to resist the full value of the design shear force is mandatory, if the clear length of the column, $l_{cl}$, is less than $1.5h_c$ (corresponding to a value of the shear span ratio, $L_s/h_c$, less than 0.75).

To prevent shear failure of columns with a masonry infill on only one side, a length, $l_c$, at the top and the bottom of the column over which the diagonal strut force of the infill may be applied, should be verified in shear for the smaller of the following two design shear forces: *Clause 5.9(4)*

(1) the horizontal component of the strut force of the infill, taken as equal to the horizontal shear strength of the panel, as estimated on the basis of the shear strength of bed joints (shear strength of bed joints multiplied by the horizontal cross-sectional area of the panel, $b_w$, multiplied by the clear panel length $L_{bn}$)
(2) the shear force computed from equation (D5.12), applied with clear length of the column, $l_{cl}$, taken as equal to the contact length, $l_c$, and the parentheses in the numerator equal to twice the design value of the column flexural capacity, $2M_{Rd,\,c}$.

In case 2 the contact length should be taken as equal to the full vertical width of the diagonal strut of the infill. This is consistent with the calculation in case 1, which conservatively assumes that the full strut force is applied to the column. It is also closer to the reality at the top of the column, as there the joint between the top of the infill and the soffit of the beam may be open due to creep of the masonry or concrete infill.

## 5.10. Design and detailing of foundation elements

*Clause 5.8.1(1)* Foundation elements are normally made of concrete, even when the superstructure may consist of another structural material. *Section 5* gives the design and detailing rules which apply to concrete foundation elements (footings, tie beams, foundation beams, foundation slabs and walls, piles and pile caps) even when the vertical elements founded through them are made of a different material. *Section 5* also gives rules for the connection of concrete foundation elements to the vertical ones of the superstructure, applying only when the latter are also made of concrete.

*Clauses 5.8.1(2), 5.8.1(4)* Concrete foundation elements which are dimensioned for seismic action effects derived from either:

(1) the analysis for the design seismic action using a $q$ factor less than or equal to the value of $q$ for low dissipative behaviour (1.5 in concrete buildings, up to 2.0 in steel or composite buildings) according to *clause 4.4.2.6(3)* of EN 1998-1 or
(2) capacity design calculations according to *clauses 4.4.2.6(2)* and *4.4.2.6(4)–4.4.2.6(8)* of EN 1998-1

are allowed to follow the simpler dimensioning and detailing rules applying to DCL (i.e. those of Eurocode 2 alone, plus the requirement to use steel of at least Class B), irrespective of the ductility class for which the superstructure is designed. The reason is that they are expected to remain elastic under the design seismic action (even when this is just due to the overstrength inherent in the $q$ factor value for low dissipative behaviour in case 1 above).

*Clause 5.8.1(3)* Although choices 1 and 2 above are the only ones allowed for the verification of the foundation, *Section 5* allows designing concrete foundation elements for energy dissipation, as in the superstructure. In that case they may be dimensioned for seismic action effects derived from the analysis for the design seismic action using the $q$ factor chosen for the superstructure. They should also meet all the special dimensioning and detailing rules pertaining to the corresponding ductility class and applying to elements of the superstructure. This provision refers in particular to tie beams and to foundation beams, which should then be dimensioned in shear for a shear force derived from capacity design calculations, and should follow all the special rules for detailing of the longitudinal and transverse steel that aim at enhanced local ductility.

*Clause 5.8.1(5)* The best foundation system of a building from the point of view of earthquake resistance is commonly considered to be a box-type configuration consisting of:

(1) Wall-like deep foundation beams along the entire perimeter of the foundation, possibly supplemented by interior ones across the full length of the foundation system. These beams are the main foundation elements that transfer the seismic action effects to the ground. In dissipative buildings they are designed according to *clause 4.4.2.6(8)* as common foundation elements of more than one vertical member, normally by multiplying the design seismic action and its effects from the analysis by a factor of 1.4. In buildings with a basement, the foundation beams on the perimeter may also serve as basement walls.
(2) A concrete slab acting as a rigid diaphragm, at the level of the top flange of the foundation beams of the perimeter (as the roof of the basement, if there is a basement).
(3) A foundation slab or a grillage of tie beams or foundation beams, at the level of the bottom of the perimeter foundation beams.

Owing to its high rigidity and strength, such a system works as a rigid body. Thus, it minimizes uncertainties regarding the distribution of seismic action effects at the interface between the ground and the foundation system and ensures that all vertical elements undergo the same rotation at the level of their connection with this system, so that they may be considered as fixed against rotation at that level. Moreover, it ensures that the base of the superstructure is subjected to the same ground motion, smoothing out any differences in the motion over the foundation and filtering out any high-frequency components of the input.

Due to the high rigidity and strength of a box-type foundation system, that part of the columns within its height, as well as all beams within the foundation system (including those at the roof of the basement), are expected to remain elastic in the seismic design situation and hence may follow the simpler dimensioning and detailing rules applying to DCL (i.e. those of Eurocode 2 alone, plus the requirement to use steel of at least Class B), irrespective of the ductility class for which the building is designed.

Plastic hinges in walls and columns will develop at the top of a box-type foundation system (at the level of the basement roof slab). If the cross-section of a wall is the same above and below that level (as in interior walls that continue down to the level of the foundation system), that part of the height of the wall below the top the foundation system should be dimensioned and detailed according to the special rules of wall critical regions down to a depth below that level equal to the height of the critical region, $h_{cr}$, above that level. Moreover, as fixity of the wall at the level of the top of the foundation system is achieved via a couple of horizontal forces that develop at the levels of the top and bottom of the foundation system, the full free height of such walls within the basement should be dimensioned in shear assuming that the wall develops at the level of the top of the foundation system (basement roof) its flexural overstrength $\gamma_{Rd}M_{Rd}$ (with $\gamma_{Rd} = 1.1$ in buildings of ductility class M and $\gamma_{Rd} = 1.2$ in those of DCH) and (nearly) zero moment at the foundation level.

*Clause 5.8.2*

The soffit of tie beams or foundation slabs connecting different footings or pile caps should be below the top of these foundation elements, to avoid creating a short column there, which is inherently vulnerable to shear failure.

Tie beams between footings and tie zones in foundation slabs should be dimensioned for the ULS in shear and in bending for the action effects determined from the analysis for the seismic design situation or via capacity design calculations, and to a simultaneously acting axial force (tensile or compressive, whichever is more unfavourable) equal to a fraction of the mean value of the design axial forces of the connected vertical elements in the seismic design situation. This fraction is specified as equal to the design ground acceleration in $g$, $\alpha S$, multiplied by 0.3, 0.4 or 0.6 for ground type B, C or D, respectively. The purpose of the additional axial force is to cover the effects of horizontal relative displacements between foundation elements not accounted for explicitly in the analysis for the seismic design situation. It may be neglected for ground type A, as well as in low-seismicity cases (recommended as those with $\alpha S \leq 0.1$) over ground type B.

The minimum cross-sectional dimensions and the minimum longitudinal reinforcement ratio of tie beams or foundation beams and of tie zones in foundation slabs used instead of tie beams are Nationally Determined Parameters. If tie beams are designed for energy dissipation (i.e. if they are dimensioned for the ULS in bending and in shear for seismic action effects derived from the analysis using a $q$ factor value higher than that corresponding to low-dissipative structures), then they should meet also the minimum reinforcement requirements of the corresponding ductility class.

*Clauses 5.8.3(1), 5.8.3(4)*

The connection of a foundation beam or a foundation wall with a concrete column or wall is essentially an inverted-T or knee 'beam–column joint'. Therefore, it should be dimensioned and detailed according to the rules for beam–column joints of the corresponding ductility class. This implies that the transverse reinforcement placed in the critical region at the base of the column or the wall should also be placed within the region of its connection with the foundation beam or wall, except for interior columns founded at the intersection of two foundation beams with width at least 75% of the corresponding dimension of the column. In that case the horizontal reinforcement is placed in the connection at a spacing which may be

double that at the column base, but not more than 150 mm. It is noteworthy that the horizontal reinforcement at the connection of a concrete wall with a foundation beam or wall is also specified through reference to the transverse reinforcement in the critical regions of DCM columns. However, as the rules are essentially the same as those for the transverse reinforcement in boundary elements within the critical region of ductile walls, the horizontal reinforcement to be placed in the connection of a wall and a foundation beam (or wall) should have the same diameter and spacing as the peripheral ties of the boundary elements of the wall critical region above, but it should extend over the entire periphery of the horizontal section of the connection region.

*Clauses 5.8.3(2), 5.8.3(3)*

In addition to being subject to the prescriptive detailing of the previous paragraph, in buildings of DCH the connection region of a foundation beam or wall with a concrete column or wall should be explicitly verified in shear. The design horizontal shear force to be used in this verification, $V_{jhd}$, should be established as follows:

- If the foundation beam is dimensioned on the basis of seismic action effects derived from capacity design considerations (i.e. in practice for the seismic action effects from the analysis for the design seismic action multiplied by 1.4), then $V_{jhd}$ may be determined from the analysis for the design seismic action. Because this analysis does not directly provide seismic action effects for the joints, $V_{jhd}$ may conservatively be estimated as the design value of the flexural capacity at the base section of the column or wall, $M_{Rd}$, divided by the depth of the foundation beam, $h_b$.
- If the foundation beam is dimensioned on the basis of seismic action effects derived directly from the analysis for the design seismic action, then $V_{jhd}$ itself should be determined via capacity design calculations, namely through equation (D5.21), using as $A_{sb1}$ and $A_{sb2}$ the areas of the top and bottom reinforcement in the foundation beam, respectively. This approach is never unconservative (on the unsafe side) for the connection region.

CHAPTER 6

# Design and detailing rules for steel buildings

## 6.1. Scope

This chapter covers the design of steel buildings for earthquake resistance according to the provisions of *Section 6* of EN 1998-1. It summarizes the important points of *Section 6* without repeating them, and provides comments and explanations for their application, as well as background information. *Clause 6.1.1*

The scope of *Section 6* covers buildings made of steel frames. However, most of the design conditions defined in *Section 6* for materials, connections, types of structures, control, etc., apply also to *Section 7* on composite steel concrete structures. Those clauses of *Section 7* relevant to the various sections of this chapter are indicated in the margin.

## 6.2. Dissipative versus low-dissipative structures

Steel buildings can be designed to be 'energy-dissipative' to a larger or smaller extent. As also explained in Section 2.2.2.1 of this guide, this term refers to the ability of some intentionally selected parts of the structure to safely undergo cyclic plastic deformations, and applies to buildings designed such that the selected zones – and only those – are indeed activated plastically. Then, the global behaviour of the building under seismic loads is characterized by a load–displacement curve with a significant 'yield plateau' – curve b in the base shear $V$–displacement $d$ diagram of Fig. 6.1 – rather than by a 'brittle' behaviour – curve a in Fig. 6.1. This ductility aspect of behaviour, which is related to providing a deformation capacity and not only strength, is less prevalent in the design of structures for non-seismic actions. *Clauses 6.1.2(1), 6.1.2(2), 6.1.2(4), 6.1.2(5), 6.1.2(6)*

In what follows, the term 'brittle', intentionally enclosed within quotation marks, has the meaning 'with little or no deformation capacity after reaching the maximum strength'. The meaning is not restricted to the classical meaning of a brittle phenomenon in steel structures (inability to prevent a crack from propagating), which is just one possibility of being 'brittle'.

As already detailed in Section 2.2.2.1 of this guide, structures designed to concept b, i.e. for ductile behaviour, are given a design premium in the sense that they are allowed to be less stiff and less resistant than those designed to concept a. This is achieved by a reduction in the elastic earthquake lateral forces by a factor $q$ ($> 1$) which is higher for dissipative structures (up to $q = 6.5$ in steel moment frames). The value of $q$ is different for different structural types, depending on their ability to dissipate energy. Only one structural type is mentioned in *Section 6* as unable to be dissipative: frames with K bracings, as explained in Section 6.8.

Regarding connections, these can be of either the full-strength or the partial-strength type, and certain conditions need to be met for both options.

There is a price to pay to be allowed to consider reduced design seismic forces in a building of reduced strength: the structural elements and their connections have to comply with all the Eurocode 8 requirements.

Designing for a reduced earthquake action should in principle generate a more economic structure. This seemingly straightforward conclusion may be wrong in some cases, because a design does not have to comply with the seismic requirements only but with all design requirements, such as the limitation of floor vertical deflection under gravity loading, interstorey drift limits and resistance to wind. As the design checks corresponding to the ultimate limit state (ULS) resistance under the design earthquake are not necessarily the most controlling ones, the complete design process may generate a structure with more strength than strictly needed for the resistance to the design earthquake. Then, some specific Eurocode 8 requirements for ductility, such as the 'overstrength design' of connections relatively to the strength of connected members, or the 'overstrength design' of columns relatively to beams, apply to structural components stronger than strictly necessary for the resistance to earthquakes, leading to a general over-design of the structure. In that case, the option of designing a dissipative structure may not be economic. This is often the case:

- in low seismicity areas
- when a structural system is used that lends itself more for resistance than for ductility, e.g. systems using thin-walled profiles or using partial strength connections
- for flexible structures, in which the serviceability limit states (SLSs) are the controlling ones.

Designing a dissipative structure is certainly uneconomic if the wind resultant force is greater than the earthquake resultant force obtained with the minimum code value of the $q$ factor ($q = 1.5$). Whether this is the case can be approximately assessed as follows at the pre-project stage. The maximum earthquake horizontal resultant force is estimated as $F_b = mS_d(T)\lambda$ (cf. *equation (4.5)* and equation (D4.5) in Section 4.5.2.3). Without knowledge

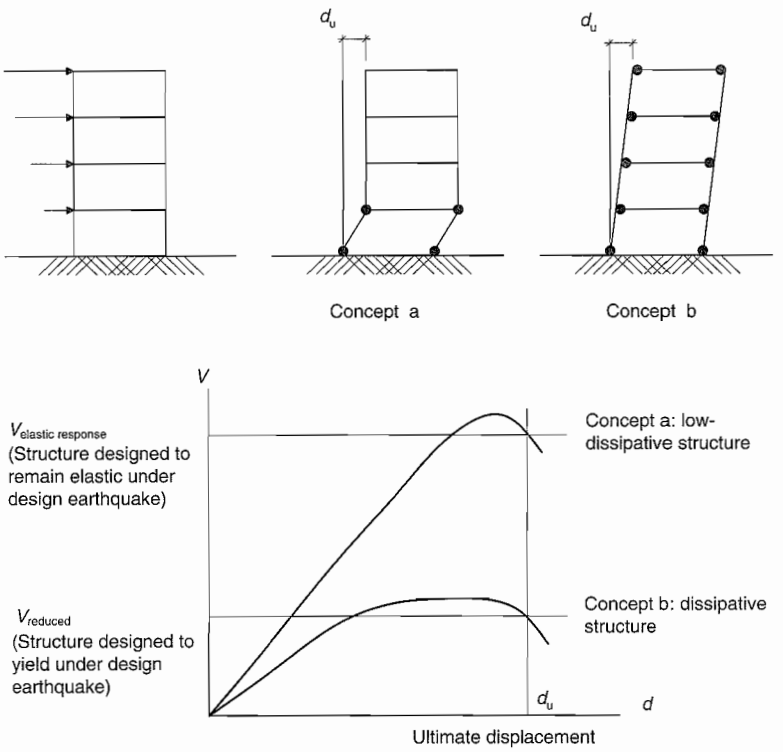



**Fig. 6.1.** Definition of design concepts **a** and **b**

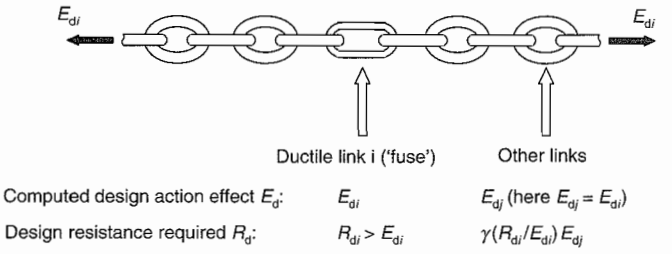



**Fig. 6.2.** Principle of capacity design

of the structure period and of the soil, a safe-side approach uses the maximum spectral ordinate $2.5a_gSm/q$ (*clause 3.2.2.5*) for the most adverse conditions, $S = 1.5, q = 1.5$ and $\lambda = 1$, so that $F_b = 2.5a_gm$ is an estimate of the earthquake horizontal resultant force $F_b$ to be compared with the wind resultant force $F_w$. If $F_b < F_w$, designing for dissipative structural behaviour is uneconomic.

## 6.3. Capacity design principle

The awareness of favourable and unfavourable factors for energy dissipation, which are listed in Section 6.4 below, allows the design of reliable dissipative zones. It remains to ensure that energy dissipation takes place in dissipative zones and not in 'brittle' ones. This is achieved through capacity design. *Clause 6.2(2)*

In the capacity design concept, all 'brittle' structural elements or components are protected against failure, by providing them with a strength greater than that corresponding to the development of the maximum feasible strength in the plastic regions.

The following features characterize the procedure:

- potential plastic regions within the structure are clearly defined and designed to have dependable strengths
- potentially 'brittle' regions or those components not suited for stable energy dissipation are protected, by ensuring that their strength exceeds the demands originating from the plastic regions.

To highlight the concept of capacity design, the chain shown in Fig. 6.2 is often considered.[68] As the strength of a chain is the strength of its weakest link, one ductile link may be used to achieve ductility for the entire chain. The nominal tensile strength of the ductile link is subject to uncertainties of material strength and strain hardening effects at high strains. The other links are presumed to be 'brittle', but their failure can be prevented if their strength is in excess of the real strength of the ductile weak link at the level of ductility envisaged.

As detailed below, sound application of the capacity design principle thus requires knowledge of the material properties, in particular of the yield stress, both of the plastic zones and of the neighbouring ones.

Capacity design develops a hierarchy of strength. To be effective, the real strength of the 'brittle' parts and of the ductile parts must be under control. For the 'brittle' parts, the standard nominal yield stress $f_y$ is a lower bound which guarantees that those parts have at least the design strength needed to remain elastic. For ductile zones, however, the yield strength of the material must be limited by an upper bound value, in addition to the standard minimum value $f_y$ corresponding to the grade of steel. This raises practical problems, because the definition of an upper bound yield stress is not a usual practice for steel products and also because of a lack of statistical data on that subject. Eurocode 8 tackles this problem considering different possible circumstances.

Case (a) in *clause 6.2(3)* refers to the standard situation, in which the real yield stress of the material of dissipative zones is not known at the design stage. Then, the upper bound *Clause 6.2(3)*

yield stress is estimated as $\gamma_{ov} f_y$; $\gamma_{ov}$ is a coefficient based on statistics of yield stresses characterizing steel products, which may vary from one steel plant to another. For European rolled sections, the estimate is $\gamma_{ov} = 1.25$, but the designer may choose a different value, larger or smaller. Then, the upper bound of the actual yield stress of the material provided at the construction stage for dissipative zones should not be greater than $f_{y,\,max} = 1.1\gamma_{ov} f_y$, in which 1.1 provides an additional margin.

If the actual yield strengths of the materials to be used in the construction are accessible at the design stage, then a real value of $\gamma_{ov}$ can be used as $\gamma_{ov,\,act} = f_{y,\,ac}/f_y$, as stated in case (c) of *clause 6.2(3)*.

Case (b) in *clause 6.2(3)* refers to a situation in which steel producers would provide the market with a 'seismic' steel grade for which both a nominal (lower bound) value and an upper bound value $f_{y,\,max}$ are defined. If the design of all sections is made considering only that one 'seismic' steel grade and if the steel for the non-dissipative parts is specified to belong to a higher grade than the 'seismic' one, then the general hierarchy criterion is automatically fulfilled. This would, for example, be the case if an S235 steel with $f_{y,\,max} = 355$ MPa is a 'seismic' grade used for dissipative zones, with S355 steel used for non-dissipative parts or members. In such a case, there is no need for $\gamma_{ov}$, and its value can be set equal to 1.

Sound application of the capacity design principle requires, in addition to knowledge of the material properties, notably the yield stress, of both the plastic and neighbouring zones, a correct evaluation of the stresses and strains sustained by the various components of the plastic zones: steel profiles, welds, bolts and plates. In or near connections, the real distribution of stresses and strains are very different from what they are in beams, columns, etc., and can only be defined by sophisticated numerical or experimental studies. There are several reasons for this: plane sections do not remain plane, the presence of stress concentrations, etc. Without such sophisticated studies, safety is based on simple estimates by means of the 'overstrength design factor', equal to 1.1 in the relationship $f_{y,\,max} = 1.1\gamma_{ov} f_y$

Sound application of the capacity design principle also requires adequate materials for the plastic zones, where 'adequate' refers to the required properties: elongation, $f_u/f_y$, toughness and weldability. It also requires good design of the plastic zones, avoiding in particular a localization of strains. This requirement is explained in Section 6.7.

## 6.4. Design for local energy dissipation in the elements and their connections

### 6.4.1. Favourable factors for local ductility

*Clauses 6.2(7), 6.2(8), 6.2(9), 6.5.2(1), 6.5.2(2), 6.5.2(3), 6.5.4(1), 6.5.5(4)*

Steel is a ductile material, if a correct steel grade is selected: a material elongation over 20% and a material ductility ratio $\varepsilon_{y,\,max}/\varepsilon_y$ over 10 can provide highly ductile dissipative zones. If the designer makes good choices in the design, the plastic mechanism developed in a structural component, such as a beam or a diagonal bar in a truss, can be fairly ductile and dissipative.

Reliable energy dissipation at the element scale can be found in:

(1) Bars yielding in tension. This possibility directly relies on the use of an adequate material, in a design avoiding stress concentrations or excessive section reduction (see Section 6.7), in which only tension develops. For these reasons, high-strength bolts in tension should not be used as dissipative components, because they are not made of a very ductile material and may be subjected to effects other than pure tension, such as being bent when a connection deforms. The cyclic plastic strains involved can result in early low cycle fatigue failure.

*Clause 6.7.2(3)*

(2) Bars yielding in compression, if early buckling is prevented. Yielding in compression inevitably ends in buckling. However, stocky elements with $\bar{\lambda} < 0.2$ can keep their strength and develop some plasticity in compression. Recently, designs in which the

compressed bar is inserted in a tube only intended to provide lateral support against buckling have been tested and used successfully.[71]

(3) Plates yielding in bending.

(4) Profiles yielding in bending, if flange buckling takes place at sufficiently large enough deformation (see *Table 6.3* of EN 1998-1). Buckling of flanges cannot be prevented totally, but is delayed when the slenderness ratio $c/t$ of walls of sections is small. Eurocode 8 uses Classes 1, 2, and 3 of sections of Eurocode 3 to characterize the ability of sections to develop a 'plastic hinge', meaning their ability to provide a stable full plastic moment resistance when cyclic plastic rotations are applied. The required ability is linked to the demand for plastic rotation. For instance, sections belonging to Class 1 are able to undergo cyclic plastic rotations over 35 mrad without losing significant strength. *Clauses 6.5.3(1), 6.5.3(2)*

(5) Plate yielding in shear. Yielding in shear is a ductile and stable mechanism. The plastic strength in shear is related to the slenderness of the plate; its reduction with slenderness has to be considered. *Clause 6.6.3(7)*

Some ductile local phenomena can also contribute to the ductility of connections: *Clause 6.5.5(5)*

(1) Ovalization of bolt holes takes place in plates of connections which are made of a ductile constructional steel, in contrast to the failure of bolts in shear or of welds. Even in a connection which is capacity ('overstrength') designed to the assembled bars, which in principle need not fulfil any further design condition, the designer can obtain more assurance of ductile behaviour by designing one or both assembled plates so that their bearing resistance is less than the bolt shear resistance. This possibility is recommended for the following reason: even if the bolted connection has been designed to be 'non-slip', experiments have shown that there is always a relative movement between two assembled plates, once the connected elements are subjected to plastic cycles. In practice, this means that after few cycles the bearing resistance becomes the true mode of resistance of the bolted connection. This justifies the requirement to check the bearing resistance and to have it as 'weak link' in the chain of resistances.

(2) Friction between plates: connections made with pre-tensioned bolts working in shear are a site of friction, once relative movement between assembled plates takes place. As friction dissipates energy, which is favourable, and also to avoid destructive shocks in bolts between loose parts of connections, pre-tensioning of bolts is prescribed. These two positive influences are not quantified in the energy dissipation of the structure. Category B bolted joints in shear (slip-resistant in the SLS but not in the ULS) and surface preparation class B (alkali–zinc painting applied to a prepared surface) are permitted by *clause 6.5.5(4)*. In practice, this means that slippage is allowed under seismic conditions, because it is a ULS situation. *Clauses 6.5.5(4), 6.5.5(5)*

(3) Reliable energy dissipation can take place also in the joints, rather than in the members themselves, if the joints are designed to develop one or several of the dissipative mechanisms listed above. This concept raises the problem of knowledge of the cyclic behaviour of the components of a joint, in order to select the ductile joints (e.g. plates in bending) and to capacity design the 'brittle' ones (e.g. bolts in tension). Section 1.8 of EN 1993-1-8 on the design of joints may serve as a basis for ascertaining this information, though the cyclic behaviour aspect is not covered. As the reality of connections behaviour may be complex, this design possibility will widen when the results of ongoing and future studies become available. *Clause 6.5.2(5)*

### 6.4.2. Unfavourable factors for local ductility

If one of the following unfavourable circumstances is realized, local ductility will be small.

Little energy dissipation is expected: *Clauses 6.2(1), 6.2(7), 6.2(8), 6.5.5(1)*

(1) In zones made totally or in part of brittle or low-ductility material. This rule first concerns the steel of sections and plates, which should comply with requirements on steel grade, toughness and weldability. It also concerns the weld material, the adjacent

heat-affected zone, the welding process and the quality of execution of welds, all of which may affect negatively the base material.

(2) If plastic strains take place in a too narrow zone; this is a 'localization of strains' or 'stress concentration' situation. Even if all conditions related to materials and execution are correct, the design of elements and, in particular, of connections can be such that plastic strains take place in a short zone. A high elongation in a short zone can correspond to a very low deformability of a component, which can be significantly below the expectations of the designer and the requirements of the code. Bad design generates this outcome. This issue is discussed further in Section 6.7.

*Clause 6.5.3(1), 6.5.3(2)*

(3) If early local or global buckling affects the element or local zone. As recalled above, rules define limits of global and local slenderness to control these effects. *Clause 6.5.3(1)* and *Table 6.3* in EN 1998-1 aim at prevention of local buckling. Global buckling checks are those of Eurocode 3, but several relationships in Eurocode 8 aim at a safe-side definition of design action effects in columns. This is the case in *clauses 6.6.3(1)* and *6.6.3(2)* for columns in moment frames and *clause 6.7.4(1)* for beams in moment frames.

## 6.5. Design rules aiming at the realization of dissipative zones

*Clause 6.2*

The following clauses of Eurocode 8 aim at creating conditions for an effective local dissipation of energy based on the principle of capacity design:

- *Clause 6.2* defines conditions on material properties such that the yield stress of the different components is under control and the ductile 'weak link' is really a weak link in the chain of resistances.

*Clause 6.5.5(3)*

- For design in which dissipative zones are intended in the structural elements and not in the connections, a factor of 'overstrength' design for connections equal to $1.1\gamma_{ov}$ is specified in *clause 6.5.5(2)*.

*Clause 6.5.2(5)*

- Accordingly, if the designer plans to achieve energy dissipation in the joints, overstrength of the connected bars is needed and imposed by *clause 6.5.2(5)*. This is necessary because in that design option, the bars of the structure, which are not presumed to be the site of energy dissipation, are not required to fulfil specific conditions for their ductility in the plastic stage, such as belonging to certain classes of sections.

*Clauses 6.1.3(2), 6.5.2, 6.6.1, 6.6.3(1), 6.6.3(2), 6.7.4(1), 6.8.3(1), 6.8.4(1)*

Several rules in the sections of EN 1998-1 on steel and composite steel–concrete structures aim at a correct capacity design of structural elements which are **not** intended to be dissipative. Their rationale is always to enforce a hierarchy between the dissipative and non-dissipative structural elements. For this reason they may be called **hierarchy criteria**. This type of logic is explained here with details.

The capacity design of structural elements which are not intended to be dissipative takes place in the following manner:

(1) The action effect in the non-dissipative element, e.g. the axial force $N_{Ed,E}$ in a column, is computed by means of an elastic analysis of the structure under the design seismic action (i.e. for the design spectrum, obtained from the elastic spectrum reduced by the $q$ factor). The element considered (e.g. a column) is not the one in which the dissipative zone develops; let us assume that it is a plastic hinge in a beam.

(2) The action effect in the dissipative element, e.g. the bending moment $M_{Ed,E}$ in that beam, is computed by means of the same elastic analysis of the structure under the design seismic action.

(3) The section selected for the beam may be the location for two different causes of excess strength with respect to the minimum strength:

- A section overstrength $\Omega$, if there is no section where strictly $M_{pl,Rd} = M_{Ed}$. Then, $\Omega = M_{pl,Rd}/M_{Ed}$.

- A material overstrength, due to the fact that real materials can provide yield stresses higher than nominal and that the material overstrength in the dissipative element can be greater than in the non-dissipative element. This is accounted for by the factor $\gamma_{ov}$ above.

(4) The capacity design of the column, such that the plastic hinge forms in the beam while the column remain elastic, has to consider the two mentioned sources of overstrength. The design seismic axial force is thus $1.1\gamma_{ov}\Omega N_{Ed,E}$. In that expression, 1.1 is the safety factor explained in Section 6.3 above.

(5) Depending on the type of structure or of structural element, the seismic axial design force is combined with gravity effects:

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \quad \text{(D6.1)}$$

(6) The design check under axial force in the column is

$$N_{pl,Rd} \geq N_{Ed} \quad \text{(D6.2)}$$

Many expressions in Eurocode 8 correspond to this logic of taking, as amplification factors in the evaluation of design forces of non-dissipative elements, the overstrength of the dissipative elements:

- *clause 4.4.2.3* of EN 1998-1, as detailed in Section 4.11.2.3 of this guide (see equation (D4.23)) for all moment-resisting frames
- *equations (6.6)*, *(6.12)*, *(6.30)* and *(6.31)* in EN 1998-1 for steel structures, which also apply to composite steel–concrete structures.

## 6.6. Background of the deformation capacity required by Eurocode 8

*Clauses 6.6.4(3), 6.6.4(4), 6.6.4(5), 6.7.3, 6.8.2(10), 6.8.4(2), 7.7.4(1), 7.8.3(1), 7.9.3(2)*

In Eurocode 8, there is no explicit requirement on the deformation capacity of the full member, when the full member is intended to be the dissipative zone, as is the case for diagonals of frames with concentric bracings. The reason behind this lack of explicit requirement is that the elongation capacity of the steel material applied to long members certainly provides the plastic deformation capacity needed in seismic conditions (see the note to the example on. p. 148). Requirements are explicit for structural typologies in which dissipative zones are rather localized. They are expressed in terms of capacity of absolute deformation related to the ductility class of the structure chosen by the designer:

- In moment-resisting frames, *clause 6.6.4(3)* defines the minimum required values of the plastic hinge's rotation capability in beams or in connections at beam ends as being 25 mrad in Ductility Class Medium (DCM) and 35 mrad in Ductility Class High (DCH).
- For frames with eccentric bracings, *clause 6.8.2(10)* specifies the deformation capacity to be provided by ductile links.
- For frames with bracings, either concentric or eccentric, in which the designer intends to achieve plastic deformation in the partial strength connection, *clauses 6.7.3(9)(a)* and *6.8.4(2)(a)* in EN 1998-1 specify that the required deformation capacity provided by the localized dissipative zones has to be related to the computed global deformation of the structure. This evaluation of the local deformation capacity does not need a very high accuracy. The demand is available from the analysis for the design seismic action in the following way, using the symbols in Fig. 6.3:
  - The analysis of the structure provides the relative interstorey displacement $d_r$ at the ULS. If the analysis for the design seismic action is linear, based on the design response spectrum (i.e. the elastic spectrum with 5% damping divided by the behaviour factor $q$), then the value of the relative interstorey displacement under the design seismic action is that from the analysis multiplied by the behaviour factor

$q$ (i.e. $d_r = q d_{r,el}$); if the analysis is non-linear, the interstorey displacement is determined directly from the analysis for the design seismic action.

- Then the required elongation $dl$ can be computed as $dl = d_r/\cos\alpha$, where $\cos\alpha = l/(l^2 + h^2)^{1/2}$. $dl$ should be compared with the sum of the elongation capacities at the joints at diagonal ends (presuming that both joints are activated, which requires a behaviour curve with strain hardening).
- For example, if $d_r/h = 3.5\%$, $l = 6$ m, $h = 3$ m; then $\cos\alpha = 0.894$, $d_r = 0.035 \times 3 = 0.105$ and $dl = 0.117$ m = 117 mm.
- Note: if the diagonal had been the place for that extension, it would have meant: $\varepsilon = dl/(l^2 + h^2)^{1/2} = 0.017 = 1.7\%$, which is easily achieved by the diagonal.

For moment-resisting frames, the requirements are explicit: for instance, a plastic hinge in a moment-resisting frame of DCH should have a rotation capacity of 35 mrad or 3.5% in the cyclic plastic conditions imposed by an earthquake. These requirements on local ductility can be approximately related to the global ductility performance of a structure: Fig. 6.4 indicates that a moment frame with a DCH joint should be able to sustain a global relative drift of around 3.5%.

The required deformation capacity of dissipative zones has been defined to ensure safety in a worst-case European situation: medium-rise buildings in high-seismicity regions. The real deformation capacity needed in dissipative zones may be smaller under other circumstances, which could be computed in an explicit evaluation of $d_r/H$. The code requirements on local deformation capacity in moment frames may thus seem excessive. However, these requirements have another justification, which results from the huge research effort after the observation of the dramatically poor behaviour of moment connections during the Northridge (1994) and Kobe (1995) earthquakes. Experiments have shown that well-designed and well-fabricated steel components can reach without difficulty the target values mentioned above, while poor designs fail. In that sense, the deformation capacity prescribed for moment connections in Eurocode 8 have the meaning of a threshold or a criterion in a mechanical test, defined in order to distinguish poor from good design details and/or execution. The examples given in Section 6.7 demonstrate this.

## 6.7. Design against localization of strains

*Clauses 6.5.5(1), 6.5.4(1), 7.5.4(1)*

In Eurocode 8, design against localization of strains is stated as a general requirement for joints in *clause 6.5.5(1)*. No imposed design of connections is provided. Some explicit rules are related to the mitigation of strain localization. Conformity to standards on steel material is one of them:

The development of a dissipative zone involves a 'spreading' of yield, which requires strain hardening. As the material becomes harder with plastic strains, strain-hardened sections become more resistant than adjacent non-hardened zones; as a result, the latter yield and harden, which generates a progressive extension of the yielding zone until a plastic zone is formed.

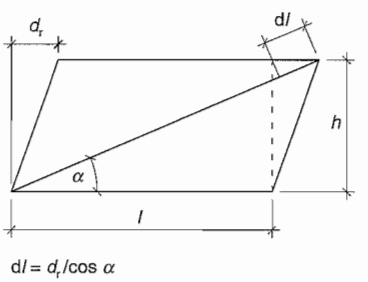



**Fig. 6.3.** Evaluation of the required elongation capacity $dl$ of a diagonal

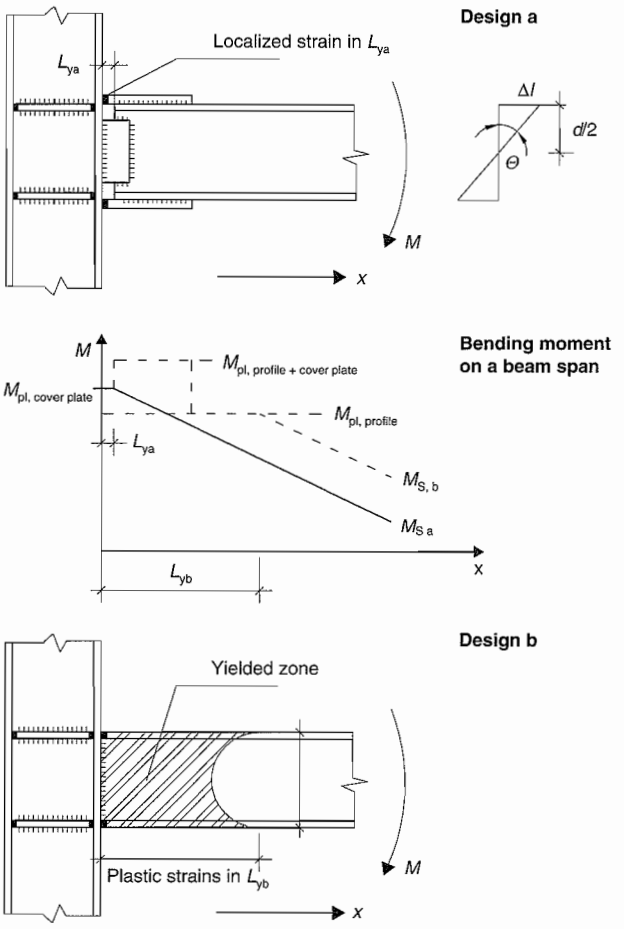



**Fig. 6.4.** Design **a** involves localization of strains which are avoided in design **b**

Strain-hardening of the material corresponds to $f_u/f_y > 1$, and is a necessary property to propagate yield and to avoid yielding all taking place in the narrow section where yielding first occurred. Steels conforming to EN 10025 have $f_u/f_y \geq 1.4$.

*Clause 6.5.4(1)*, which refers to a Eurocode 3 rule for bars in tension, also aims at the mitigation of a localized 'brittle' failure.

It is worth recalling this typical 'capacity design' condition, which requires that the ultimate strength of the 'brittle' section with holes $A_{net}$ should be **greater** than the plastic strength of the full ductile section $A$ (no holes, no stress concentration), so that yielding of the section without holes takes place before failure of the section with holes:

$$Af_y/\gamma_{M0} < A_{net}f_u/\gamma_{M2} \qquad \text{(D6.3)}$$

Applied to a bar, equation (D6.3) guarantees that yielding can affect the whole length of the bar.

To achieve the absolute deformation capacity required in dissipative zones, yielding must take place in zones that are large enough. The meaning of 'large enough' depends on the absolute deformation required and on the deformation scheme envisaged in the dissipative zone.

A dissipative zone can have a large enough dimension, if yielding does not take place in a short-length zone surrounded by a bigger section. If that were the case, yielding would not propagate, and the ductility of the element would appear much lower than that of the material. That situation, in which all yielding takes place in a narrow zone, is called

'localization of strains', and should be avoided, as required in *clause 6.5.5(1)P*. Localization of strains is generally due to design 'details'.

*Clause 6.6.4(3)*

Design against localization of strains is best explained by considering an example, referring to the design of a dissipative zone at a beam end in a moment-resisting frame, in the case of a rigid connection between the beam and the column. Due to the shape of the seismic bending moment diagram, the beam ends in moment-resisting frames are inevitably the sites for dissipative zones. Many design details are possible for the connection of the beam to the column:

- in design **a** of Fig. 6.4, yielding can only develop in a narrow zone of length $L_y$, because the value of $M_{Ed}/M_{Rd}$ in the beam section with the cover plate or further in the beam section alone is smaller than at the connection close to the column face
- in design **b** of Fig. 6.4, the length of flange that may potentially yield is not limited, and it can extend, for instance, up to a length $L_y$ equal to the depth $d$ of the member.

The plastic rotation capacity of these two details can be estimated in two realistic examples: consider a profile with a depth $d = 400$ mm made of steel S500 ($f_y = 500$ MPa) with an elongation corresponding to the yield plateau equal to $\varepsilon_{y,max} = 10\varepsilon_y = 10f_y/E = 10 \times 500/210\,000 = 2.38\%$. This corresponds to an elongation at failure above 20%. The plastic rotation is found by the relationship $\theta = \Delta l/(d/2)$, with $\Delta l = l\varepsilon_{y,max}$ (see Fig. 6.4).

- In design **a** of Fig. 6.4:

  $L_y = 10$ mm, $\varepsilon_{y,max} = 2.38\% \Rightarrow \Delta l = 0.0238 \times 10 = 0.238$ mm, $\theta = 0.238/(400/2) = 1.2$ mrad $\ll 25$ mrad

- In design **b** of Fig. 6.4:

  $L_y = 400$ mm, $\varepsilon_{y,max} = 2.38\% \Rightarrow \Delta l = 9.52$ mm, $\theta = 9.52/(400/2) = 47.6$ mrad $\gg 35$ mrad

Design **b** offers a significant plastic rotation, greater than the 35 mrad requirement for DCH in *clause 6.6.4(3)* of EN 1998-1. Design **a** is unable to provide the required plastic rotation, even for DCM.

The practical conclusions of this simple example are straightforward:

- a length of yielded zone of the order of the depth of the section is needed for an effective plastic hinge to form. This requires avoiding localization of strains
- material with high enough values of $\varepsilon_{y,max}$ and $f_u/f_y$ are required
- for a given material, higher beam depth $d$ means smaller plastic rotation capacity since $\theta = \Delta l/(d/2)$.

Further explanation on moment connections is given in Section 6.9.

## 6.8. Design for global dissipative behaviour of structures

### 6.8.1. Structural types and behaviour factors

*Clauses 6.3.1, 6.3.2(1)*

The behaviour factor $q$ characterizes the ability of a structure to dissipate energy in plastic deformations. A structure can provide high values of $q$ if:

- dissipative zones are able to undergo significant plastic deformations without losing strength
- the topology of the structure is such that a large number of dissipative zones are activated.

The values given in *Table 6.2* of EN 1998-1 have been determined in background studies; however, there is a direct logic relating the topology of structures to the favourable factors mentioned:

- Plastic hinges in beams can be very dissipative local mechanisms; moment-resisting frames can develop a large number of such hinges, and their $q$ can be between 4 and 8;

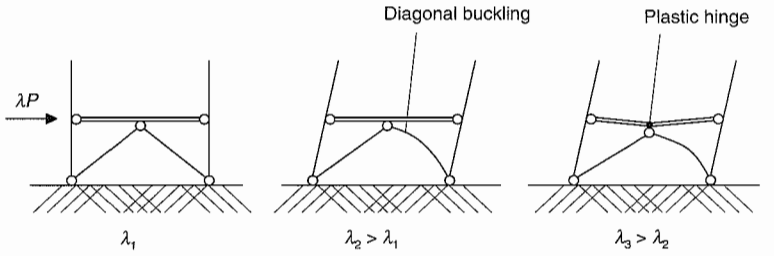



**Fig. 6.5.** In V bracings with strong beams, collapse corresponds to buckling of one diagonal and formation of one plastic hinge in the beam

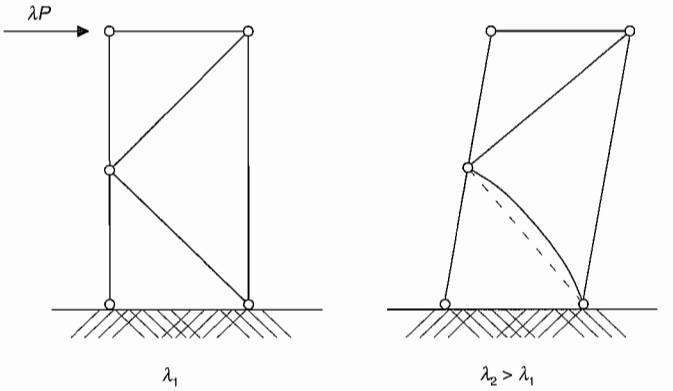



**Fig. 6.6.** In K bracings, collapse corresponds to buckling of one diagonal

frames with eccentric bracings can also be designed to develop a large number of such hinges ($q$ between 4 and 8).

- Diagonals in tension can be very dissipative; frames with concentric bracings can be designed such that many diagonals in tension are activated as dissipative 'links' ($q = 4$) and a global mechanism is formed.
- Panels in shear are very dissipative local mechanisms; frames with eccentric bracings can be designed to develop a large number of these sheared panels ($q$ between 4 and 8).
- Moment-resisting frames in which fewer plastic hinges can form are less dissipative; this is the case for 'inverted-pendulum'-type structural systems ($q$ between 2 and 2.2).
- Structures which require the contribution of elements subject to buckling for their stability, such as V bracings, are less dissipative ($q = 2$ or 2.5), but still possess some ductility if the design of beams is made assuming that they are not supported on bracings for carrying the gravity loads (Fig. 6.5). It should be noted that concentric V bracings are close to certain types of eccentric V bracings in their shape and stiffness, but the $q$ factors of these two topologies differ significantly. The designer in search of both stiffness and a high $q$ factor can benefit from this observation.
- Frames with K bracings cannot be considered as dissipative at all, because once a diagonal has buckled, the frame becomes a portal frame with a plastic hinge at mid-height of a column, which is an unstable structure (Fig. 6.6).

### 6.8.2. Selection of the behaviour factor for design purposes

The values of behaviour factors $q$ provided in *Table 6.2* of EN 1998-1 are the maximum allowed values. The designer can always decide to work with a smaller $q$. This can be justified in some cases by the fact that, as mentioned in Section 6.2, the seismic design condition is not necessarily the most controlling factor for the structure, and trying to make use of the highest

*Clause 6.3.2(1)*

possible value of $q$ may have no practical impact. It should be stressed that, whatever the value of $q$ used in the analysis, the displacements of the structure are the same, due to the application of the equal displacement rule. If drift control criteria prevail in the design, increasing the value of $q$ does not help. On the contrary, considering higher $q$ values will require the use of more stocky steel sections (smaller $b/t$ values) due to the imposed relationship between global ductility (high $q$) and local ductility (prevention of buckling) expressed in *Table 6.3* of EN 1998-1.

In practice, for flexible structures such as moment-resisting frames, the values of $q$ to use in order to avoid iterations are often closer to 3 than to 6. If deformation criteria prevail, it may be preferable to select a stiffer typology of earthquake-resistant structure, e.g. frames with concentric or eccentric bracings, which are more likely to profit from $q$ values close to those of *Table 6.2*.

## 6.9. Moment-resisting frames

### 6.9.1. Design objective

*Clause 6.6.1(1)*

Like moment-resisting frames made of other materials, the design objective for steel moment frames is that plastic hinges form in the beams and not in the columns. This requirement is waived at the base of the frame, at the top floor of multi-storey buildings and for one-storey buildings. This requirement is assumed to be fulfilled if equation (D4.23) is satisfied.

Equation (D4.23) expresses a local hierarchy criterion between plastic resistances of beams and columns intersecting at one node, in line with the capacity design concept. This criterion has the advantage of simplicity. However, parametric studies using dynamic non-linear or pushover analysis show that it does not guarantee that plastic hinges occur exclusively in beams. Absolute prevention of plastic hinging in columns would require that the overstrength factor 1.3 in equation (D4.23) be raised to higher values. In fact, in order to ensure the global plastic mechanism defined as the design objective, it has been demonstrated[73] that the use of more sophisticated design methods, such as that proposed by Mazzolani and Piluso,[74] are needed. However, as the plastic mechanisms obtained by fulfilling equation (D4.23) are always global ones, despite involving some plastic hinging in columns, the decision has been taken to keep equation (D4.23) as the design criterion in EN 1998-1 to achieve the strong column–weak beam design of moment-resisting frames.

### 6.9.2. Analysis issues in moment-resisting frames

*Clause 4.4.2.2(2)*

Moment-resisting frames are sensitive to $P$–$\Delta$ effects. However, if the sensitivity coefficient defined in equation (D4.20) (see Section 4.6.5) is less than 0.1 at every storey, the action effects may be determined using a first-order theory considering the initial geometry of the structure. As the damage limitation requirements on drift (see Section 4.11.1) are demanding, the sensitivity coefficient is in practice always less than 0.1.

*Clauses 6.3.2(1), 6.3.2(3), 6.3.2(5)*

Background studies[74,75] have shown that overstrength in moment-resisting frames can be high, in particular in design for moderate earthquake regions. By undertaking pushover analysis of a trial design, it is possible to better evaluate the value of the parameter $\alpha_u/\alpha_1$ and to increase the value of $q$ from the default value of 1.1–1.3 to a value up to $q = 1.6$, as allowed in *clause 6.3.2(5)*. This effort in analysis may generate a significant economical impact in structures which are not governed by limits of deformation under vertical and horizontal actions (damage limitation earthquake included). Heavy low-rise industrial frames are most likely to correspond to this situation.

*Clause 4.4.2.2(1)*

Under a combination of gravity loading and seismic action effects, the values of the maximum positive and negative bending moments in beams can be very different. To be adequate, sections must be related to the absolute maximum values. However, following the general statement in *clause 4.4.2.2(1)*, redistribution of bending moments according to



**Fig. 6.7.** Support of the bottom flange to prevent lateral buckling of beam ends under hogging moments. (Courtesy of Chia-Ming Uang, University of California, San Diego)

EN 1993-1 for steel structures is allowed, which brings a reduction in the design moment of beams. The practical interest of this redistribution is explained further later in this chapter.

### 6.9.3. Design of beams and columns

In beams of classes 1 and 2, the value of $b/t$ of walls of sections are such that local buckling only takes place after substantial plastic rotations, large enough to fulfil the plastic rotation demand from the earthquake. *Clause 6.6.2(1)*

The prevention of lateral torsional buckling is another serious concern in beam elements, in particular those made of H or I sections, due to coupling between the local and lateral buckling instability phenomena: inward buckling on one side of the flange is accompanied by outward buckling on the other side, which makes the profile unsymmetrical and generates lateral movement. Substantial lateral restraint of the flanges is thus absolutely needed to develop the full plastic capacity of beams. Connection to a floor slab provides excellent lateral support; however, both the upper and lower flanges should be restrained, because of the reversal of plastic moments at beam ends (Fig. 6.7).

The rule for calculation of the design shear force, *Clauses 6.6.2(2), 7.7.3(3)*

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \quad \text{(D6.4)}$$

reflects a capacity design requirement: the seismic component $V_{Ed,M}$ of the design shear $V_{Ed}$ in a beam is related to the ULS situation, in which the plastic moments $M_{pl,Rd}$ develop at both beam ends (and not only the bending moments computed as seismic action effects in the elastic analysis), following a rationale explained in Section 6.5 above,

$$V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B})/L \quad \text{(D6.5)}$$

where A and B denote the beam end sections.

The rule for calculation of the design axial force in columns, equation (D6.1) in Section 6.5, and similar rules for calculation of the design shear force and bending moment of the column, $V_{Ed}$ and $M_{Ed}$ respectively, also reflect capacity design requirements. In this case, the *Clauses 6.6.3(1), 6.6.3(2), 7.7.1*

element considered (a column) is not the same as the element in which the plastic zone develops (a beam). Attention has to be paid to the fact that the yield stress of the beam may be higher than the design yield stress, so that the value of the axial force $N_{Ed}$ in the column corresponding to the formation of the plastic hinge in the beam is higher than the value $N_{Ed,E}$ computed from the elastic analysis. The factor $1.1\gamma_{ov}$ takes care of this material overstrength problem, while $\Omega$ takes into account the section overstrength resulting from the fact that the value $M_{pl,Rd,i}$ of most sections is higher than the value of $M_{Ed,i}$ computed from the analysis.

Formation of plastic hinges in columns at the base of the frame is an expected feature of moment-resisting frames, as it is required for compatibility of deformations in the global plastic mechanism. Stability checks of base columns at the ULS have to consider a bending moment diagram which corresponds to this situation.

*Clauses 6.6.3(6), 6.6.3(7), 6.6.4(1), 6.6.4(4)*

The design of the panel zone of the column has to fulfil

$$V_{wp,Ed} \leq V_{wp,Rd} \tag{D6.6}$$

Due to the existence of plastic bending moments of opposite signs at the beam ends adjacent to a column as indicated in Fig. 6.8, the design shear $V_{wp,Ed}$ applied to the panel zone is high (Fig. 6.9). If the plastic hinges are formed in the beam sections adjacent to the column on its left- and right-hand sides, the horizontal design shear $V_{wp,Ed}$ in the panel zone is equal to

$$V_{wp,Ed} = M_{pl,Rd,left}/(d_{left} - 2t_{f,left}) + M_{pl,Rd,right}/(d_{right} - 2t_{f,right}) \tag{D6.7}$$

The value of $V_{wp,Ed}$ computed from equation (D6.7) has to be compared according to equation (D6.6) with the design resistance of the panel zone of the column, $V_{wp,Rd}$, computed considering the geometric dimensions of the column section, in particular the column depth $d_c$ (see Fig. 6.9) and the column depth of the web $h_w$. If the plastic hinges are formed at a distance $D$ from the column face, the moments to consider in equation (D6.7) are

$$M_{Sd,left} = M_{pl,Rd,left} + V_{Ed,M,left}D \qquad M_{Sd,right} = M_{pl,Rd,right} + V_{Ed,M,right}D \tag{D6.8}$$

Equation (D6.6) refers to the case of column web panels of small slenderness, which are able to develop their full plastic strength. Buckling limits the capacity of more slender webs, in which case the shear buckling resistance of the web panel should be used on the right-hand side of equation (D6.6).

The design shear $V_{wp,Ed}$ generally surpasses the shear resistance $V_{wp,Rd}$ of the panel zone in columns made of standard rolled sections, requiring that reinforcing plates are installed, either in the form of a 'doubler' plate welded onto the column web or by means of two plates welded to the flanges and transverse stiffeners (Fig. 6.10). Welds should be sized to the additional plate thickness.

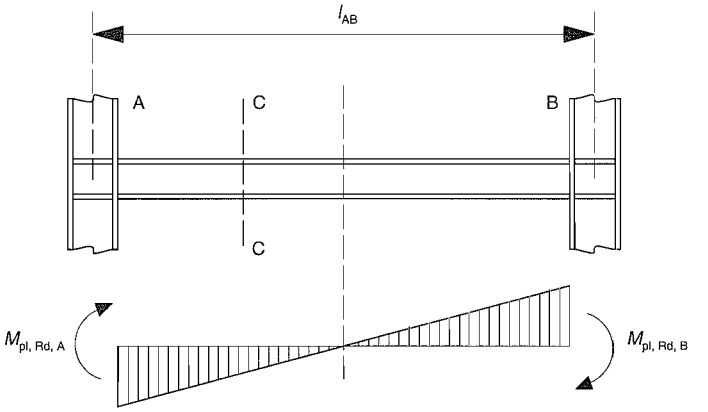



**Fig. 6.8.** Design moments used to compute $V_{Ed,M}$

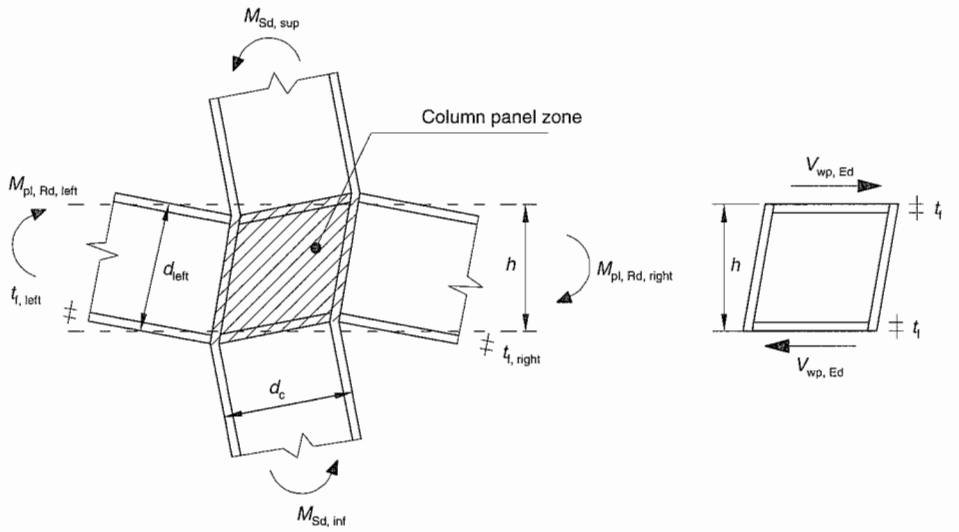



**Fig. 6.9.** Moments of opposite signs generate high shear in the column panel zone

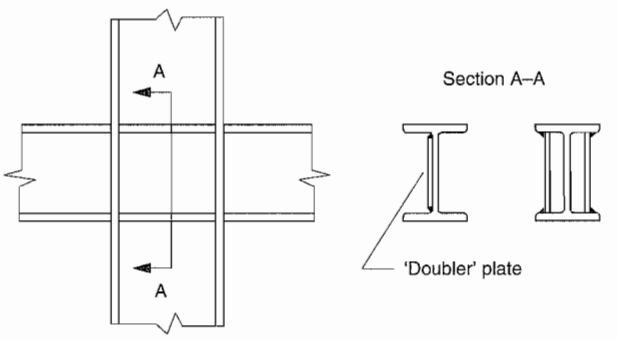



**Fig. 6.10.** Additional plates to increase resistance of the panel zone

Equation (D6.6) reflects a decision on the acceptability of the plastic deformation in shear of column web panels. This plastic mechanism is known to be very ductile and stable. However, two reasons justify why it should not be the basic local energy dissipative mechanism in moment-resisting frames:

- The global mechanism selected as a design objective for moment-resisting frames corresponds to 'weak beams–strong columns', the intention being that beam yielding spreads throughout the whole structure, a local 'soft-storey'-type mechanism being avoided. In this concept, columns remain fully elastic. Accepting dissipative mechanisms in column web panels would violate the concept.
- The plastic deformation in shear of column web panels results in local bending of the column flanges at the location where web stiffeners and column flanges intersect. If the beam is welded to the column flanges, the aforementioned local bending of column flanges may cause high plastic strains in the connection zone and generate early failure.

The decision behind equation (D6.6) means that the plastic shear deformation of column web panels is accepted in a limited manner, by allowing plastic hinge deformation in the beam and a plastic shear deformation in the web panel to take place simultaneously. This decision is also supported by experiments that have shown that the most ductile connection behaviour is observed when the two phenomena take place simultaneously. In the case of

experimental evidence, the contribution of the web panel to the plastic rotation capability is limited to 30% of the total.

### 6.9.4. Design of dissipative zones

*Clauses 6.6.4(2), 6.6.4(3), 6.6.4(5), 6.6.4(6), 6.5.5(7)*

As mentioned before, dissipative zones in moment-resisting frames should be plastic hinges activated by bending moments. They appear at beam ends, due to the shape of the bending moment diagram under the seismic action (see Fig. 6.8).

Plastic hinges can take place in the connections, in the case of partial-strength or semi-rigid connection design. There are many possible designs, using connecting components of various types: flexible end plates, angles, etc. Such designs have to face the problem of assessing the deformation capacity – in this case the rotation capacity – under dynamic cyclic conditions. However, there does not yet exist a list of adequate designs, prescriptive or otherwise. The designer thus has to develop his or her own design, referring to the existing literature.[76] Recourse can also be made to EN 1993-1-8, by using the 'components method' after selecting as dissipative components those listed in Section 6.4 above as 'favourable factors' for ductility. National Annexes to EN 1998-1 may provide explicit reference to other literature as guidance on connection design.

Plastic hinges in moment-resisting frames are typically selected in beams, not only because of the lack of data on partial strength connections but also because moment-resisting frames are flexible by nature and add flexibility to the connections, resulting in the need to compensate for the high flexibility by bigger steel sections, which may result in an uneconomic solution.

If rigid connections are used, two design options exist. The design can be such that:

(1) the plastic hinges occur in the beam sections adjacent to the column flanges, which is the standard situation with classical connection design, or
(2) the plastic hinges are displaced from the column flanges, in order to separate the stress concentration of the connection from the plastic strains of the plastic hinge. This can be achieved by:
   - strengthening the connection
   - weakening the beam by trimming the flanges (reduced beam sections or a 'dogbone' design, as shown in Fig. 6.7). This original idea[77] has been developed further and incorporated in design recommendations.[78]

Examples of the strengthening and the weakening techniques are sketched in Fig. 6.11.

Possibilities for displacing the plastic hinges from the column flanges have been widely developed in the USA as a result of the poor performance of moment connections in moment-resisting frames in the Northridge (1994) and Kobe (1995) earthquakes. These are justified design options when there is concern about the quality of the most stressed zone, which is that at the column face. The inquiry and research following these earthquakes have shown that several factors other than the connection design generated the poor performance observed:

- base material with low weldability and low elongation capacity
- improper weld material (of low toughness)
- weld preparation promoting stress concentration and defects (V preparation with a cope hole and welding on a backing bar – both made necessary by the fact that only one-sided welding from above was feasible for on-site welding)
- improper welding techniques (site welding, gas protection).

However, there is extensive experimental evidence of more classical connections achieving the target plastic rotations without difficulty. Such observations have been made in tests on H and IPE profiles with a depth of up to 450 mm, using European standards for materials and fabrication with standard precaution, such as suitable weld preparation (K), weld metal and welding procedure (e.g. in the case of K preparation, welding from one side is followed

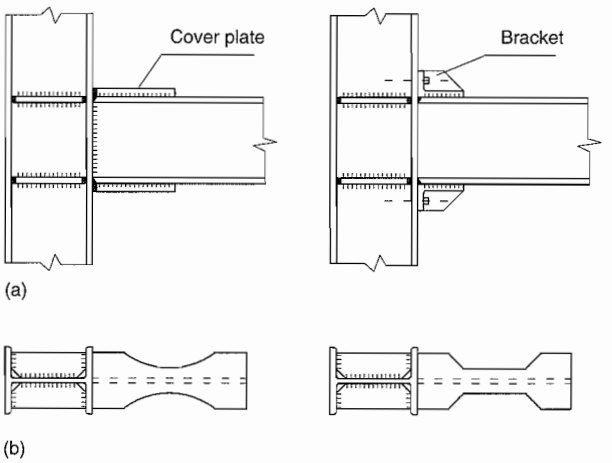



**Fig. 6.11.** Examples of (a) strengthening and (b) weakening techniques in moment frame connections

by welding from the other side), and base material of appropriate toughness and weldability. This justifies the position adopted in EN 1998-1, that detailed prescriptions and limitations on connection design are not yet required. Further details may be included in the National Annexes of EN 1998-1. Some uncertainties remain for sections larger than H and IPE profiles with a depth of 450 mm, for which research is still ongoing.

Whatever the type of connection, the demonstration of deformation capacity has to be provided in terms of absolute rotation capability $\theta_p$, as stated in *clause 6.6.4(3)*. The wording corresponds to a definition of $\theta_p$ related in the simplest possible manner to process test data, not to theoretical definition of elastic and plastic rotations. $\theta_p$ is the deflection measured at the midspan of the beam, divided by half of the span length. If the test is made on a shorter length, the obtained results can be adjusted to this definition. $\theta_p$ includes the following contributions:

- deformation of the connection, including column web panel deformation
- plastic hinge rotation
- elastic deformation of the beam.

$\theta_p$ should not include any elastic contribution of the column deformation outside the panel zone, because this deformation has nothing to do with the rotation capability of the plastic zones under characterization in a test, namely those in the beam and its connection to the column.

### 6.9.5. Limitation of overstrength

As explained in Section 6.2, the complete design process may generate a structure with more strength than strictly needed for the resistance to the design earthquake. The excess in material may have several origins, for example:

- the limits of deflection in the design of beams under gravity loading may lead to sections larger than those needed for resistance to earthquakes
- the capacity design of columns to meet equation (D4.23).

If the design is such that the drift limits under the damage limitation SLS earthquake are verified, the designer can consider reducing the excess size of structural members. EN 1998-1 offers three possibilities for reducing the overstrength of beam and column sections:

- The design of beams under seismic loading is made considering a combination of gravity loading and seismic action effects which result in an unsymmetric bending moment

diagram, in which the absolute values of maximum positive and negative bending moments can be quite different (Fig. 6.12). The usual approach is to select the beam section such that in the beam $M_{pl,Rd} \geq \max(|M_{Sd}^{+}|, |M_{Sd}^{-}|)$. But smaller sections can be justified via a redistribution of bending moments within the limits allowed by EN 1993-1. Changing the reference line of the moments reduces the maximum design moment and the beam section accordingly.

- Due to the shape of the diagram of beam moments, the capacity design of columns (see equation (D4.23)) refers to the plastic resistance of the beam ends sections, $M_{pl,Rd,b}$, or to the plastic resistance of the connection, $M_{pl,Rd,c}$, whichever is smaller. If beam design produces significant overstrength with $M_{pl,Rd} \gg \max(|M_{Sd,E}^{+}|, |M_{Sd,E}^{-}|)$, compliance to equation (D4.23) brings about a similar overstrength of columns. This can be avoided by a reduction of $M_{pl,Rd,b}$ through a reduction in the beam section (reduced beam or 'dogbone') at the beam ends.
- The column overstrength can also be avoided by making use of the beam-to-column partial strength connections designed to achieve $M_{pl,Rd,c} \approx \max(|M_{Sd,E}^{+}|, |M_{Sd,E}^{-}|)$.

Of course, if these possibilities are used, the modified structure is more flexible than the first design, and its response is changed. A new analysis and a verification of the structure considering the modifications have to be performed.

## 6.10. Frames with concentric bracings

### 6.10.1. Analysis of frames with concentric bracings considering their evolutive behaviour

*Clauses 6.7.1(1), 6.7.2(2)*

In Eurocode 8, the design concept for frames with concentric bracings is that diagonals in tension are the reliable dissipative zones, while diagonals in compression buckle and do not contribute significantly to stiffness and resistance.

The problem is that the reality is evolutive. In a first stage, the compression action effect in the diagonal can increase up to the buckling strength $N_{pl,Rd}$; however, in the following cycles the strength of that diagonal in compression decreases, due to permanent deformation from the first buckling stage, so that the value of the buckling strength, $N_{pl,Rd}$, cannot be reached anymore. This decrease in resistance is sharp, progresses with loading cycles and cannot easily be estimated. The stiffness and strength of the structural system accordingly decrease to the reference situation envisaged in EN 1998-1. The question is to propose a design approach which is safe-sided in estimations of the evolutive behaviour. This reality is behind the Eurocode 8 clauses which deal with diagonals in compression, although these are in principle outside of the reference design model.

*Clause 6.7.2(3)*

For all types of bracing, design considering explicitly the contribution of the compression diagonals using a justified non-linear procedure is allowed, as stated in *clause 6.7.2(3)*. For the sake of simplicity, Eurocode 8 proposes simple alternatives applicable with a standard

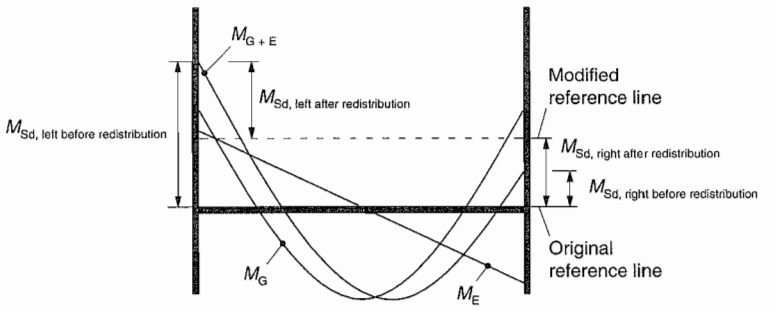



**Fig. 6.12.** Moment redistribution to reduce design moments

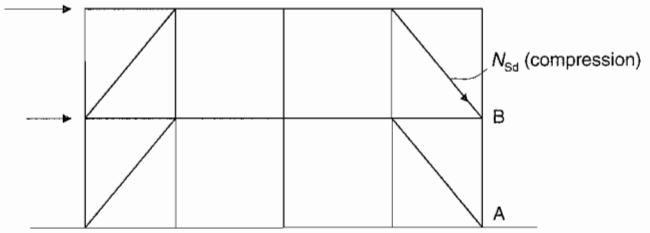



**Fig. 6.13.** Decoupled diagonal bracings

linear elastic analysis. These alternatives entail special requirements, which are discussed later.

### 6.10.2. Simplified design of frames with X bracings

*Clauses 6.7.2(1), 6.7.4(1), 6.7.3(1), 6.7.3(2), 6.7.3(3), 6.7.3(4)*

In a standard design, the following simplified approach may be used:

- The analysis of the structure is realized considering that only one diagonal in each X bracing is present, the other diagonal being considered as already buckled and unable to provide strength. This corresponds to an underestimation of both the stiffness and the strength of the structural system at the initial (pre-buckling) stage, but to a safe-side estimate at the post-buckling stage.
- The beams and columns are capacity designed according to the real yield strength of the diagonals, for bending with axial force given by equation (D6.1) and bending moment from the analysis for the combination of the design seismic action with gravity loads according to Section 4.4.1.

However, this simplified approach could be dangerous for the stability of the structure, if it does not take into account that action effects of compression in columns and beams at the pre-buckling stage are higher than in the post-buckling stage envisaged in the analysis. Indeed, if the buckling loads of the diagonals are close to their yield load in tension, the initial shear resistance $V_{init}$ of the X bracing is underestimated by a model where only one diagonal is considered present. If low-slenderness diagonals are used, $V_{init}$ can be close to double the value of $V_{pl,Rd}$ computed with the hypothesis of one active (yielded) diagonal. The only way to prevent this unsafe situation is to design slender diagonals having their buckling load at most around $0.5N_{pl,Rd}$. This condition is behind the prescribed lower bound limit value of 1.3 for the slenderness $\bar{\lambda}$. The prescribed upper bound limit $\max(\bar{\lambda}) = 2$ in *clauses 6.7.3(1)* to *6.7.3(3)* is justified by the aim to avoid shock effects during the load reversal in diagonals. This limitation is removed for low-rise structures (up to two storeys); this means that rods or cables can be used as diagonals in such buildings.

### 6.10.3. Simplified design of frames with decoupled diagonal bracings

*Clauses 6.7.3(2), 6.7.4(3)*

The structural behaviour of decoupled diagonal bracings is similar to that of X bracings, but as there are two braced zones, each containing one diagonal instead of two, the problem of overstrength described above for X bracings takes place differently, as twice as many columns are involved in the bracing.

For this reason, *clause 6.7.3(2)* states that no lower bound value of slenderness restricts the possible dimensions of the diagonals in this case. However, to be complete the design must also address, as stated in *clause 6.7.4(3)*, the part of the frame in which the diagonals are in compression, because they introduce compression in the column intersecting their lower end, for instance in column AB of Fig. 6.13.

### 6.10.4. Simplified design of frames with V bracings

*Clauses 6.7.2(2), 6.7.4(2)*

The simplified design approach for V or Λ bracings (called inverted V in Eurocode 8 and 'chevron' bracings in North American terminology) is different from that considered for X bracings, due to the fact that both the tension and the compression diagonals are considered

in the analysis. This results in higher computed stiffness and strength of the structure with respect to an analysis ignoring the compression diagonal. This more complete model with respect to X bracings is, however, compensated to take into account the degradation of resistance in the compression diagonal, by associating frames with V bracings to a lower value of the $q$ factor,

- $q = 2$ in DCM
- $q = 2.5$ in DCH,

instead of 4 for X bracings. Moreover, the beams should be dimensioned for:

- the action effects of all non-seismic actions without considering the intermediate support provided by the diagonals
- the unbalanced vertical seismic action effect applied to the beam by the braces after buckling of the compression diagonal, calculated assuming that the force in the brace in tension is equal to $N_{\text{pl, Rd}}$ and that in the brace in compression to a buckling load equal to a fraction of $N_{\text{pl, Rd}}$ with a recommended value of 30%.

### 6.10.5. Criterion for the formation of a global plastic mechanism

*Clauses 6.7.3(5), 6.7.4(1)*

The design requirement for diagonals in *clause 6.7.3(5)* simply expresses the need for a section able to take the computed action effect $N_{\text{Ed}}$: $N_{\text{pl, Rd}} \geq N_{\text{Ed}}$. To form a globally dissipative bracing structure, which means achieving a global plastic mechanism in which yielding affects a significant number of diagonals, two conditions must be fulfilled:

(1) The beams and columns have to be capacity designed to the real strength of the diagonals. This prevents pre-emptive yielding in non-dissipative elements, such as columns.
(2) The design criterion should give to each diagonal a significant probability of yielding under seismic loading.

To fulfil condition 1, beams and columns are capacity designed according to the real yield strength of the diagonals, for bending with axial force $N_{\text{Ed}}$ given by equation (D6.1) However, the ratios $\Omega_i = N_{\text{pl, Rd},i}/N_{\text{Ed},i}$ are normally different in each diagonal, as $N_{\text{Ed},i}$ or $N_{\text{pl, Rd},i}$ are different. In fact, due to the heightwise distribution of the seismic shear in the structure, $N_{\text{Ed}}$ decrease from the bottom to the top of the frame. So, if all the diagonals have the same section, yielding will take place only in the diagonals of the first storey, possibly reaching the second storey due to strain hardening in the ground storey diagonals. In this case, a 'soft-storey' type of mechanism will form (Fig. 6.14).

*Clause 6.7.3(8)*

In order to force the formation of a global mechanism, a second condition is imposed, which aims to fit better the distribution of diagonal strengths $N_{\text{pl, Rd}}$ to the distribution of computed action effect $N_{\text{Ed}}$. The ratios $\Omega_i = N_{\text{pl, Rd},i}/N_{\text{Ed},i}$ which define the section overstrength of diagonals with respect to their minimal required strength ($N_{\text{Ed},i}$) may not vary widely over the height of the structure. $\Omega$ considered in equation (D6.1) being the minimum $\Omega_i$, the values of all other $\Omega_i$s should be in the range $\Omega$ to $1.25\Omega$. In practice, this condition forces the use of different sections of diagonals over the height of the structure.

### 6.10.6. Partial strength connections

*Clause 6.7.3(9)*

Partial strength connections are not a familiar option in frames with concentric bracings, though several reasons justify the statement that concentric bracings are an excellent application field for such connections. These reasons are:

- Frames with concentric bracings possess a high stiffness because of their topology, and easily fulfil deformation criteria. Thus, unlike moment-resisting frames, additional flexibility in the connections is not penalized by the need to increase the sections of structural elements whenever semi-rigid connections are used.
- Partial strength connections can be designed so that their plastic strength is lower than the buckling strength of diagonals. In this case, the difficulties in dealing with systems

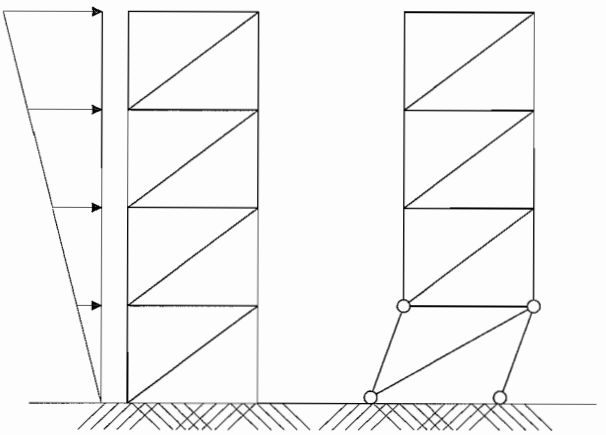


**Fig. 6.14.** Soft-storey development in truss braces

with evolutive stiffness and strength explained above for 'tension-diagonal-only' systems do not apply.

- In practice, the use of partial strength connections means that all diagonals can be present in the model used for the analysis, which brings additional stiffness in comparison with the 'tension-diagonal-only' structural model discussed above. This positive contribution compensates for the flexibility possibly introduced by the semi-rigid connections.
- Partial strength connections can be developed as industrial products with 'calibrated strength', reducing the effect of uncertainties in the overstrength design of beams and columns; both $\Omega$ and $\gamma_{ov}$ can then be taken as equal to 1.0.
- It can be demonstrated that the behaviour factor $q$ of frames with concentric bracings and partial strength connections for the diagonals is higher than without such connections, due to better control of the global plastic mechanism.
- If the structure is damaged by an earthquake, yielded and permanently deformed components only require replacement in localized areas of the dissipative connections.

The soundness of these reasons has been demonstrated in experimental tests on connections and frames and by numerical modelling of complete structures under earthquake action. Specific designs[79] have demonstrated a potential in terms of strength, stiffness and elongation capacity, complying for instance to the requirement of 117 mm for two connections in the short example of Section 6.6 (see p. 148).

## 6.11. Frames with eccentric bracings

### 6.11.1. General features of the design of frames with eccentric bracings

*Clauses 6.8.1(1), 6.8.1(2), 6.8.1(3)*

The geometry of frames with eccentric bracings is close to that of frames with concentric bracings; some intentional eccentricities in the layout of bars generate bending moments and shear. These structures resist horizontal forces essentially by axially loaded members, but they are designed to yield first in shear or bending in 'seismic links'. The latter are zones created by the shift of bars of the reference concentric brace away from the usual intersection with other bars (Fig. 6.15).

The analysis of frames with eccentric bracings does not require all the approximations made in the case of concentric bracings, because such frames are not designed so that diagonals will buckle under seismic conditions. Diagonals are part of the non-dissipative zones; they are capacity designed to the strength of the links, in order to remain elastic and to avoid buckling.

There are several reasons for selecting a frame with eccentric bracings for an earthquake-resistant structure:

- eccentric bracings combine stiffness with a high $q$ factor (between 4 and 8)
- connections are between three bars, not four as in frames with concentric bracings – this results in less complicated connection details, which may also simplify the erection of the structure
- diagonals are parts of the structural system taking gravity loads, and are considered to provide strength and stiffness against these loads.

### 6.11.2. Short links versus long links

*Clauses 6.8.2(3), 6.8.2(4), 6.8.2(6), 6.8.2(8), 6.8.2(9)*

Seismic links are designed for the computed seismic action effect in shear or in bending of the link, by complying to

$$V_{Ed} \le V_{p,\,link} \qquad M_{Ed} \le M_{p,\,link} \tag{D6.9}$$

in which $V_{p,\,link}$ and $M_{p,\,link}$ are, respectively, the plastic shear and bending resistance of the link. In H sections,

$$V_{p,\,link} = (f_y/\sqrt{3})t_w(d - t_f) \qquad M_{p,\,link} = f_y b t_f (d - t_f) \tag{D6.10}$$

*Equation (6.17)* in EN 1998-1 allows computation of $V_{p,\,link}$ taking into account the interaction of shear with axial force, while *equation (6.18)* in EN 1998-1 allows computation of $M_{p,\,link}$ taking into account the interaction of bending with axial force.

The local plastic mechanism in seismic links depends on the topology of the structure. The topology of a structure can be such that bending moment diagrams in the link are symmetric (Fig. 6.16) or unsymmetric (Fig. 6.15).

The plastic mechanism in seismic links also depends on their length, $e$. Short links yield essentially in shear; the energy dissipated in the plastic mechanism is $W_V = V_{p,\,link}\theta_p e$.

Long links yield essentially in bending; in a long link subjected to the symmetrical action effect $M$ as in Fig. 6.16, the energy dissipated in the plastic mechanism is $W_M = 2M_{p,\,link}\theta_p$.

The limit between long and short links corresponds to the situation in which yielding may take place either in shear or in bending:

$$W_M = W_V \Rightarrow 2\,M_{p,\,link}\theta_p = V_{p,\,link}\theta_p\, e \Rightarrow e = 2M_{p,\,link}/V_{p,\,link} \tag{D6.11}$$

However, for values of $e$ around the limit of equation (D6.11), significant bending moments and shear forces exist simultaneously, and their interaction has to be considered. In EN 1998-1, the value of $e$ for considering only plastic mechanism only in shear is

$$e < e_s = 1.6M_{p,\,link}/V_{p,\,link} \tag{D6.12a}$$

The value of $e$ for considering only a plastic bending mechanism is

$$e > e_L = 3M_{p,\,link}/V_{p,\,link} \tag{D6.12b}$$

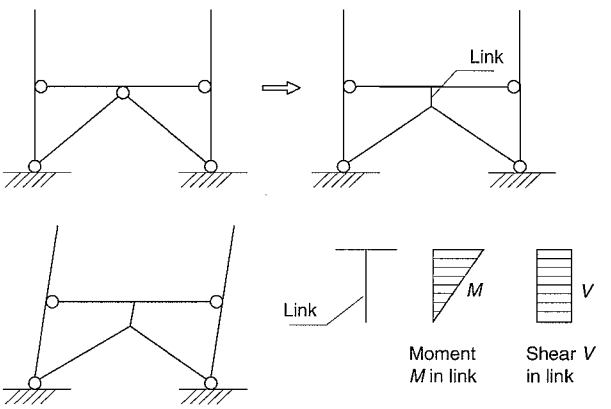



**Fig. 6.15.** Conversion of concentric V bracing to eccentric bracing with a vertical link

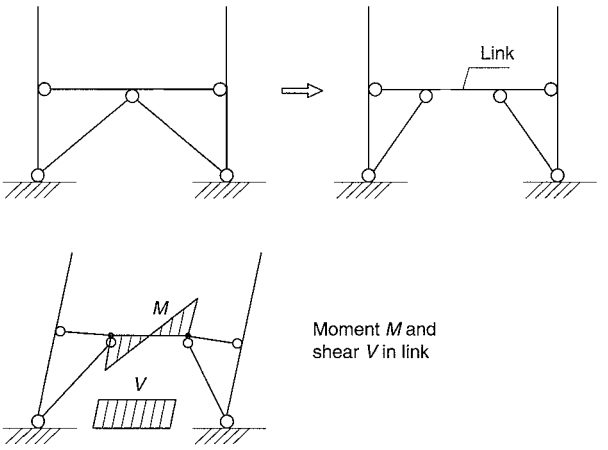



**Fig. 6.16.** Conversion of concentric V bracing to eccentric bracing with a horizontal link

Between these two values, $e_s$ and $e_L$, links are termed 'intermediate', and the interaction between shear and bending has to be considered by making use of *equations* (*6.17*) to (*6.20*) in EN 1998-1.

If the topology of the structure is such that the bending moment diagram is not symmetric, only one plastic hinge will form if the link is long, so that $W_M = M_{p,\,link}\theta_p$. Then, the limit length between long and short links corresponds to

$$e = M_{p,\,link}/V_{p,\,link} \tag{D6.13}$$

This is, for instance, the case for the vertical shear links in Fig. 6.15. Values of $e_s$ and $e_L$ are defined in *equations* (*6.24*) to (*6.26*) in EN 1998-1 to cover this situation. These expressions are made continuous with those for symmetric situations by means of a factor $\alpha$ which characterizes the shape of the diagram of bending moments. In the example of Fig. 6.15,

$$\alpha = M_{Ed,\,A}/M_{Ed,\,B} = 0 \qquad e_s = 0.8M_{p,\,link}/V_{p,\,link} \qquad e_L = 1.5M_{p,\,link}/V_{p,\,link} \tag{D6.14}$$

### 6.11.3. Criteria to form a global plastic mechanism

The criteria to form a global plastic mechanism in frames with eccentric bracings are similar to those with concentric bracings, because they correspond to the same concept: *Clauses 6.8.3(1), 6.8.2(7)*

(1) The beams, columns and connections are capacity designed to the real strength of the seismic links. This is achieved by complying to the following expressions, which are analogous to equation (D6.1):

$$N_{Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed,\,G} + 1.1\gamma_{ov}\Omega N_{Ed,\,E} \tag{D6.15}$$

$$E_d \geq E_{d,\,G} + 1.1\gamma_{ov}\Omega_i E_{d,\,E} \tag{D6.16}$$

(2) A criterion gives to each seismic link a similar probability of attaining yield under seismic loading. This is achieved by enforcing limits on the link overstrength ratios with respect to their minimum required strength, $\Omega_i$, similar to those applying for the diagonals of frames with concentric bracings: the ratios $\Omega_i$ should remain within a 25% range of variation, so that yielding appears simultaneously at several places over the height of the building, and a global mechanism is formed.

### 6.11.4. Selection of the typology of eccentric bracings

There are many possible typologies of eccentric bracings, involving seismic links, which can be either short or long.

The choice between short and long links is partly determined by the following considerations:

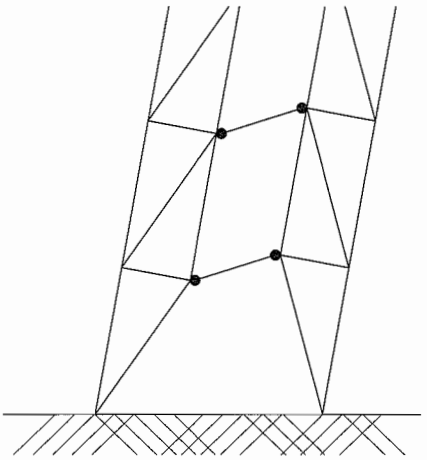


**Fig. 6.17.** Example of eccentric bracing that enforces a global plastic mechanism

- Short links provide more stiffness than long ones.
- Shear deformations essentially are in-plane deformations of the web of the link section, without a marked tendency to lateral torsional buckling.
- Long links mean strong bending effects and plastic hinges in bending with buckling of flanges, which distorts the symmetry of sections under cyclic load conditions, with the potential for lateral torsional buckling. This effect should be prevented, which requires strong lateral restraints of both the upper and the lower flange of the profile.

The choice between various typologies is influenced by many factors, including architecture, which can impose a choice by requiring certain openings.

Structural considerations may also intervene:

- It has been recalled above (with reference to *clause 6.8.2(7)* in EN 1998-1) that there is a requirement for 'tuning up' of the strength of the dissipative zone to the heightwise distribution of the seismic shear in the building, in order to make the spread of yielding over the height of the structure uniform.
- If the seismic links are in beams, while the beam sections are determined by design checks other than those of the ULS under seismic conditions, the above 'tuning up' may require an important overstrength of beams and consequently of all other structural components, due to their capacity design to beam strength. Frames with V or inverted-V eccentric bracings in which the Vs have a flat horizontal tip correspond to this situation.
- One way to escape this penalizing situation is to select a typology which forces all seismic links to yield simultaneously, such as the evolution of the frame with eccentric V bracings in Fig. 6.17.
- Vertical seismic links as in the typology of Fig. 6.15 can more easily be designed as specific 'ductile fuses', because gravity loading subjects them essentially to axial forces, which do not interact significantly with their bending and/or shear resistance.

### 6.11.5. Partial strength connections

*Clauses 6.8.4(2), 6.8.4(3)*

Frames with eccentric bracings making use of partial strength connections do not correspond to the original intention of such bracings, which is to dissipate energy in zones of structural members called 'seismic links'. Nevertheless, using partial strength connections in a topology of frame in which bracings are not concentric is just another way of thinking of the 'link'; so there is no real reason against such a choice, once the ductility and stability issues raised in *clause 6.8.4* of EN 1998-1 are satisfied. All the favourable aspects of partial strength connections mentioned in Section 6.10.6 for frames with concentric bracings apply.

## 6.12. Moment-resisting frames with infills

There are several possible types of moment-resisting frames with infills, depending on the type of infill and its connection to the frame. *Clauses 6.10.3(2), 6.10.3(3)*

The relative stiffness of the steel moment frame and of the panels influences the distribution of forces between them. If the infill panels are stiffer than the frame, they attract the earthquake action effect. Depending on their composition, the panels can be:

- The primary component of earthquake resistance: frames with well-designed, well-connected reinforced concrete or masonry infill panels correspond to this situation.
- The effective earthquake-resisting structure only in the first stage of the response, after which the infills crush and fail. Then, the moment frame takes over. However, as such panel crushing can be unevenly distributed, a soft-storey mechanism may develop and be the global failure mode.

The design approach has to consider the many possible situations by selecting some standard ones for which the behaviour can be properly assessed:

- If the infills are well-designed reinforced concrete panels well-connected to the frame, the structure is in fact a composite wall and has to be designed as such, according to *Section 7* of EN 1998-1.
- If infills are high-quality masonry panels intended to act as part of the seismic-resistant structural system, the structure should be considered as a confined masonry one, and designed as such, according to *Section 9* of EN 1998-1.
- If infills of whatever nature are structurally disconnected from the moment-resisting frame, which means that the infills are supported on the bottom beam with a horizontal gap under the top beam and vertical gaps on both sides, then they act only as a mass and make no contribution to earthquake resistance. Then, the openings $c$ of vertical gaps should be sufficient to prevent any contact between the infills and the structure: $c \geq d_r$, where $d_r$ is the interstorey drift at the ULS. A deformable material should be used to fill side and top gaps, for tightness. Prevention of out-of-plane movement of the structurally disconnected infill (possibly leading to overturning) may prove difficult.
- If infill panels consist of a material characterized by a low in-plane Young's modulus $E$ and a low yield resistance, the situation is close to total disconnection.
- If infills are made of panels having a very low in-plane resistance and/or crush immediately at their perimeter, the result is again close to total disconnection, because the panel material behaves like a deformable gap-filling material.

In practice, the last two statements have to be assessed by comparing:

- the order of magnitude of the shear resistance $V_{Rd}$ of the steel columns around the infill to the yield or failure strength $N_{Rd}$ of a compression diagonal made of the infill material
- the interstorey drift $d_r$ of the steel moment frame under the design seismic action, to the deformation d$l$ at maximum strength of the same compression diagonal.

One may consider the infills as structurally ineffective and ignore them, if $N_{Rd} < 0.05V_{Rd}$ and $dl > d_r$.

## 6.13. Control of design and construction

The requirement on control of design and construction of steel buildings in EN 1998-1 reflects the concern expressed in Section 6.3 about creating the conditions for an effective global dissipative behaviour of the structure. This goal requires in particular that the yield stress of all materials used in the dissipative components remains within well-defined ranges, notably below an upper bound value. The complete background of these requirements has been given in Section 6.3. *Clause 6.11*

Regarding problems with materials provided on site, three types of reaction can be envisaged:

(1) If the yield stress of the material of dissipative zones exceeds the standard upper bound of $1.1f_{y,\,max}$, the capacity design calculations in the design should be repeated, considering in these checks the real yield stresses of the materials.
(2) If the problem persists, a pushover non-linear analysis of the structure can be made to provide an explicit evaluation of the global plastic mechanism, including the first yield and the ultimate load, which may prove satisfactory, even if some criteria in EN 1998-1 are not fulfilled.
(3) Another option in the case of material overstrength is to reduce the sections of dissipative zones so that their real strength, $R_{d,\,act}$, comes closer to the design strength, $R_d$. In moment frames, this approach makes use of the reduced beam section design mentioned in Section 6.9. A similar approach can be used in other typologies of structures. In all cases, special care has to be taken to achieve a length of the yield zone large enough to provide the needed rotation/elongation/shear deformation capability. The section reduction technique also requires that the surface aspect of the 'shaved' sections be appropriate: rough surfaces produced by oxygen cutting, for instance, cannot stay rough and must be machined out.

CHAPTER 7

# Design and detailing of composite steel–concrete buildings

## 7.1. Introductory remark

*Section 7* on composite buildings in EN 1998-1 has a similar structure to *Section 6* on steel buildings. The information provided for steel buildings in *Section 6* applies to composite buildings as well. In the following, the emphasis is placed on those aspects which are specific to the composite character of structural elements and structural types.

*Clauses 7.1.2(5), 7.1.2(6), 7.7.5(1), 7.7.5(2)*

The section on composite buildings in Eurocode 8 has been developed on the basis of one general design objective in order to achieve ductile dissipative zones with reliable resistance: that the integrity of concrete is maintained during the seismic event and yielding takes place in the steel sections and/or in the rebars.

## 7.2. Degree of composite character

Any steel structure mixed with certain concrete structural components, such as concrete floors or walls, may be defined as a composite steel–concrete structure, since these materials interact to a certain extent. This interaction may be:

- Limited to the resistance to gravity loads or fire. The advantage of this option is that no seismic detailing is required; the structure behaves like a steel structure, and is analysed as such. However, the analysis of the structure should correspond to its real behaviour, and correct estimates of stiffness and strength of all structural elements must be made. In particular, the 'hidden' contribution of concrete to the strength of dissipative zones cannot be a neglected; otherwise, the capacity design of the zones which have to remain elastic will have a wrong basis. For this reason, care must be taken to 'disconnect' concrete in the vicinity of dissipative zones. Statements in *clause 7.7.5* refer to such a disconnection for beams of moment-resisting frames. There may be other cases without explicit indication given in the code, as stated in *clause 7.1.2(6)*.
- Such that the structural elements contribute by their composite character to dissipate the earthquake energy in the structure.

*Section 7* of Eurocode 8 has been developed for composite structures providing the highest possible composite resistance.[80,81]

## 7.3. Materials

*Clause 7.2.1(1)*

The first requirement in *clause 7.2* refers to the strength classification of structural concrete:

- The concrete should correspond to class C25/30 as a minimum; C25/30 is a standard value for structural applications in buildings, in particular for slabs.
- The concrete should correspond to class C40/50 as a maximum; the reason behind this restriction is the reduction of the crushing strain $\varepsilon_{cu}$ with increased concrete strength.

In developments of clauses for ductility of beam sections with a slab, $\varepsilon_{cu2} = 2.5 \times 10^{-3}$ has been considered as the crushing strain for reinforced concrete under seismic cyclic conditions; this may be taken to correspond to $\varepsilon_{cu2} = 3.5 \times 10^{-3}$ under static loading conditions. There is nothing preventing the use of a concrete class above C40/50, as long as the value $\varepsilon_{cu2} = 3.5 \times 10^{-3}$ is demonstrated under static conditions.

*Clauses 7.2.2(1), 7.2.2(2), 7.2.2(4), 7.2.3(1)*

The other requirements in *clause 7.2* refer to structural steel and the rebars, with reference to prescriptions for ductility similar to those of *Sections 5* (concrete buildings) and *6* (steel buildings). The simpler option in design is to use ductile materials; however, adverse situations are considered: *clause 7.2.2(4)* covers the situation in which non-ductile welded meshes are used in dissipative zones of composite structures. The rule is to duplicate the non-ductile rebars in dissipative zones by ductile ones of the same cross-sectional area and to rely on these last rebars to evaluate a safe-side plastic capacity. However, both the non-ductile and the ductile rebars present in the case of such a duplication should be used in the evaluation of the upper bound resistance of the cross-section, used as reference for the capacity design of other elements.

The problem covered by this complicated statement is that a reliable negative plastic moment in the beam-to-column connection zone in moment frames can only be based on reinforcement with guaranteed ductility, while the beam plastic moment considered in the capacity design of the column must include all possible contributions of the reinforcements, non-ductile welded mesh included. When duplication of non-ductile reinforcement by means of ductile reinforcement is realized, the capacity design of columns produces an overdesign of these columns.

In practice, the most economic solution can be obtained either by:

- using ductile welded mesh or
- avoiding the continuity of non-ductile reinforcements in dissipative zones; this can be done by using standard ductile rebars there, and by placing the overlap between ductile and non-ductile reinforcement away from the dissipative zone.

## 7.4. Design for local energy dissipation in elements and their connections

### 7.4.1. Favourable factors for local ductility due to the composite character of structures

The use of composite steel–concrete frames can have positive effects on local ductility; these effects are in addition to the phenomena described in Section 6.4 for steel structures:

*Clauses 7.6.1(4), 7.6.4(8), 7.6.4(9), 7.6.4(10), 7.6.5(3), 7.6.5(4), 7.6.5(6)*

- The positive effect of concrete encasement around steel profiles. Concrete encased in a profile, or between the flanges of a profile, prevents inward local buckling of steel walls and reduces strength degradation due to buckling. For this reason, some limits of wall slenderness for composite sections are higher than those of pure steel sections. The limit values for web slenderness of H sections are higher by one class than those for steel sections, as indicated in Eurocode 4,[82] to which the basic *clause 7.1.1(1)* refers, provided that the steel web is connected to the concrete in the way specified in clause 5.5.3(2) of Eurocode 4. For partially or fully encased H sections, the limit values of flange slenderness given in *Table 7.3* are the same as those of sections of class 1, 2 and 3 in Table 5.2 of Eurocode 4,[82] which in turn are the same as in Table 5.2 of Eurocode 3[83] for steel

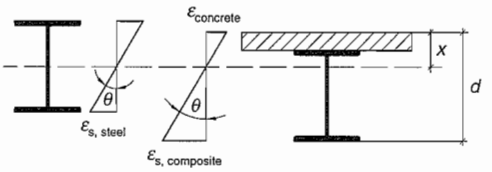



**Fig. 7.1.** Strains in steel and composite sections

sections. Fully encased beams are at present outside the scope of Eurocode 4; only partially encased beams are considered. Because of the lack of data, it was not possible in Eurocode 8 to make a distinction between partially and fully encased beam sections. In addition to the standard situations envisaged in Eurocode 4, Eurocode 8 introduces the possibility of increasing the limits of flange slenderness by mitigating the buckling of flanges of encased H profiles. The limits can be increased by up to 50% in certain cases by means of specific measures:

- additional stirrups for fully encased profiles (see *clauses 7.6.4(9)* and *7.6.4(10)*)
- additional straight bars welded to the inside of the flanges for partially encased profiles (see *clauses 7.6.5(4)* and *7.6.5(6)* and *Fig. 7.8* in EN 1998-1).

These details can improve the design at relatively low cost, since the additional rebars or stirrups need be present only in the critical regions of columns or in the length of dissipative zones of beams, which are of the order of the beam depth.

- The positive effect of steel plates and sections encased in a concrete wall. Consideration of steel inside concrete in this way may seem like a repetition of the previous case of the positive effect of concrete encasement around steel profiles. There is a difference, though, in the reference structural element, which is a concrete wall. By transforming a concrete wall into a composite one, the designer can significantly improve the ductility and strength of the wall and solve typical design problems such as providing a higher shear resistance within concrete dimensions limited by architectural considerations. *Sections 7.10* and *7.11* of EN 1998-1, which are presented in Section 7.15 below, provide design guidance for reinforced-concrete walls composite with steel plates and sections. *Clauses 7.10, 7.11*
- Increased damping, in comparison with steel structures, due to cracking and to friction at steel–concrete interfaces. Although commonly recognized, this positive effect on energy dissipation is not explicit in the design process because it is considered that, at the ultimate limit state (ULS), the energy dissipation by damping due to structural elements is secondary with respect to the energy dissipated in plastic mechanisms.

### 7.4.2. Unfavourable factors for local ductility due to the composite character of structures

The use of composite steel–concrete frames can have some adverse affects on local ductility; these are in addition to the phenomena described in Section 6.4 for steel structures: *Clauses 7.6.2(1), 7.6.2(7), 7.6.2(8), 7.6.4(2), 7.6.5(1)*

- Concrete crushing in compression. Concrete failure in compression is not ductile. Many aspects of *Section 7* of Eurocode 8 aim at defining conditions to avoid such failure by keeping stresses and strains in concrete below their values at failure:
  - The limit values of $x/d$ in steel beams with a slab defined in *clauses 7.6.2(7)*, *7.6.2(8)* and in *Table 7.4*, which limits the position of the neutral axis in a composite T beam of moment frames in order to keep the maximum strains $\varepsilon_{concrete}$ in the slab below an acceptable value (Fig.7.1).
  - The design conditions of the rebars of the slab presented in *Annex C* of EN 1998-1, which are defined with the twofold objectives of: *Clauses 7.5.4(5), 7.6.2(9)*
    (a) taking advantage of the concrete strength in order to maximize the potential of resistance of sections made of a steel H beam composite with a concrete slab, by defining steel reinforcement of the slab which works as 'ties' to equilibrate concrete compression 'struts'

(b) mitigating slab cracking around columns in moment-resisting frames, by creating a concrete-confining effect.

*Clauses 7.5.4(7), 7.6.4(7), 7.6.5(2)*

- Concrete disintegration under alternate shear. Concrete failure under alternate shear is not ductile, because alternate cracks may rapidly cause complete disintegration of concrete, reducing concrete strength to practically nil. This explains why the resistance $V_{wp,\,Rd}$ of the panel zone of composite columns, which can include the contribution of concrete compression struts inside the 'box' formed by the column flanges and the transverse stiffeners, must be capacity designed to the beam plastic strength in bending by complying with *equation (7.3)* in EN 1998-1:

$$V_{wp,\,Ed} \leq 0.8V_{wp,\,Rd} \qquad \text{(D7.1)}$$

The analogous condition for steel structures is equation (D6.6) in Chapter 6: $V_{wp,\,Ed} \leq V_{wp,\,Rd}$. It is clear that increased safety is required against panel 'failure' in the case of composite columns than in the case of steel columns, precisely in order to keep the shear stresses in the panel zone below the yield stress of steel and below the ultimate stress of concrete.

The dramatic decrease in the shear strength of sections subjected to plastic bending and in which concrete fails under alternate shear also justifies that, in dissipative zones of columns made of an encased steel profile, no confidence is given to the shear resistance of the composite section. *Clauses 7.6.4(7)* and *7.6.5(2)* express this, by requiring that the shear resistance of the steel section alone should be considered.

*Clauses 7.6.2, 7.7.1(4)*

- Negative impact of the rise of the neutral axis in beams composite with a concrete slab. In composite beams made of a steel H section with a concrete slab connected by shear connectors to the steel section, the neutral axis is raised to the upper part of the section (currently the steel flange), which involves increased strains $\varepsilon_{s,\,composite}$ in the bottom flange of the steel section, in comparison with the strains $\varepsilon_{s,\,steel}$ developed at equal rotation in a symmetrical steel section (Fig. 7.1). These higher strains cause faster strength degradation due to buckling, and accordingly reduce the ductility of the section. This effect is considered by values of the limits of wall slenderness *c/t* of webs which are more restrictive for webs fully in compression (in beams with a slab) than for webs in bending (in symmetrical steel sections). The limit values of the wall slenderness *c/t* for flanges remain unchanged.
- Negative impact of the increase in effective resistance of composite girders. The composite action significantly increases the effective strength of steel girders, particularly in sections where curvature of the girder places the top flange into compression. It is of primary importance that capacity design of non-dissipative structural elements is based on the values of the composite strength, which is the case in EN 1998-1. Design ignoring the effective composite strength of girders can result in structural problems. For instance, in moment-resisting steel frames, the development of a global 'weak column–strong beam' mechanism or a local 'weak panel zone' condition, or the reduction of the effectiveness of reduced beam section design. It has been evaluated that the distress in moment connections in the Northridge (1994) earthquake was partly explained by the higher stress and strain demand in the bottom flange of composite beams.

## 7.5. Design for the global dissipative behaviour of structures

### 7.5.1. Behaviour factors of structural types similar to steel

*Clauses 7.3.1, 7.3.2*

The values of *q* factors for composite structures are the same as in steel structures with the same structural system, namely moment-resisting frames,[84] or frames with concentric or eccentric bracings, as given in *Table 6.2* in EN 1998-1. There are, however, some limitations for these types of composite structures, which deserve an explanation. These limitations are:

- Braces of composite concentrically braced frames cannot be composite (*clause 7.3.1(1)b*). Assessing the real behaviour of a concentrically braced frame in a simple way is not easy, as explained in Section 6.10. Using composite brace members in which the concrete is effective under compression only, would make the design even more complex. In a simplified analysis, the resistance of the diagonals under compression is neglected (see Section 6.10), and only the diagonals which are in tension are assumed to resist the lateral earthquake loads. However, in the capacity design of the entire system the diagonals which experience compression under seismic loading need to be considered. As explained in Section 6.10, if the buckling capacity of a diagonal under compression at a certain storey is more than 50% of the tensile capacity $N_{pl,Rd}$ of the other diagonal at that same level, the columns will undergo larger axial forces than considered in the simplified analysis. This justifies the lower bound limit on $\bar{\lambda}$ in *clause 6.7.3(1)* of EN 1998-1. Introducing a composite brace would increase the difficulty to design a slender diagonal member complying with $\bar{\lambda} > 1.3$. *Clause 7.3.1(1)*
- *Clauses 7.3.1(1)(c)* and *7.9.3(1)* require that pure steel sections are used to realize seismic links in composite eccentrically braced frames (they can possibly be composite with slabs). The links may not include encased steel sections. *Clause 7.9.1(1)* requires them to be designed such that the dissipative action occurs essentially through yielding in shear of the links. *Clauses 7.3.1(1), 7.9.3(1), 7.9.1(1)*

  Capacity design requires a correct evaluation of the plastic resistance of dissipative zones. In long links, in which plastic bending is developed, plastic rotations are larger than in moment frames undergoing the same global displacement. The experimental background which would allow reference to the composite resistance of composite long links is missing. An initial composite resistance is certainly developed, probably followed by a concrete deterioration which leads to a link with the steel section capacity alone so that two values of link resistances should probably be considered:
  - the composite resistance, which would serve in the capacity design of braces, beams and columns
  - the steel resistance, which would serve to evaluate the global resistance of the structure at ULS.

  However, there are uncertainties for both of these values. Disconnection of the slab in order to refer to a pure steel section in bending raises similar questions, because it is not obvious that a local disconnection allows a reference to steel resistance only. Indeed, experiments in the context of moment frames have shown that disconnections of a very local character have a limited effect: the composite resistance is still there. The conclusion is that links working in bending in beam elements with a slab raise serious questions and cannot be designed in a reliable way.

  In beams, only short links, made of an unencased steel section possibly with a slab, working in shear correspond to a well-controllable situation, because:
  - the plastic resistance in shear of the steel section can be reliably computed
  - the slab contribution in the shear resistance of the links is negligible.

  Vertical steel links also correspond to a controllable situation.

### 7.5.2. Behaviour factors of composite structural systems

Concrete walls with encased steel profiles or steel plate components organized in different ways are typical composite wall structural systems. Their values of $q$ are basically the same as in concrete wall systems, with an increased value for systems in which steel or composite beams frame into the walls, because in these cases there is energy dissipation in both the wall and in the beams. *Clauses 7.3.1(1), 7.3.2(1)*

The most interesting and attractive feature of composite walls is not the higher value of $q$ but their higher shear and bending resistance and the higher stiffness that they can provide for a given wall section. These qualities can help in solving various practical problems, by:

- providing the aforementioned resistance and stiffness characteristics with smaller cross-sectional dimensions
- providing continuity and regularity in stiffness and strength in elevation, in cases where the wall dimensions have to be small at a certain storey.

## 7.6. Properties of composite sections for analysis of structures and for resistance checks

### 7.6.1. Difficulties in selecting mechanical properties for design and analysis

*Clauses 7.4.2, 7.5.3*

The mechanical properties of structural elements and structures made of two structural materials depend on the properties of each material and on the interconnection between them. In particular, in the case of steel–concrete composite elements, the following should be considered:

- Concrete offers resistance to compression, but not (reliably) to tension. This implies that both stiffness and resistance at yield of composite sections depend on the sign of the stresses in the concrete. As a result, two values are defined for each mechanical property of sections in structural elements in which there is a reversal of the sign of the bending moment and/or normal force. This contributes to the apparent complexity of design for composite structures.
- The evaluation of the global resistance of a structure should be based on the nominal strengths of the materials, which are lower than the mean values. Capacity design of zones which are to remain elastic should, in contrast, normally be based on estimates of the real resistances of dissipative zones, which are average or upper bound values; this results, in some cases, in two values for the resistance of sections, contributing to the apparent complexity of seismic design of composite structures.
- The periods of a structure depend on its stiffnesses. Considering a lower bound value of the modulus $E$ of concrete causes underestimation of the stiffness, an overestimation of the periods and of seismic displacement demands and underestimation of the force action effects (base shear, etc.).
- In the seismic context, there are types of mechanical stress, such as alternate shear, which induce fast degradation of the concrete; this situation results in the necessity of considering non-composite resistances of some sections.
- The value of the maximum resistance to compression of a given concrete varies widely, depending on the confinement achieved in the most stressed zones; $f_{cd}$ is the design value of strength considered in resistance calculations, but the design checks may implicitly include the favourable influence of confinement at the ULS.
- In sections working in a composite way, there is generally shear transfer at the concrete-to-steel interface; in dynamic cyclic conditions, requirements for the effectiveness of this transfer may be higher than under static loading, because, for instance, there may not be strong confidence in high bond resistance. This will imply that in certain cases more shear connectors are required in seismic design than in static design.

### 7.6.2. Stiffness of composite sections

*Clauses 7.4.2(1), 7.4.2(2), 7.4.2(3), 7.4.2(4), 7.4.2(5)*

In structural elements in which there is concrete in compression, the effective equivalent steel section is calculated assuming that the concrete in compression is uncracked. This is reflected in the value for the modular ratio $n = E_a/E_{cm} = 7$. In structural elements in which the concrete is in tension, the effective equivalent steel section is calculated neglecting the concrete in tension and considering only the reinforcement. As a result, the stiffness of the section can be different, depending on the sign of the seismic action effect. For example, in composite beams with a slab under positive (sagging) bending, the second moment of area $I_1$ ('moment of inertia') of the effective equivalent steel section involves all steel components (steel profile plus slab rebars) and the concrete (in compression). Under negative moments,

the second moment of area $I_2$ involves only the steel profile and the rebars of the slab. As, in addition, the effective width of the slab is different for positive (sagging) and negative (hogging) moments, $I_1$ and $I_2$ are generally different. $I_1$ and $I_2$ could be equal if and only if the equivalent section of concrete of the slab is equal to the sections of the rebars, both sections being computed with their respective effective widths under sagging and hogging bending and both sections having their centre at the same level.

For composite beams with a slab, the effective width of a slab needed to compute $I_1$ (for the sagging moment) and $I_2$ (for the hogging moment) are defined in *Table 7.5 I* of EN 1998-1.

The different values of $I_1$ and $I_2$ raise a practical problem in the analysis of a structure, as there are zones in structural elements which are subjected to positive bending, with other zones subjected to negative bending. The structural model for the analysis has to allocate the values $I_1$ and $I_2$ to these zones, which implies more elements in the model and difficulty in defining the length of sagging and hogging zones. Fortunately, a simpler alternative is provided in Eurocode 8, which allows computation of an 'equivalent' second moment of area $I_{eq}$, which is constant for the entire span. *Clauses 7.7.2(3)* and *7.7.2(4)* define $I_{eq}$ for beams and $(EI)_c$ for columns, respectively.

*Clauses 7.7.2(2), 7.7.2(3), 7.7.2(4)*

### 7.6.3. Effective width of slabs

*Tables 7.5.I* and *7.5.II* EN 1998-1 for the effective width $b_e$ of the slab in composite beams have been established for moment-resisting frames with rigid connections in which:

*Clauses 7.6.3(1), 7.6.3(2), 7.6.3(3)*

- The local plastic mechanism in dissipative zones is such that the integrity of the concrete slab is maintained during the seismic event. In fact, the values provided for $b_e$ achieve that goal in the first stage of plastic rotations, but, as the rotations increase, some degradation of concrete adjacent to the column may occur. However, the resistance of the elements in bending is kept constant due to other factors: strain hardening of steel, other resistance mechanisms in the slab, strength of the concrete around the column higher than $f_{cd}$ due to confinement, etc.
- Yielding takes place at the bottom part of the steel section and/or in the rebars of the slab. Extensive experimental and numerical work provided the background to the Eurocode 8 values of effective width, and showed that they have to be different for the elastic and the ultimate (plastic) behaviour: the values for computing $M_{Rd}$ are 2–3 times higher than those used to compute $I$. As detailed in *Annex C* of EN 1998-1, other influences have also been considered, such as the presence of 'transverse' beams (perpendicular to the beam for which an effective width is being defined), the type of façade transverse beam, the design of rebars connected to these façade beams, and the sign of the bending moments at beam ends.

## 7.7. Composite connections in dissipative zones

The following design objectives guided *clause 7.5.4* of EN 1998-1 for composite connections:

*Clauses 7.5.4, 7.5.2(3), 7.5.2(4), 7.5.2(5)*

- the integrity of the concrete slab should be maintained during the seismic event
- yielding takes place in the steel section and/or in the rebars of the slab.

The requirements and design calculations for the connections of steel parts to steel parts are identical to those for steel structures. In particular, connections can be of the partial- or full-strength type, according to *clause 7.5.2(3)*. Depending on the chosen option, *clause 7.5.2(4)* or *7.5.2(5)* applies as the rule for the capacity design of adjacent structural components.

The most specific design rules on composite connections in Eurocode 8 refer to conditions to be fulfilled in order to take advantage of the composite resistance of the panel zone in beam-to-column connections of moment-resisting frames. This is achieved by defining the dimensions of the walls of a panel zone 'box' which is filled by concrete. This composite box possesses higher resistance and stiffness than the reference steel panel zone, due to effective

*Clauses 7.5.4(7), 7.5.4(8), 7.5.4(9), 7.5.4(10)*

**Table 7.1.** Limits of wall slenderness in steel members of composite buildings, for different sections, design details and behaviour factor $q$ (note: $\varepsilon = (f_y(\text{MPa})/235)^{1/2}$)

| | Ductility class of structure | | |
|---|---|---|---|
| | DCM | | DCH |
| | Reference value of behaviour factor ($q$): | | |
| | $q \leq 1.5$–2 | 1.5–2 $< q \leq 4$ | $q > 4$ |
| **Flange outstand limits $c/t_f$** | | | |
| H or I section in steel or partially or fully encased | $20\varepsilon$ | $14\varepsilon$ | $9\varepsilon$ |
| Partially or fully encased H or I section plus hoops or straight links | $30\varepsilon$ | $21\varepsilon$ | $13.5\varepsilon$ |
| **Web width to thickness limit $c/t$** | | | |
| Partially or fully encased H or I section | $42\varepsilon$ | $38\varepsilon$ | $33\varepsilon$ |
| Partially or fully encased H or I section plus web connected | $42\varepsilon$ | $42\varepsilon$ | $33\varepsilon$ |

diagonal concrete compression struts activated when the box starts deforming in shear. The confinement of concrete delays its cracking and crushing, so that the contribution of concrete to strength is significant. This typical composite aspect of design can obviate the necessity of costly 'doubler' steel plates welded in the panel zone, as sketched in Fig. 6.10. *Clauses 7.5.4(7)* and *7.5.4(8)* consider, respectively, the cases of fully encased panel zones and of partially encased zones.

Different types of beams framing into different types of columns are envisaged:

- Steel beams, composite fully encased beams and composite partially encased beams.
- Composite partially encased columns, composite fully encased columns (*clause 7.5.4(10)*) and reinforced-concrete columns (*clause 7.5.4(9)*). In the last option, the vertical stiffeners of the steel beam placed in the plane of the outside face of the column and contributing significantly to the shear resistance of the panel zone are called 'face-bearing plates'; they are subject to the detailed rules of *clause 7.5.4(9)*.

## 7.8. Rules for members

*Clauses 7.6.1(4), 7.6.4(9), 7.6.4(10), 7.6.5(4), 7.6.5(5), 7.6.5(6)*

Some general rules are valid for all types of members: beams, columns or diagonals.

For members in tension, reference is made to the rule of Eurocode 3 for steel sections;[83] the effect of drilled holes has already been explained in Section 6.7.

The dissipative character of composite structural elements first imposes limitations on the slenderness of walls of the steel sections in relation to the ductility class intended for the structure. These limits are defined in:

- Eurocode 4[82]
- *Table 7.3* in EN 1998-1, which provides the limits for flange outstands to be met, for a structure to belong to a ductility class.
- *Clauses 7.6.4(9)* and *7.6.4(10)* for fully encased profiles and *clauses 7.6.5(4)* to *7.6.5(6)* for partially encased profiles, allowing for an increase in the limits of flange slenderness of encased H profiles by mitigating their buckling. The limits can be increased by up to 50% by means of specific measures: additional stirrups for fully encased profiles and additional straight bars welded to the inside of the flanges for partially encased profiles (see *Fig. 7.8* in EN 1998-1).

Table 7.1 summarizes the values of wall slenderness corresponding to different composite design options for columns made of H or I sections. In this table, flanges and webs are

assumed to be fully in compression; 'plus web connected' refers to clause 5.5.3(2) in Eurocode 4,[82] 'plus hoops or straight links' refers to *clauses 7.6.4(9)*, *7.6.4(10)*, *7.6.5(4)* and *7.6.5(5)* of EN 1998-1.

## 7.9. Design of columns

### 7.9.1. Design options

*Clauses 7.6.1(9), 7.6.1(8), 7.6.1(10), 7.6.1(11), 7.5.4(12), 7.6.1(13), 7.6.6(3)*

There are three design possibilities for composite columns:

- Non-dissipative composite columns. However, due to the seismic context, these have to comply with some specific rules (discussed below), in addition to the rules of Eurocode 4.
- Dissipative composite columns.
- Composite columns considered as steel columns in the design.

### 7.9.2. Non-dissipative composite columns

The majority of column members are non-dissipative, because the intended global dissipative mechanisms of structures involve as little energy dissipation in columns as possible. For this reason, only their elastic response needs to be assured, which is done essentially by complying to Eurocode 4. There are, however, differences between a standard design to Eurocode 4 and the earthquake context.

The origin of the first difference is in the 'cyclic' aspect of the response, which can degrade the shear strength $\tau_{Rd}$ due to bond and friction. This is accounted for by *clause 7.6.1(11)*, which prescribes reducing by 50% the values of $\tau_{Rd}$ in Table 6.6 of Eurocode 4.[82]

A second difference is due to the uncertainties in the evaluation of earthquake action effects, which are covered by a specific requirement on the dimensions of fully encased columns: they should not be less than 250 mm (as stated in *clause 7.6.1(8)*).

A third requirement defined in *clause 7.6.1(12)* corresponds to the situation in which a column is capacity designed to the strength of another structural element. In such cases, it is vital that at the locations of internal force transfer the column provides its full composite resistance. Otherwise, a local 'connection zone' failure may take place, and the intended global plastic mechanism will not be achieved. This requires that the action effects, axial force, shear and bending moment in columns are effectively shared between the concrete and the steel parts in accordance with the hypotheses made in the analysis and the verifications of the structure. This can be assured only if the complete shear transfer required in *clause 7.6.1(12)* is ensured between the steel and the reinforced-concrete parts.

For example:

- In a frame with concentric bracings, columns are capacity designed to the strength of the diagonals. In the columns of braced frames, axial forces are more significant than bending moments; their response should be elastic. It must be assured that the axial force in composite columns is shared between the concrete parts and the steel parts, in particular in zones where the column axial force is highest; this is the case at floor levels where bracing members and beams are connected to the columns. The bond demand in those places is high and strength may easily be insufficient if shear connectors are not provided close to the connection of the column to the diagonal.
- In a moment-resisting frame, columns are capacity designed for beam strength. The composite connection zones in moment-resisting frames have to carry the beam bending moment and shear, and distribute it between the steel and concrete of the column. *Clause 7.5.4(10)* on composite connections provides guidance on how the beam shear has to be distributed to the components of the column. Some shear connectors may be required close to the beam–column connection. For filled columns, shear transfer can be achieved through the connection detail in the interior of the column.

It should be noted that *clause 7.6.1(12)* allows a column without shear connectors to be designed as composite, whenever the seismic action effects are not too high.

Note that in moment-resisting frames with filled columns, it has been shown that the global response does not vary significantly with the stiffness or strength of the shear connection achieved through bond and friction. This may be due to the fact that in filled columns, compatibility of deformations of steel and concrete is achieved by stresses perpendicular to the interface between steel and concrete, and friction is always ensured where interface stresses are compressive. This cannot be the case in open sections, such as H or I.

All the design requirement for non-dissipative composite columns given above are also valid for dissipative columns, in addition to the specific criteria discussed in the following section.

### 7.9.3. Dissipative composite columns

*Clauses 7.6.1(10), 7.6.1(12), 7.6.4(1), 7.6.4(2), 7.6.4(3), 7.6.4(4), 7.6.4(5), 7.6.4(7), 7.6.4(8), 7.6.5(1), 7.6.5(2), 7.6.5(3), 7.6.6(1), 7.6.6(2)*

Dissipative composite columns need to be detailed to ensure an adequate cyclic plastic response.

The only location where columns will certainly be called on to dissipate energy in the intended plastic mechanisms is at the bottom of ground-storey columns of moment-resisting frames or of certain frames with eccentric bracings. At other levels of the same structural systems, the requirement for energy dissipation capacity is confined to critical regions, to cover uncertainties in the exact structural response independently of the intended response.

In other types of structural systems, there is normally no plastic energy dissipation in columns.

EN 1998-1 includes design provisions for dissipative columns belonging to three cross-section types: fully-encased (*clause 7.6.4*), partially encased (*clause 7.6.5*), filled rectangular and circular sections (*clause 7.6.6*).

The dissipative character of composite columns imposes limitations first on the slenderness of the walls of the steel sections, depending on the ductility class of the structure, as explained in Section 7.8.

The second limitation for dissipative composite columns refers to the local composite plastic mechanism: it should be ductile, which is possible only if a flexural plastic hinge develops, and not through shear deformations. This is stated in *clause 7.6.4(7)* for fully encased profiles, in *clause 7.6.5(2)* for partially encased ones and in *clause 7.6.6(2)* for filled composite columns. This has also been discussed in Section 7.4. A consequence of this requirement is that the design shear in the plastic hinge zone of composite columns is derived from the plastic moment of that column.

The third difficulty in achieving dissipative composite columns comes from the fact that the concrete encasement or infill has to contribute fully to the flexural resistance of the member, as well as to its axial resistance. It must thus be checked that a full shear transfer between the concrete and steel parts is achieved, as required by *clause 7.6.1(10)*. If transfer of shear through bond stress and friction is insufficient, *clause 7.6.1(12)* requires the provision of shear connectors.

Another specific requirement addresses fully encased columns only, which are similar to reinforced-concrete columns. The rule in *Section 5* of EN 1998-1 for reinforced-concrete columns considers critical regions at the top and bottom of all columns, where more tranverse reinforcement in the form of hoops is required to protect the concrete in those zones and to prevent buckling of longitudinal bars. The same requirements apply to fully encased composite columns of the same ductility class.

### 7.9.4. Composite columns considered as steel columns in the model used for analysis

*Clauses 7.5.3(3), 7.5.3(4), 7.6.1(7)*

If a composite column is considered as a steel column in the structural model, it is a dissipative member, but with only the resistance of the steel parts considered in the dissipative zones. The advantage of this option is that all requirements in *Section 7* of

EN 1998-1 for composite columns can be ignored. However, the general requirement in *clause 7.5.3(4)* still applies: the capacity design of connections or of the foundation of that column has to based on the upper bound value of the resistance of the column, i.e. its composite resistance as a result of the presence of concrete.

## 7.10. Steel beams composite with a slab

### 7.10.1. Ductility condition for steel beams with a slab under a sagging (positive) moment

*Clauses 7.6.2(1), 7.6.2(7), 7.6.2(8)*

The ductility condition for steel beams with a slab subjected to sagging moments complies with the general objective of achieving ductile dissipative zones with a reliable resistance: the integrity of concrete is maintained during the design seismic event and yielding takes place in the steel section and/or in the rebars. The problem of ductility appears only if the concrete might reach its crushing strain in compression in a brittle way due to the application of positive (sagging) bending moments. Under negative (hogging) moments, there is no such problem, because the tensile strength of concrete is neglected and the composite section thus behaves like a section without concrete.

Considering the distribution of strains under a positive bending moment in a composite section consisting of a steel beam with slab, the ductility objective can be reached if, at the ULS, the concrete strain at the top fibre is below the concrete crushing strain, $\varepsilon_{cu2}$, while significant total strain $\varepsilon_a$ develops at the bottom fibres (steel). Geometrical considerations in Fig. 7.1 define the corresponding limit position $x/d$ of the neutral axis (*equation (7.4)* in EN 1998-1). All $x/d$ values below that limit achieve the ductility condition $\varepsilon < \varepsilon_{cu}$, specified in *clauses 7.6.2(7)* and *7.6.2(8)*.

The values in *Table 7.4* in EN 1998-1 have been established in the following way:

- for concrete under cyclic conditions, $\varepsilon_{cu} = 2.5 \times 10^{-3}$ (a fairly conservative value)
- considering, for example, a beam made of S355 steel in a Ductility Class Medium (DCM) structure with $q = 4$,

$$\varepsilon_a = q\varepsilon_y = qf_y/E = 4 \times 355/205\,000 = 6.92 \times 10^{-3} \Rightarrow x/d = 0.27$$

The values in *Table 7.4* have been calibrated through cyclic tests on beam–column subassemblages and in three-dimensional tests.

*Clauses 7.6.2(4), 7.6.2(5), 7.6.2(6)*

When a profiled steel sheeting with ribs transverse to the supporting beams is used, the reduction factor $k_t$ of the design shear resistance of connectors given in Eurocode 4[82] should be reduced further by the rib shape efficiency factor $k_r$ mentioned in *clause 7.6.2(6)*. This reduction takes into account the uplift forces induced at the ULS in the slab by a steel deck with trapezoidal shape and a positive slope $\alpha$. These uplift forces may cause concrete failure over a cone around the connector, at an applied shear less than the design shear resistance.

### 7.10.2. Ductility condition for steel beams with a slab under a hogging (negative) moment

*Clause 7.6.1(4)*

Sufficient local ductility of steel members which dissipate energy working in compression or bending is ensured by restricting the slenderness of the walls of the section, as explained in Section 7.8. The limit value of the $c/t$ ratio of a web subjected to bending and compression depends on the slenderness of the part in compression, which depends in turn on the position of the plastic neutral axis in the section.

For composite sections consisting of a steel profile and a slab, the critical situation for the determination of the class of the walls of the steel section is the presence of a negative (hogging) bending moment. This essentially reduces to the determination of the class of the lower flange of the steel section, which is fully in compression, and to the determination of the class of the web, which is in bending. The position of the plastic neutral axis of the composite section under a negative moment determines the slenderness of the part of the

web in compression and consequently its class. The position of the neutral axis is directly linked to the cross-sectional area of the reinforcement within the effective width of the slab. With an increase of the amount of reinforcement in the effective width, the neutral axis rises within the section, and the limit slenderness of the web in compression increases. The ductility requirement for the steel web thus leads implicitly to a condition on the cross-sectional area of rebars: $A_S < A_{\text{limit class I}}$ within the effective width $b_{\text{eff}}$.

The higher the desired class of the section, the lower is the value of $A_{\text{limit}}$ which enables a section to belong to a given class. *Table 7.3* of EN 1998-1 does not provide directly applicable guidance on this design problem. The more complete Table 7.1 in Section 7.8 gives the worst-case limits for the slenderness $c/t$ of webs of H or I sections, which correspond to a web being fully in compression when the section is subjected to a hogging plastic moment.

### 7.10.3. Seismic reinforcement in the concrete slab in moment-resisting frames

*Clauses 7.5.4(5), 7.6.2(9), 7.6.3(1)*

Under the design seismic action, the bending moment diagram of the beams of moment-resisting frames typically has the shape shown in Fig. 6.8. At the ULS this implies that:

- Both the positive and negative plastic bending moments of the beam section are reached at the beam ends.
- A complete reversal of moments takes place across the interior joints.
- the beam–column connection zone must be designed for the ability to transfer these large action effects due to these moments; to ensure this, it is certainly not enough to have a steel frame with some encasement by concrete or slabs. Special design measures are needed.

Eurocode 8 provides the two necessary pieces of information necessary for such a design:

- The effective width of the slab in the beam–column connection zone, as related to different designs, is provided in *Table 7.5.II* of EN 1998-1; this allows the value of $M_{\text{pl, Rd}}$ of beams to be computed.
- A design method for the slab reinforcement in composite beams with the slab at the joints of moment-resisting frames ('seismic rebars' of the slab) is presented in *Annex C* of EN 1998-1. It allows dimensioning of the rebars (cross-sectional area, anchorage) and of the connectors to the beams, both for positive and negative bending moments.

The design proposed in *Annex C* is based on the consideration of the following force paths.

Under a positive moment in the beam at the beam–column connection, the three mechanisms of transfer of the compression force $F_{\text{Sc}}$ of the effective width of slab to the column shown in *Fig. C2* of EN 1998-1 can take place. They are necessary, alone or in conjunction, in order to develop the full composite positive moment $M_{\text{pl, Rd}}$ of beams. They can be described as follows:

- **Mechanism 1**: direct compression on the column. The design force resistance developed by this mechanism cannot exceed the value $F_{\text{Rd1}} = b_b d_{\text{eff}} f_{\text{cd}}$, where $d_{\text{eff}}$ is the overall depth of the slab for solid slabs, or the thickness of the slab above the ribs of the profiled sheeting for composite slabs, and $b_b$ is the bearing width of the concrete slab on the column (the column width, possibly extended). *Figure 7.7* of EN 1998-1 presents the bearing width $b_b$ for different column configurations.
- **Mechanism 2**: compressed concrete struts inclined to the sides of the column and transferred to the column with the help of the roughness created by indentations of its side surfaces. If the inclination of the struts is assumed to be 45°, the design force resistance developed by this mechanism cannot exceed the value $F_{\text{Rd2}} = 0.7 h_c d_{\text{eff}} f_{\text{cd}}$, where $h_c$ is the depth of the steel section of the column.
- **Mechanism 3**: when a transverse beam is present, a force transfer involving the façade steel beam is activated. The design force resistance developed by this mechanism cannot

exceed the value $F_{Rd3} = nP_{Rd}$, where $n$ is the number of connectors within the effective width of the slab and $P_{Rd}$ is the design resistance of one connector.

To be effective, each mechanism requires certain conditions to be fulfilled:

- mechanism 1 requires anti-bursting transverse reinforcements in the immediate vicinity of the column face
- mechanism 2 requires the presence of transverse reinforcement to act as ties to equilibrate the concrete compression struts at some distance from the column face – these ties should be long enough to cover the inclined 'struts' on both sides of the column.

Under a negative moment in the beam at the beam–column connection, the plastic tensile force $F_{St} = A_s f_{yk}$ in the longitudinal rebars within the effective width of the slab needs to be anchored in order to develop the full composite negative moment $M_{pl,Rd}$ of the beams.

Three design possibilities exist for anchorage of the rebars of the slab of beams attached to an exterior (façade) column. These possibilities are shown in *Fig. C1(c)–(e)* of EN 1998-1. They are as follows:

- A reinforced-concrete cantilever edge strip, as in the only detailing considered for this case in Eurocode 4, constitutes the façade beams. Then, anchorage is achieved through horizontal hairpins, which mobilize the compression of the concrete against the back face of the column and the concrete compression struts. For ductility, the compression stresses should not be high enough to cause crushing of the concrete and be the weak link in the mechanism.
- The façade includes beams and columns. Then, anchorage of the slab rebars is achieved through bars bent around shear connectors at the façade steel beams.
- A combination of the two solutions above, in which both hairpins and bars bent around connectors are used.

At an interior joint, the moment transfer may involve transfer of the full positive plastic moment from one side and of the full negative plastic moment from the other. In this case, the plastic tensile force $F_{St}$ in the longitudinal rebars within the effective width of slab on one side of the column is added to the compression force $F_{Sc}$ of the effective width of slab on the other side. As the transfer capacity relies on the transfer of compression from the slab to the column, and as the only available mechanisms are mechanisms 1, 2 and 3 described above, a limit can be reached if the resistance provided by mechanisms 1, 2 and 3 is less than the sum of the action effects: $F_{St} + F_{Sc}$. The condition to check is given by *expression C.18* in EN 1998-1:

$$1.2(F_{St} + F_{Sc}) \leq F_{Rd1} + F_{Rd2} + F_{Rd3} \qquad \text{(D7.2)}$$

Though not required by Eurocodes 4[82] or 8, the rebars of the slab should preferably be positioned under the level of the head of the connectors, as this has two positive effects:

- homogenization of the displacement of connectors and better correspondence of reality to the design hypothesis of uniform resistance provided by the connectors
- prevention of slab uplift.

## 7.11. Design and detailing rules for moment frames

### 7.11.1. General

As for moment-resisting frames made of concrete or steel, the design objective for composite steel–concrete moment frames is that plastic hinges form in the beams and not in the columns. This requirement is waived at the base of the frame, at the top floor of multi-storey buildings and in one-storey buildings. The requirement is checked by fulfilling equation

*Clause 7.7.1(1)*

(D4.23). The explanation of this check given in Section 6.9 applies to composite steel–concrete moment frames as well.

### 7.11.2. Analysis and design rules for beams, columns and connections

*Clauses 7.7.2, 7.7.3, 7.7.4, 7.5.4*

The data to consider in the analysis for beams and columns are defined in *clause 7.7.2* of EN 1998-1, with reference to *clause 7.4* therein. The background to the rules has been discussed above, in Section 7.6.

Under a combination of gravity loading and seismic action effects, the values of the maximum positive and negative bending moments in beams may be very different. The acceptable sections are related to the absolute maximum of these two moments. However, following the general statement in *clause 7.1.1(1)* of EN 1998-1, redistribution of bending moments according to EN 1994-1[82] for composite steel–concrete structures is permitted, which allows a reduction of the design moment, as has been explained in Section 6.9 of this guide.

The rules for beams and columns are defined in *clause 7.7.3* of EN 1998-1, with reference to *clause 7.6.2* for composite T beams and to *clause 7.6.5* for partially encased beams. Several aspects of *Section 6* of EN 1998-1 on steel structures continue to apply, e.g. *clauses 6.6.2(2)* and *6.6.3(1)*.

The following inequality should be checked for composite columns of moment-resisting frames:

$$N_{Ed}/N_{pl,Rd} < 0.3 \qquad \text{(D7.3)}$$

It corresponds to the fact that, in moment-resisting frames, the column response is primarily flexural and may have to be dissipative. To ensure a satisfactory cyclic response, it is necessary to limit column axial force to below a certain value. However, this rule is not overly restrictive as the moment capacity of composite columns decreases significantly above this level of axial load.

The specific rules for composite connection design are given in *clause 7.5.4* of EN 1998-1. Their background has been discussed above in Section 7.7. The specific rules for composite connections supplement the general rules for steel connections in *clause 6.5.5* of EN 1998-1 and the paragraphs specific to moment-resisting frames in *clause 6.6.4* therein.

### 7.11.3. Disregarding the composite character of beams with a slab

*Clause 7.7.5*

A designer can design and detail the structure in such a way that the composite character is used in the central part of beams in order to provide stiffness and strength, while at beam ends only the steel profile is activated, so that the plastic zones at the ULS under the design seismic action are limited to the steel profile. Several reasons may justify this design choice:

- Beams are made composite in the central part of the beam span, to profit from the composite action of the slab with the steel profile for stiffness and resistance to gravity loads.
- The discontinuity of the slab connection at beam ends allows bypassing of the seismic design of the slab, the provision of seismic rebars and other detailing.
- Beams are not considered composite at the beam-to-column connection, so that the design check at the beam ends is simply $M_{Ed} \leq M_{pl,Rd}$, in which $M_{pl,Rd}$ is the readily available design resistance of the steel profile only.
- Column sections designed to meet the 'strong column–weak beams' concept are capacity designed to the beam plastic resistance by the application of equation (D4.23). If the beam design resistance is that of a composite section, this may result in serious oversizing of column sections, in particular in the upper storeys of the building. Managing the design in such a way that only the steel section resistance is involved may help to reduce the excess in column sections.

This 'slab disconnection' option raises some practical problems and implications. The disconnection must be sufficiently effective to ensure that the plastic moment at the beam ends is actually the plastic moment of the steel section alone. The beam-to-column

connection zone is three-dimensional; experiments have shown that preventing contact between concrete and steel only at the column face does not always achieve an effective disconnection. The disconnection from the steel frame has to be more complete, following *clauses 7.7.5(1)*, *7.7.5(2)* and *7.7.5(3)*: there must be total disconnection within a circular zone of diameter $2b_{eff}$ around the column, $b_{eff}$ being the greater of the effective widths of the beams connected to that column. 'Total' disconnection means that there is no contact between the slab and any vertical side of any steel element (e.g. columns, shear connectors, connecting plates, corrugated flange, or steel deck nailed to the flange of the steel section).

Of course, the analysis of the frame has to consider the two different stiffnesses encountered in a beam span: at beam ends, the section stiffness $EI$ is that of the steel profile along the length of the disconnection, while in the central part of the span the stiffness is that of a composite section.

### 7.11.4. Limitation of overstrength

As explained in Section 6.9 of this guide and for the same reasons, the seismic design process of composite moment-resisting frames may produce structures with excess material. The countermeasures are similar to those proposed in Section 6.9. Figure 6.7 presents an example of the application of the reduced beam section concept to composite structures. Another option is to use the slab disconnection technique described in the previous section, which is in fact another form of the reduced beam section.

## 7.12. Composite concentrically braced frames

*Clause 7.8*

The non-dissipative structural elements of composite frames with concentric bracings, columns and beams can be either structural steel or composite; however, the dissipative elements (the bracings) have to be structural steel. The reasons for this choice have been explained in Section 7.5:

- composite bracings increase the possible overloading of beams and columns in the first buckling stage, which results in an increased overstrength of the structure in comparison with a frame with steel bracings
- composite bracings have not been sufficiently studied; there are uncertainties concerning their cyclic behaviour in tension as well as in compression.

No restriction like that of equation (D7.3) for moment-resisting frames is imposed on the ratio $N_{Ed}/N_{pl,Rd}$ in columns of braced frames, as bending moments are present to a much lesser extent than in moment-resisting frames. Moreover, in braced frames the concrete encasement increases the axial resistance of the members and helps to prevent buckling.

Except for the possibility of using composite sections for the non-dissipative elements, *clause 7.8* in EN 1998-1 for composite concentrically braced frames is identical to its counterpart for steel concentrically braced frames, which has been analysed in Section 6.10.

## 7.13. Composite eccentrically braced frames

*Clauses 7.9.1, 7.9.2, 7.9.3, 7.9.4*

The non-dissipative structural elements, columns and beams, can be either structural steel or composite. The dissipative elements, called seismic links, can be:

- Pure steel sections; in this case, there is no restriction to design, and *clause 6.8* in EN 1998-1 applies to the links.
- If the links are composite, they must be short or intermediate, and work essentially in shear. Links consisting of steel beams composite with slab are allowed, because the contribution of the slab to the shear resistance of a beam is minimal and thus under

control. Links should not include encased steel sections, due to the uncertainties in the contribution of concrete to shear resistance in this case.

These limitations have been explained in detail in Section 7.5.

As in moment-resisting frames, the analysis of the structure has to consider two different stiffnesses for the zones which are under sagging and hogging moments, following the guidance in *clauses 7.4.2* and *7.9.2*.

Specific construction details are given in *clauses 7.9.3(3)* and *7.9.4(2)*:

- face bearing plates as prescribed in *clause 7.5.4(9)*, for links framing into reinforced-concrete columns
- transverse reinforcements as prescribed in *clause 7.6.4*, for fully encased composite columns adjacent to links.

Besides these aspects, the background for the design of composite eccentrically braced frames is similar to that for steel eccentrically braced frames, for which detailed explanations have been given in Section 6.11.

## 7.14. Reinforced-concrete shear walls composite with structural steel elements

### 7.14.1. General

*Clause 7.10.1*

Concrete shear walls composite with structural steel elements can be perceived as a composite equivalent of steel or composite frames with bracings: they comprise two vertical steel profiles acting as 'flanges' in a vertical beam, in which the bracings are replaced by a 'web' of concrete. In walls of Type 1, this web also includes horizontal steel profiles.

Concrete shear walls composite with structural steel elements can also be perceived as concrete shear walls in which the vertical reinforcement has been replaced by two vertical steel profiles. This latter description is favoured for the design process.

As in reinforced-concrete walls, energy dissipation is pursued through flexural behaviour of the wall and achieved by the yielding of the vertical 'rebars'.

The advantage of composite shear walls is their higher flexural resistance and stiffness compared with a reinforced-concrete wall of the same cross-sectional dimensions.

### 7.14.2. Analysis and design rules for beams and columns

*Clauses 7.10.2, 7.10.3*

In accordance with the reference concept for reinforced-concrete walls, composite walls are given the stiffness properties of such walls, including the contribution of steel beams and columns. For the steel components, the analysis refers to an equivalent concrete section which is computed considering the modular ratio $n = E_a/E_{cm} = 7$.

The verification checks of sections of the wall subjected to combined compression and bending consider the concrete stresses in the vertical direction and the vertical steel components of the wall in the same way as for a reinforced-concrete column or wall:

- Concrete is assumed to not resist tension, and only the vertical steel profiles and the adjacent rebars are considered effective.
- On the compression side, the concrete works together with the steel profile and the rebars. The section of steel profile should be selected for its resistance to local buckling, in relation to the intended ductility class of the structure, following *Table 7.3* in EN 1998-1.

The design of the wall in shear, including dimensioning of its web reinforcement, is similar to that of ductile walls in *Section 5* of EN 1998-1 (Fig. 7.2). The shear resistance of composite walls involves one specific design aspect which is related to the shear transfer in the web of a reinforced-concrete beam: the latter is based on the consideration of a strut-and-tie mechanism, in which compression struts are made of concrete, and the transverse rebars are the tension ties. For this mechanism to be effective, the struts and the

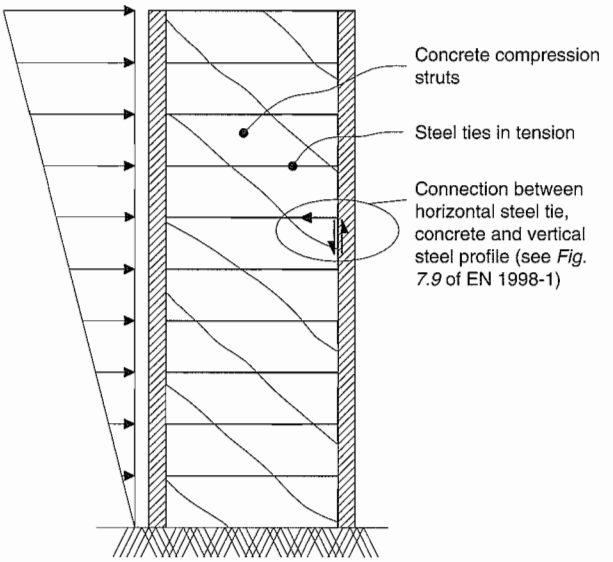



**Fig. 7.2.** A composite wall considered as a concrete wall

ties should be 'connected', which is achieved by the stirrups placed around the longitudinal rebars. The detailing in composite walls must achieve a similar connection, as required by clause *7.10.1(2)*: the reinforced-concrete web has to be tied to the steel boundary elements, to prevent separation. The ties should be able to sustain tension forces perpendicular to the vertical steel boundary elements equal to the forces in the stirrups (which are horizontal in this case). Different details for the ties are possible, and two types of detail are suggested in *Fig. 7.9* of EN 1998-1. The first makes use of bars welded to the steel column. The other employs anchorages within a confined concrete volume, including a fully encased H section.

Following the vertical reinforced-concrete beam concept explained above, the horizontal steel profiles present in walls of Type 1 are ties that differ from classic stirrups but play the same role.

Headed shear studs or welded reinforcement anchors are needed to transfer the shear forces between the structural steel of the boundary elements and the reinforced concrete. These shear connectors are the equivalent of the ribs of rebars which provide their bond resistance and are necessary for longitudinal rebars to act as longitudinal reinforcements in reinforced-concrete members. Shear connectors on horizontal steel profiles in walls of Type 1 create the bond resistance for this specific type of horizontal stirrup. The shear connection requirement provides a more uniform transfer of forces between the web and the boundary members. The detailing of *Fig. 7.9* of EN 1998-1 shows two examples of shear connections to vertical steel profiles, one by means of welded reinforcement anchors (*Fig. 7.9a*) and the other one by headed studs (*Fig. 7.9b*).

Tests on walls of Type 1 have shown that if there are no shear connectors, the storey shear forces are carried primarily through diagonal compression struts in the web of the wall, which involve high forces in localized areas of the wall and of connections.

## 7.15. Composite or concrete shear walls coupled by steel or composite beams

*Clauses 7.10.4, 7.10.5*

Besides composite structures in which structural elements are composite, other structural schemes bear the characteristics of composite systems: in them, steel or composite structural elements frame into reinforced-concrete structural elements. One possibility is a system in

which coupling steel or composite beams frame into and couple reinforced-concrete walls. The $q$ factor of such systems is higher than that of systems of concrete-coupled walls, due to the possibility of high energy dissipation in the walls and in the coupling beams.

The only specific aspect of the design of such systems concerns the connection zone between beams and walls. This aspect is mostly covered in *clause 7.5.4* on connection design; further design guidance is provided in *clauses 7.10.4* and *7.10.5* about

- the embedment length of the steel profile, defined in *clause 7.10.4(1)*
- the vertical reinforcements in the wall, defined in *clause 7.10.4(3)*
- the transverse reinforcements needed for better confinement in the embedment length for DCH structures, defined in *clause 7.10.5(1)*.

Tests have shown that properly detailed coupling beams yield at the face of the concrete wall and provide stable hysteretic behaviour under reversed cyclic loads.

## 7.16. Composite steel plate shear walls

*Clause 7.11*

Composite shear walls reinforced by a steel plate can be used most effectively where storey shear forces are large and the required thickness of conventionally reinforced shear walls is excessive. The provisions in EN 1998-1 limit the shear strength of the wall to the yield strength of the plate, because there is insufficient basis for the development of design rules for combining the yield strength of the steel plate and that of the reinforced-concrete panel. Moreover, since the steel plate will normally be designed so that its shear strength is much greater than that of the reinforced-concrete encasement, neglecting the contribution of the concrete does not have a significant impact.

The connection between the plate and the boundary members (columns and beams), as well as the connection between the plate and the concrete encasement, should be designed so that the full yield strength of the plate can be developed. This means that locally, as well as overall, plate buckling has to be avoided. Regarding the avoidance of local buckling, this can for instance be achieved by limiting the stud spacing in such a way that the length-to-thickness ratio of unstiffened parts of the steel plate remain under a certain limit. It is recommended that overall buckling of the composite panel be checked using elastic buckling theory, considering the section stiffness of the composite wall.

CHAPTER 8

# Design and detailing rules for timber buildings

## 8.1. Scope

*Clause 8.1*

This chapter covers the rules for the seismic design of timber buildings, following in a loose way *Section 8* of EN 1998-1. However, it does not elaborate on all clauses in that section; and neither does it strictly follow the sequence of clauses.

It is important to stress that for the overall design of a timber building, the rules of EN 1998-1 are additional to those presented in EN 1995-1-1.

## 8.2. General concepts in earthquake resistant timber buildings

*Clauses 8.1.2, 8.1.3*

Timber is generally considered to be a suitable structural material for construction in seismic areas, due to its relatively light weight and reasonable strength, both in tension and in compression.

This is apparent from Table 8.1, where typical values of unit mass and strength of different structural materials are presented, together with the ratio of these two variables, which is a particularly relevant characteristic for the earthquake resistance of structures. The good performance of wood is apparent from the table, with the range of $f/\rho$ values for wood being similar to that of structural steel. However, the behaviour of timber elements does not present large deformational ductility, which is also an important property for the seismic resistance of structures (as discussed in Chapter 2). In fact, the response of timber elements

**Table 8.1.** Typical values of unit mass and strength for various structural materials and corresponding ratios

| Structural material | | Unit mass $\rho$ (kg/m$^3$) | Strength $f$ (range) (MPa) | Ratio $f/\rho$ ($10^{-3}$ MPa/kg/m$^3$) |
|---|---|---|---|---|
| Wood | Compression and tension | 550 | 20–30 | 35–55 |
| Structural steel | Compression and tension | 7800 | 275–355 | 35–45 |
| Concrete | Compression | 2400 | 25–80 | 10–30 |
| | Tension | 2400 | 2–3.5 | 0.8–1.5 |
| Reinforced concrete | Bending | 2500 | 10–25 | 4–10 |
| Masonry | Compression | 2100 | 4–8 | 1.9 – 3.8 |
| | Tension | 2100 | 0.3 – 0.5 | 0.1–02 |

up to failure is approximately linear elastic with sudden collapse, mostly associated with defects inherent to its natural origin.

As for other structural materials, EN 1998-1 distinguishes between **dissipative structural behaviour** and **low-dissipative structural behaviour** for timber structures.

Dissipative structural behaviour is characterized by the existence in the structure of parts (dissipative zones) which are able to resist the earthquake action in its inelastic range, and thus are able to dissipate hysteretically the energy that the earthquake inputs into the structure. In contrast, low-dissipative structural behaviour is characterized by a response of the structure to the seismic action in its elastic range, i.e. without significant yielding of its members or hysteretic dissipation of energy.

Taking into account the limited ability of wood to behave non-linearly as discussed before, in timber structures the dissipation of energy should take place mostly in the connections, with the timber elements themselves behaving in the elastic range, in which the response is very good.

The brittle failure of timber elements results from natural defects in the material, for instance knots. Only for compression perpendicular to the grain may some non-linear response be expected, with some capability for energy dissipation. In total contrast, tension perpendicular to grain exhibits a markedly brittle failure associated with splitting of the material.

Accordingly, what establishes the main difference between dissipative and low-dissipative timber structures is the nature of their connections. Hence, EN 1998-1 presents a basic distinction between **semi-rigid joints** and **rigid joints,** the former being associated with the possibility of being provided with the capability of dissipating energy under reversed cycling, whereas the latter (essentially glued solid timber joints) are devoid of such capability.

Dissipation of energy in semi-rigid joints results normally from two main sources:

- cyclic yielding of the metallic (normally steel) dowel-type fasteners of the connection (as nails, staples, screws, dowels or bolts)
- crushing of the wood fibres bearing against the dowel.

The first mechanism tends to be stable, and provides large hysteretic loops (typical of reversed yielding of steel in bending), whereas the second mechanism is characterized by thin hysteretic loops, which are markedly pinched and present significant strength degradation for cycling at constant amplitude. This is caused by the progressive increase in size of the cavity that is being formed in front of the dowel by the wood crushing, as the succession of cycles progresses. Naturally, the response of the connection as a whole results from the interaction between the two mechanisms, and so to achieve good dissipative behaviour it is crucial to achieve a correct balance between the wood crushing and the dowel yielding, for which the more significant parameter is the slenderness of the dowel-type element (i.e. the ratio between the thickness of the connected member and the fastener diameter).

Besides the main distinction between dissipative and low-dissipative structures, EN 1998-1 subdivides dissipative structures into two **ductility classes**, namely Ductility Class Medium (DCM) and Ductility Class High (DCH), with Ductility Class Low (DCL) being assigned to low-dissipative structures. The use of the concept of ductility classes is established in general terms in Section 2 of EN 1998-1 (see *clause 2.2.2(2)*); the subdivision of classes indicated for timber structures is in line with those established for other structural materials (reinforced concrete, steel and composite).

The choice between ductility classes in timber structures is left to the designer, as is the case for other materials. However, in this case it is foreseen that national authorities in the relevant National Annex may establish some limitations in the use of the various ductility classes (i.e. the applicability of the different ductility classes to timber structures is a Nationally Determined Parameter). In any case, as noted in Section 2..2.2, the choice between different ductility classes corresponds roughly to the trading between higher lateral

resistance on the one hand and higher deformation capability in the non-linear range (ductility) and energy dissipation capacity on the other hand.

For timber structures the parameters that influence its ductility classification are the structural type (essentially reflecting the greater or lesser redundancy of the structure as a whole) and the nature of the structural connections (essentially reflecting its ductility and energy dissipation capacity). For the latter, EN 1998-1 states in general that the properties of the dissipative zones (i.e. the connections) should be determined by tests, in accordance with prEN 12512.[85] However, for the most common connection types, some deemed to satisfy rules are presented in the code, in order to reduce the burden of testing connections in ordinary design situations.

As a final remark and in line with other structural materials, it should be noted that for timber structures of DCL, the behaviour factor may be taken up to $q = 1.5$. Even though, as indicated above, for this ductility class no significant decrease of the earthquake forces is expected on account of a non-linear response; the use of a behaviour factor slightly greater than $q = 1$ is justified by the overstrength that structures normally present under earthquake action.

## 8.3. Materials and properties of dissipative zones

*Clause 8.2*

In general, the requirements for wood materials set out in EN 1995-1-1 (Eurocode 5 for timber structures) also apply in the design of earthquake-resistant timber structures covered by EN 1998-1. The same is true for the steel elements included in joints and connections, concerning the requirements set forth in EN 1993-1-1[83] (Eurocode 3 for steel structures). However, in order to ensure the required dissipative behaviour in timber structures of DCM and DCH, some additional requirements have to be fulfilled with regard to the mechanical characteristics of the materials and to the characteristics of joints. In all cases, the objective of the additional requirements is to avoid brittle failures and to obtain connections with stable behaviour under large deformation reversals.

For connections in framed systems, no specific conditions are set, besides the general reference to the need to demonstrate by testing that the connections have a stable low-cycle fatigue response and the explicit indication that glued joints may not be considered as dissipative zones (since they respond elastically up to failure which is essentially brittle due to debonding). In contrast, for sheathing materials some minimum mechanical characteristics are required, namely:

- for particle-board panels the density should be at least 650 kg/m$^3$
- for particle-board and fibre-board sheathing the thickness should be at least 13 mm
- for plywood sheathing the thickness should be at least 9 mm.

This is intended to ensure the excellent ductile behaviour shown by nailed shear panel systems, which is usually superior to conventional diagonal bracing but relies very much on the properties of the sheathing boards.[86] Also important for the appropriate (i.e. stable) response of this sort of system is the avoidance of the pull-out of the nails under transverse cycling. To this end, a point side penetration of 6–8 times the sheathing thickness is appropriate, and smooth nails should be avoided or be provided with additional provision against withdrawal (e.g. by coating or clenching).

## 8.4. Ductility classes and behaviour factors

*Clause 8.3*

The choice of ductility class has a direct influence on the value of the behaviour factor $q$ that, in turn, controls the lateral forces for which the structure has to be designed (as discussed in general terms in Chapters 2 and 4, independently of the structural material). For timber structures, EN 1998-1 presents upper limit values of the behaviour factor depending on the ductility class and the type of structure and connections used.

**Table 8.2.** Maximum values of the behaviour factor $q$ for timber structures of DCM and DCH

| Structural type | DCM | DCH |
|---|---|---|
| Wall panels with **glued diaphragms** connected with nails and bolts | **Glued** panels<br>$q = 2.0$ | **Nailed** panels<br>$q = 3.0$ |
| Wall panels with **nailed diaphragms** connected with nails and bolts | — | **Nailed** panels<br>$q = 5.0$<br>$(q = 4.0)$ |
| Trusses | **Doweled** and **bolted** joints<br>$q = 2.0$ | **Nailed** joints<br>$q = 3.0$ |
| Mixed structures with timber framing and non-load-bearing infills | $q = 2.0$ | — |
| Hyperstatic portal frame with doweled and bolted joints | $\mu \geq 4$<br>$q = 2.5$ | $\mu \geq 6$<br>$q = 4.0$<br>$(q = 2.5)$ |

Besides the general upper limit of $q = 1.5$ for DCL accounting for overstrength, for DCM and DCH the values indicated for $q$ in *Table 8.1* of EN 1998-1 are reproduced in Table 8.2 with a different arrangement that highlights the influence of the various parameters on the ductility of timber structures (namely the superior behaviour of correctly designed and executed nailed connections).

The values presented in Table 8.2 are appropriate for buildings, which are **regular** in elevation (see Chapter 4). For buildings with **non-regular** structure in elevation, the behaviour factor should be reduced by 20%, as is also required for buildings of other structural materials, in order to account for the expected higher local ductility demands in those cases.

The values presented in Table 8.2 are applicable if the dissipative zones in the structure are able to withstand, without a decrease of strength of more than 20%, three fully reversed cycles at a ductility demand of $\mu = 4$ for DCM and $\mu = 6$ for DCH. For portal frames the ductility should be evaluated in terms of the rotational capacity of the joints, whereas in wall panels the ductility should be evaluated in terms of shear displacements of the panels.

In principle, the available ductility should be measured by testing. For non-bilinear responses (i.e. a response without a clear identification of the yielding point), as is normally the case for timber joints with doweled metal connectors, it is sometimes difficult to evaluate the available ductility, due to the absence of a precise value for the yielding displacement (or rotation). To overcome this difficulty, a bi-linear diagram enveloping the real constitutive diagram may be used as an equivalent response, in which case the yielding is defined precisely. It is suggested[87] that for timber structures the second branch of such an equivalent bi-linear envelope has a stiffness which is one-sixth of the stiffness of the initial (linear) branch.

This requirement of testing would be very cumbersome in most ordinary design cases, and so the following **deemed to satisfy rules** are given in EN 1998-1:

(1) The slenderness of the fasteners in doweled, bolted and nailed connections should be greater than 10 ($t/d \geq 10$, with $t$ being the thickness of the connected member and $d$ the fastener diameter) and the fastener diameter should not be larger than 12 mm.
(2) The sheathing material (wood based) in shear walls and floor diaphragms should have a thickness larger than four times the fastener diameter ($t \geq 4d$), and the nail diameter should not exceed 3.1 mm.

These requirements reflect that, for good performance of the connections under cyclic load, thick timber and slender dowels are preferable because they allow for yielding in bending of the fasteners (whereas with stocky dowels the failure mode will mostly be

associated with the crushing and splitting of the timber fibres, which does not allow for the dissipation of energy).

In any case, the numerical values are relatively severe, and it has been suggested[86] that an even less demanding value of $t/d \geq 8$ is still very much on the safe side in terms of the ductility of connections. Also it is worth mentioning in this context that EN 1995-1-1 (Eurocode 5 for timber structures in general) allows much larger bolts and dowels with diameters up to 30 mm (clauses 8.5.1.1 and 8.6 of EN 1995-1-1).

Hence, when the above requirements are not met strictly (i.e. if $t/d \geq 8$ and $t \geq 3d$, respectively, for cases 1 and 2 above) it is still permitted to avoid testing of the connections of dissipative structures, but the maximum values of the behaviour factor should be decreased as shown in parentheses in Table 8.2.

## 8.5. Detailing

*Clauses 8.5.2, 8.5.3*

For buildings of DCM and DCH, additional detailing rules are required in comparison with the general provisions of EN 1995-1-1. These additional rules are intended to enhance the behaviour of connections and horizontal diaphragms.

For bolts, an absolute limit of 16 mm is established for their diameter, unless toothed ring connectors are also used These provide some confinement of the wood in front of the bolt and allow for the larger bearing forces associated with larger bolts. Furthermore, it is required that in pre-drilled connections they are tightly fitted. This is because oversized holes may cause a non-uniform distribution of loads in different bolts of the same connection. In such cases there may be a tendency to overload some bolts, which triggers premature splitting and crushing of the wood bearing against these bolts, initiating a chain collapse in the other bolts.

For floor diaphragms, the additional detailing provisions are intended to increase the effectiveness of the sheathing material and the stability of its connection (particularly at the edges of the panels) to the framing timber elements. This is reflected by forbidding consideration of the increased resistance of edge fasteners (allowed for in general terms for 'non-seismic/non-ductile' cases by clause 9.2.3.1 of EN 1995-1-1) and by more strictly controlling (i.e. limiting) the nail spacing at the panel edges allowed by clause 9.2.3.2 of EN 1995-1-1.

Also closer spacing has to be adopted for fasteners in areas of discontinuity in case of relatively high seismicity ($a_g S \geq 0.2g$) to avoid the premature initiation of rupture in those areas and to somehow compensate for its decreased stiffness. In any case, the minimum spacing established in EN 1995-1-1 (clause 10.8.1) should always be respected to ensure that splitting of the wood is prevented. Accordingly, in these areas of discontinuity, the dimensions of the timber elements should be generous to allow effective nailing that is not too closely spaced.

## 8.6. Safety verifications

*Clause 8.6*

Safety verifications are to be made using the general resistance models provided in Sections 5 and 6 of EN 1995-1-1. Naturally, in the seismic design situation, the strength modification factor $k_{mod}$, which accounts for the influence of the duration of the load (and also of the moisture content) on the resistance of timber or wood-based materials should be taken with the value appropriate for instantaneous actions (see Table 3.1 of EN 1995-1-1).

Regarding the partial factor for material properties $\gamma_M$ to be used in the ultimate limit state verifications, an important distinction is made between:

- structures of DCL, for which the $\gamma_M$ values for the fundamental load combinations are recommended, and
- structures of DCM or DCH, for which it is permitted to use the smaller (equal to 1.0) values recommended for the accidental load combinations (see Table 2.3 of EN 1995-1-1).

This is an important departure from analogous recommendations in other sections of EN 1998-1 for other structural materials (namely *Section 5* for reinforced concrete, *Section 6* for steel and *Section 7* for composites), in which it is recommended that $\gamma_M$ values for the fundamental load combinations are used in the seismic design situation. This rule has an important influence on the outcome of the design for the two types of structure (low dissipative and dissipative), and reflects the more reliable response of timber connections and timber structures satisfying the additional requirements for dissipative structures which are set forth in this section of EN 1998-1.

CHAPTER 9

# Seismic design with base isolation

## 9.1. Introduction

*Clauses 10.1(1), 10.1(2), 10.1(4), 10.2(1), 10.10(4), 10.10(5)*

The seismic protection of conventional structures is based on the favourable changes of their dynamic characteristics, induced by yielding and damage occurring in structural and non-structural elements under intense seismic action. Such changes can be essentially described as an increase of flexibility and of damping. Due to the usual spectral characteristics of earthquakes and/or to the energy dissipation occurring in the structure, these changes give rise to a considerable reduction in the structural mass accelerations and, then, of the inertia forces. This makes it possible for a ductile structure to survive a 'destructive' earthquake without collapsing. To account for this behaviour, current codes, including EN 1998-1, introduce a factor (the behaviour factor in *clause 3.2.2.5(2)* of EN 1998-1), depending on the ductility capacity of the different types of structures, which reduces the elastic forces.

In the last two to three decades, new strategies have been developed which still rely upon deformation and energy dissipation capabilities. These properties, however, are concentrated in special devices, in the form of rubber or sliding bearings, of energy dissipating and/or re-centring viscous or hysteretic devices, etc. Such devices are incorporated in the structure so as to store and dissipate most of the input energy. The inertia forces acting on the structure during a strong earthquake are considerably reduced, so that no damage to structural and non-structural elements is in principle required to further reduce them, and hence higher levels of seismic protection are obtained.

Several implementations of these strategies, which are usually classified within the broad category of 'passive control' of seismic vibrations, are presently available, differing in the type of construction they are intended for, the desired modification of the dynamic characteristics, and the utilized technology.

The two most frequently used 'passive control' strategies for buildings are:

- energy dissipation
- seismic isolation.

The energy dissipation strategy[88–90] consists of the introduction within the structural system of elements specifically designed to dissipate energy in the dynamic deformation of the structure. These elements may take the form of dissipative steel bracings separate from the structure and working in parallel with it, or they can be diagonal elements inserted in the structure between consecutive floors. The dissipation can be obtained by the use of friction devices, viscous dampers or elasto-plastic steel components. The type of analysis appropriate for this category of passive control, which is not explicitly dealt with in EN 1998-1 (*clause*

*10.1(4)*), depends on the specific choice of the dissipative elements and on their coupled behaviour with that of the structure.

Seismic isolation[91–93] essentially uncouples the structural movement from the ground motion by introducing a strong discontinuity in the lateral stiffness distribution along the height of the structure (usually at their base in buildings and between piers and deck in bridges, as shown in Fig. 9.1). The structure is thus subdivided into two parts (*clause 10.2(1)*): the substructure, rigidly connected to the ground, and the superstructure. They are separated by the isolation interface, which includes the isolation system.

The vertical structural continuity is provided by suitable bearing devices placed between the substructure and the superstructure, called isolators. They are characterized by a relatively low resistance to horizontal movements and, normally, high vertical stiffness. They are usually meant to be effective only for horizontal isolation, since the vertical ground motion component is not normally as dangerous as the horizontal one.

The behaviour of the isolation system can vary from quasi-linear (e.g. almost visco-elastic) up to strongly non-linear (e.g. elasto-plastic), allowing for different isolation strategies to be realized (*clause 10.1(2)*). Essentially, two strategies can be identified, i.e. (Fig. 9.2):

(1) period elongation, accompanied by different amounts of energy dissipation
(2) force limitation, accompanied by different amounts of energy dissipation.

In the first strategy, quasi-elastic devices are used to increase significantly the period of the structural system so as to obtain a strong reduction in the (spectral) acceleration acting on the structural masses. Acceleration response spectra for soil types ranging from firm to medium stiff exhibit strong amplification in the 0.2–0.8 s range, where the fundamental period of most buildings up to 10 storeys falls, while decreasing rapidly for higher periods. For periods of the order of 2–4 s, spectral accelerations can be at least 5–10 times less than in the previously mentioned interval. On the other hand, the displacement response increases

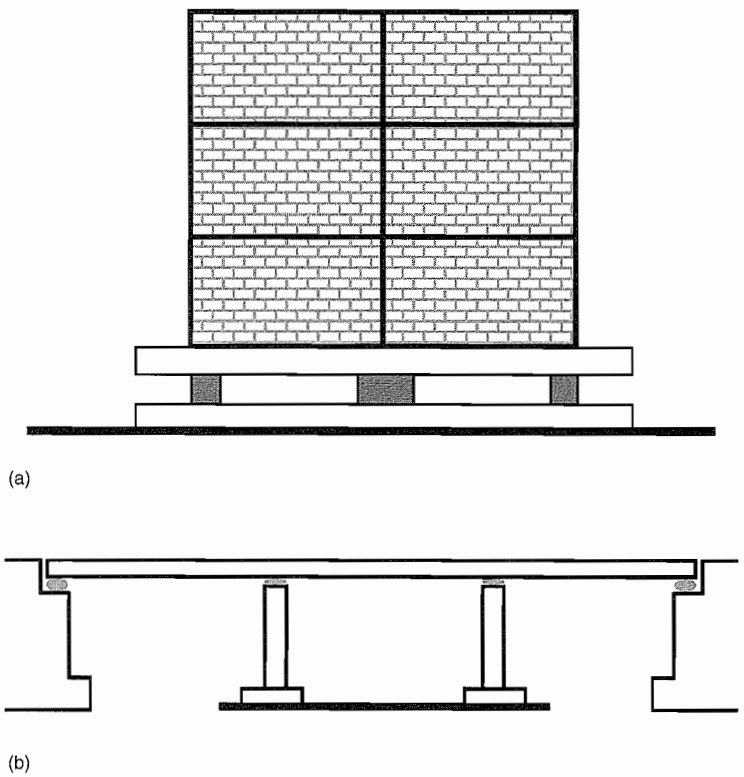



**Fig. 9.1.** Seismic isolation of (a) buildings and (b) bridges

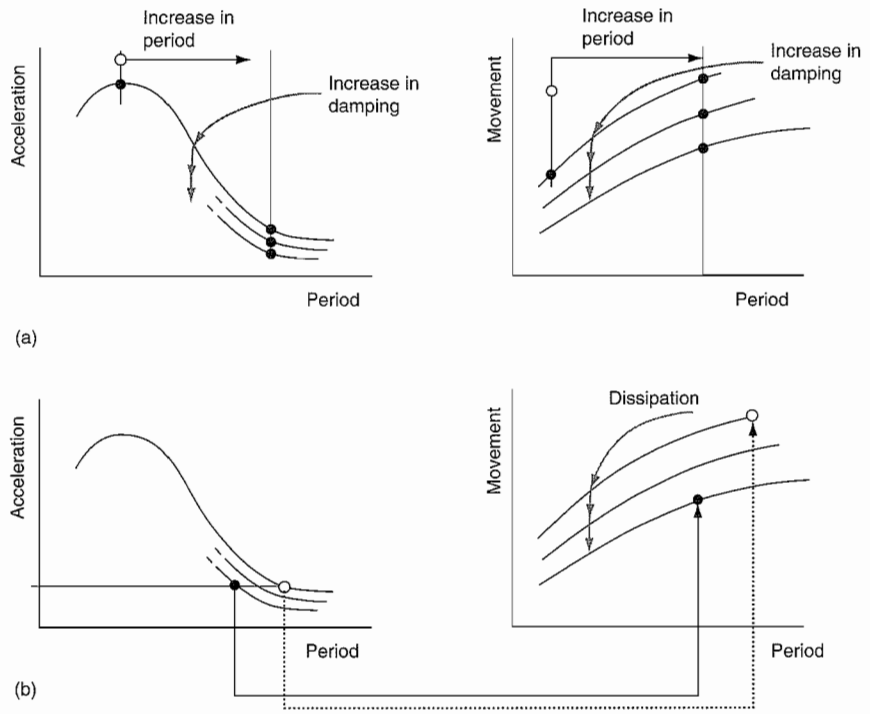



**Fig. 9.2.** Strategies of demand reduction using seismic isolation: (a) period elongation (and damping); (b) force limitation (and damping)

with increasing period, reaching quite high values for the periods of an isolated structure (see Fig. 9.2). Most of the displacement, however, occurs at the isolation system level, while structural displacements remain limited, the superstructure behaving almost as a rigid body. In terms of energy, this corresponds to the fact that most of the energy input into the structural system is accumulated and dissipated in the isolation system, thus protecting the structural elements from possibly damaging deformations. The energy dissipation capability of the isolation system is essentially exploited to reduce the base displacement, which sometimes can be so large (some hundreds of millimetres) as to make structural, architectural and equipment requirements difficult to accommodate. Energy dissipation also reduces the base shear, though excessive damping might locally increase floor accelerations. This can be especially important when protection of the building contents is a priority, as high damping can excite higher modes whose periods are nearer to the periods of appendages and objects in the building.

In the force limitation strategy, the devices provide almost constant force with increasing displacement, as for example in the case of elasto-plastic devices. The reduction of the effects in the superstructure is due to the limitation of the force that the device can transmit to the structure. Imposing a limit to the transmitted forces can be seen as a special application of the capacity design criterion, where a strength hierarchy is established between the global resistance of the structure (both the substructure and superstructure) and the strength of the isolation system. The energy dissipation is essentially exploited to limit the base displacement. Abrupt variation in stiffness in their force–displacement cycles, typical of the behaviour of some strongly non-linear devices, may imply greater values of the floor acceleration for the superstructure at the high frequencies, due to the excitation of higher modes.

The period elongation strategy is, by far, the most frequently used in buildings, for several reasons, often related to technological and constructive aspects. The force limitation strategy is employed notably in those cases where the control of the seismic forces represents the

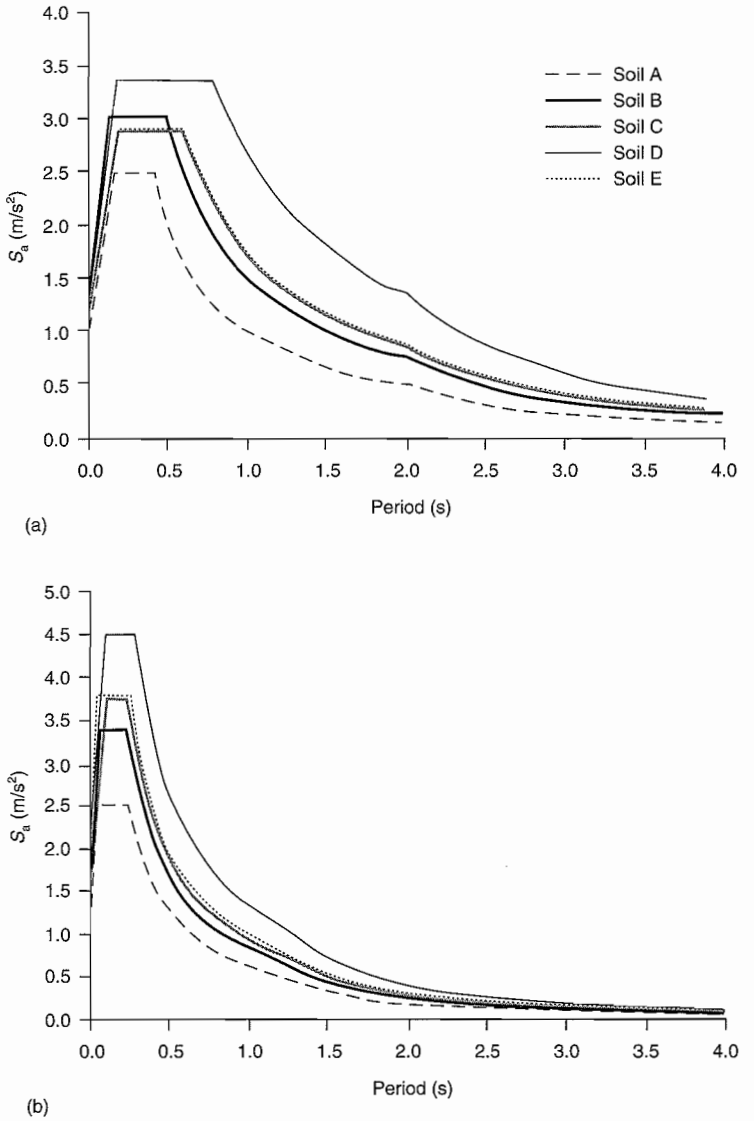



**Fig. 9.3.** Eurocode 8 elastic response spectra for 5% damping: (a, b) pseudoacceleration spectra; (c, d) displacement spectra; (a, c) recommended spectra of type 1; (b, d) recommended spectra of type 2

most crucial design aspect (e.g. in the retrofit of existing structures) and also when the period elongation strategy attains its technological and economical limits (e.g. for long-period structures or for earthquakes with high energy content in the low-frequency range).[92] Indeed, a positive aspect of the force limitation strategy is the substantial independence of its effectiveness from the characteristics of the earthquake (intensity and frequency content), conditional on the acceptance of large displacements.[94]

The reduction of the response acceleration attainable with seismic isolation just by increasing the period and damping is numerically comparable and even much larger than the behaviour factors by which the elastic spectrum is reduced based on ductility considerations. With reference to concrete buildings (*Section 5* of EN 1998-1), the behaviour factor can assume values between 1.5 (inverted-pendulum system designed for Ductility Class Medium (DCM)) and 6.75 (frame system, dual system, coupled-wall system designed for Ductility Class High (DCH), with the maximum allowed value of the system overstrength ratio $\alpha_u/\alpha_1$; 5.85 if the common default value $\alpha_u/\alpha_1 = 1.3$ is used). Considering a structure with a period

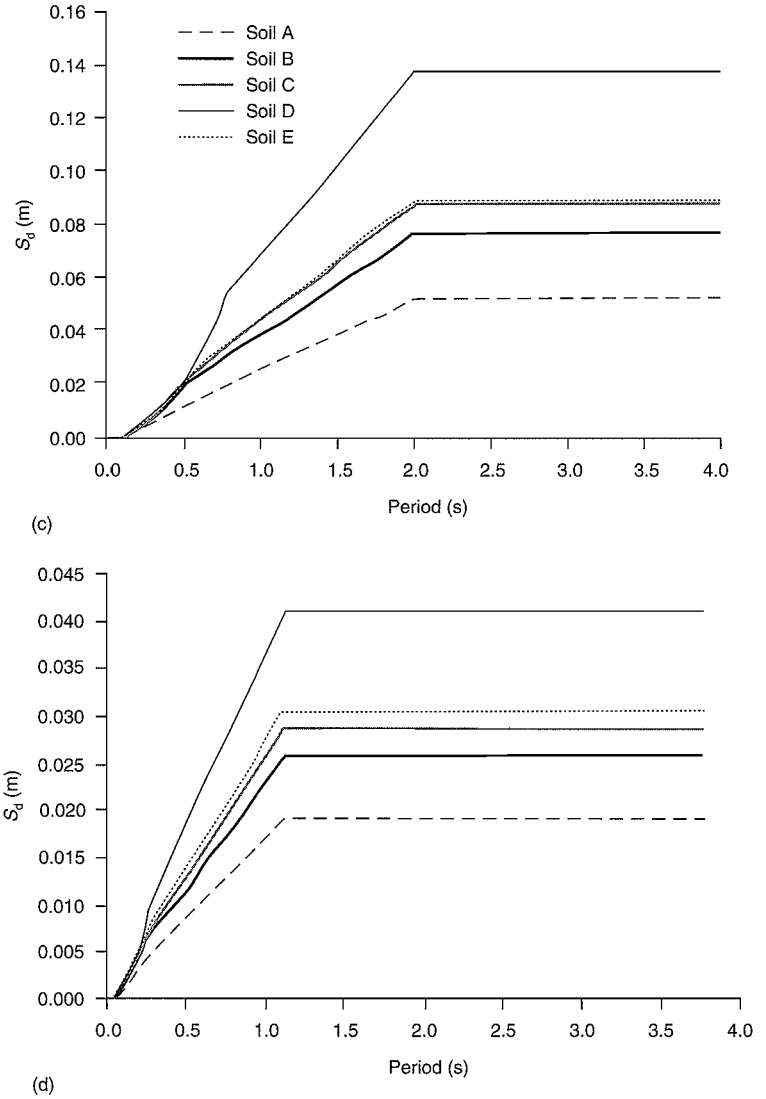



**Fig. 9.3.** (Contd)

$T = 0.4$ s, isolated to $T = 2$–4 s and 10–20% damping, and considering different soil profiles (A, B, C, D, E) of type 1 earthquakes (see Chapter 3 and Fig. 9.3), the design acceleration can be reduced with respect to the elastic acceleration by a factor of between 3.1 (soil D, $T = 2$ s, 10% damping) and 31.6 (soil A, $T = 2$ s, 20% damping). The effectiveness of seismic isolation is even greater for type 2 spectra, while it decreases when dealing with structures having a longer fixed-base period. It is clear, then, that seismically isolated structures do not need to rely upon their inelastic deformation capacity to survive strong earthquakes, and that the full isolation concept (i.e. no or very limited inelastic deformation all over the structure) can be economically applied in most design cases. On the other hand, partial isolation, i.e. the allowance for significant yielding in the superstructure, is known to be difficult to control with the use of a simple linear analysis coupled with a reduced elastic response spectrum. It is well known[95–97] that the low-frequency motion filtered by the isolation, whose central frequency is several times lower than that of the structure, leads to high ductility demands in the superstructure, if it cannot resist elastically the long period (relative to its own) acceleration pulses. This is the reason behind *clauses 10.10(5)* and *10.10(4)* in EN 1998-1, which state that '*the resistance condition of the structural elements of*

*the superstructure may be satisfied taking into account seismic action effects divided by a behaviour factor not greater than 1.5*' and '*the structural elements of the substructure and the superstructure may be designed as non-dissipative*'. This results in considerable savings in the design of the structure and of the foundation, due to the lower strength demand and detailing requirements. These savings can counterbalance and even recover the costs of the isolator units and the special arrangements at the isolation interface. On the other hand, full isolation also implies that an isolated structure undergoes strong earthquakes without any damage, which also implies considerable savings when looking at the expected costs during the entire lifetime of a building. Moreover, the possibility of using a linear elastic structural model is a guarantee of a higher level of accuracy in the analysis, which eliminates the need for 'estimating' the non-linear response from the calculated linear response, as the $q$ factor method implies for conventional structures, as well as the use of rather crude non-linear models. In other words, the analyses performed for the purposes of seismic design, due also to the accurately determined characteristics of the isolation devices, are much more reliable than for conventional structures.

Isolation systems can be made of more than one type of component (*clause 10.2(1)*), each one assigned to one or more specific tasks:

- bearing vertical loads
- providing adequate resistance to horizontal non-seismic actions (wind, traffic loads, etc.)
- assuring high flexibility under seismic actions
- dissipating an adequate amount of energy
- re-centring the structure, in order to reduce the residual displacement after the earthquake.

The benefits arising from the large reduction of the response acceleration can be summarized as:

- A reduction of the inertia forces on the structure, so as to eliminate damage to structural elements, even under strong earthquakes
- a drastic reduction of the interstorey drifts, so as to eliminate damage to non-structural elements and to permit the continued use of a building even after a strong earthquake
- a high protection of the building contents
- reduced vibrations felt by the people inside the building, resulting in less panic during an event.

All these aspects represent significant economic and social advantages after an earthquake, while the additional construction costs (due to the cost of the devices and their placement in the structure) are partially or even totally counterbalanced by the savings in the superstructure and foundation costs. The cost percentage of seismic isolation in a building is a function of several parameters, such as:

- the dimensions of the buildings and, especially, the number of storeys
- the configuration of the building in relation to a convenient arrangement of the isolation interface
- the structural layout, in relation to the number of devices necessary to realize the isolation system
- the frequency content of the design action, in relation to the reduction of the effects obtained by the period elongation
- the presence of adjacent buildings, in relation to the need for wide separation joints and to the relevant architectural and equipment problems.

The number of storeys has a significant influence on the cost fraction of seismic isolation. It can be unfavourable both if it is too small (one or two storeys), due to the larger additional cost percentage of the devices and of the constructional arrangements, and if it is too large

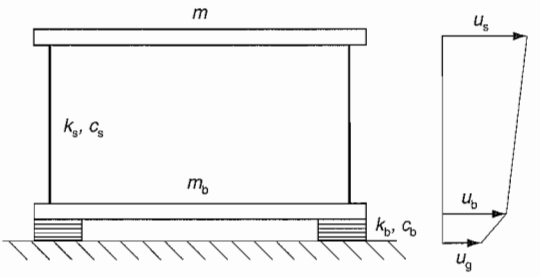



**Fig. 9.4.** Simplified two-DOF model of a base-isolated structure

(more than 8–10 storeys), due to the long fundamental period of the fixed-base structure, which reduces the effectiveness of seismic isolation.

The advantages of seismic isolation can be especially appreciated for buildings that must remain operational after a strong earthquake, such as hospitals and centres for the emergency management, buildings with very high value contents (e.g. museums) and high-risk plants.

Due to the peculiar characteristics and behaviour of structures with seismic isolation, specific criteria and rules must be used in their design, construction and maintenance. For this reason, a separate section of EN 1998-1 – and this corresponding chapter – is exclusively devoted to seismic isolation.

## 9.2. Dynamics of seismic isolation

The basic concepts of seismic isolation of buildings can be illustrated with reference to a simple two-degrees-of-freedom (DOF) system with concentrated masses, such as that shown in Fig. 9.4,[93] where $m_s$ is the mass of the superstructure, $m_b$ is the mass of the base floor, just above the isolation system, $k_s$ and $c_s$ are the stiffness and damping coefficients of the superstructure, and $k_b$ and $c_b$ are the stiffness and damping coefficients of the visco-elastic isolation system. The two DOFs correspond to the horizontal displacements of the two masses, $u_s$ and $u_b$, while $u_g$ is the horizontal ground displacement.

In order to obtain an easier interpretation of the behaviour of the system, the equation of motions will be formulated in terms of relative displacements $v_b$ and $v_s$, i.e. isolation and interstorey displacements: $v_b = u_b - u_g$ and $v_s = u_s - u_b$.

The equation of motion can be easily formulated by applying the d'Alembert principle:

$$(m_s + m_b)\ddot{v}_b + m_s\ddot{v}_s + c_b\dot{v}_b + k_b v_b = -(m_s + m_b)\ddot{u}_b \tag{D9.1}$$

$$m_s\ddot{v}_b + m_s\ddot{v}_s + c_s\dot{v}_s + k_s v_s = -m_s\ddot{u}_b \tag{D9.2}$$

If two separate systems are considered, one having the total mass $(m_s + m_b)$ and the stiffness and damping constants of the isolation system ($k_b$ and $c_b$), the other representing the superstructure as fixed just at its base, thus having mass, damping and stiffness defined by $m_s$, $c_s$ and $k_s$, then the circular frequencies $\omega_b$ and $\omega_s$, the periods $T_b$ and $T_s$ and the damping ratios $\xi_b$ and $\xi_s$ of the two **separate** systems can be defined as

$$T_b = \frac{2\pi}{\omega_b} = 2\pi\sqrt{\frac{m_s + m_b}{k_b}} \qquad T_s = \frac{2\pi}{\omega_s} = 2\pi\sqrt{\frac{m_s}{k_s}} \tag{D9.3}$$

$$\xi_b = \frac{c_b}{2\omega_b(m_s + m_b)} \qquad \xi_s = \frac{c_s}{2\omega_s m_s} \tag{D9.4}$$

as well as the mass and frequency ratios, $\gamma$ and $\varepsilon$:

$$\gamma = \frac{m_s}{m_s + m_b} \qquad \varepsilon = \frac{\omega_b^2}{\omega_s^2} = \frac{k_b m}{(m_s + m_b)k_s} = \left(\frac{T_s}{T_b}\right)^2 \tag{D9.5}$$

As already mentioned, the isolation system is usually much more flexible than the structure ($k_b \ll k_s$), while the superstructure mass is much greater than the base mass ($\gamma \cong 1$), so that $\varepsilon \ll 1$. Modal analysis of equations (D9.1) and (D9.2) yields the following expressions for the properties of the two modes:

Eigenvalues: $\omega_1^2 = \omega_b^2(1-\gamma\varepsilon)_v^2 \qquad \omega_2^2 = \omega_s^2 \dfrac{1+\gamma\varepsilon}{1-\gamma}$ (D9.6)

Eigenvectors: $\phi_1^T = \{1, \varepsilon\} \qquad \phi_2^T = \left\{1, -\dfrac{1}{\gamma}[1-(1-\gamma)\varepsilon]\right\}$ (D9.7)

Participation factors: $\pi_1 = 1-\gamma\varepsilon \qquad \pi_2 = \gamma\varepsilon$ (D9.8)

Damping ratios: $\xi_1 = \xi_b(1-\frac{3}{2}\gamma\varepsilon) \qquad \xi_2 = \dfrac{\xi_s + \gamma\xi_b\sqrt{\varepsilon}}{\sqrt{1-\gamma}}\left(1-\dfrac{\gamma\varepsilon}{2}\right)$ (D9.9)

If $S_d(\omega, \xi)$ and $S_a(\omega, \xi)$ are the displacement and (pseudo-)acceleration spectra, the maximum modal displacements can be calculated and combined using the square root of the sum of the squares rule to obtain

$$d_{b,\max} = \sqrt{(1-\gamma\varepsilon)^2[S_d(\omega_1, \xi_1)]^2 + \gamma^2\varepsilon^2[S_d(\omega_2, \xi_2)]^2} \tag{D9.10}$$

$$d_{s,\max} = \varepsilon\sqrt{(1-2\gamma\varepsilon)^2[S_d(\omega_1, \xi_1)]^2 + [1-2(1-\gamma)\varepsilon]^2[S_d(\omega_2, \xi_2)]^2} \tag{D9.11}$$

The shear coefficient can be expressed as:

$$C_s = \sqrt{[S_a(\omega_1, \xi_1)]^2 + \varepsilon^2[S_a(\omega_2, \xi_2)]^2} \tag{D9.12}$$

Taking into account that $\varepsilon \ll 1$ and hence that from equations (D9.6)–(D9.9) $\omega_1 \cong \omega_b$, $\pi_1 \cong 1$ and $\xi_1 \cong \xi_b$ and disregarding the terms with $S_d(\omega_2, \xi_2)$ in equations (D9.10) and (D9.11), since $S_d(\omega_2, \xi_2)$ is usually $\ll S_d(\omega_1, \xi_1)$ one arrives at the following approximate expressions:

$$v_{b,\max} = S_d(\omega_b, \xi_b) \qquad v_{s,\max} = \varepsilon S_d(\omega_b, \xi_b) \tag{D9.13}$$

$$C_s = S_a(\omega_b, \xi_b) \tag{D9.14}$$

This result is of great importance for the interpretation of the behaviour of isolated buildings and for their preliminary design: for small values of $\varepsilon$ and for the usual spectral shapes, the isolation system can be designed for a maximum displacement equal to $S_d(\omega_b, \xi_b)$, and the superstructure for a shear coefficient equal to $S_a(\omega_b, \xi_b)$. These values could have been obtained with reference to a simple oscillator, whose mass is equal to the total mass of the superstructure ($m_s + m_b$ in Fig. 9.4), and the stiffness and damping are equal to the corresponding quantities of the isolation system ($k_b$ and $\xi_b$). The interstorey drift, $v_{s,\max}$, on which the damage produced by an earthquake depends, is proportional to the frequency ratio $\sqrt{\varepsilon}$ and to the maximum base displacement $S_d(\omega_b, \xi_b)$.

Tables 9.1 and 9.2 show four examples of results obtained by assuming realistic values of the quantities defining the mass, stiffness and damping characteristics of the two-DOF system, referred to a response spectrum relevant to ground type A and $a_g = 0.35g$. As can be seen, the single-DOF approximation provides well-approximated values of the design parameters $v_{b,\max}$, $v_{s,\max}$ and $C_s$ (1–2% difference).

Given the good single-DOF approximation, response spectra become direct design tools, providing maximum base (isolator) displacement $S_d(\omega_b, \xi_b)$ and maximum base shear $(m_s + m_b)S_a(\omega_b, \xi_b)$. It is then useful to refer to the acceleration–displacement response spectrum (ADRS) representation, as shown in Figs 9.5 and 9.6, where spectral displacements and pseudo-accelerations are plotted along the abscissa and ordinate, respectively. The straight line passing through the origin, the slope of which is equal to $\tan\theta = S_a/S_d = 4\pi^2/T^2$, provides the period.

**Table 9.1.** Four examples of base isolation modelled as two-DOF systems (basic parameters, equations (D9.3)–(D9.5))

| Case | $\xi_s$ | $\xi_b$ | $m_s$ (t) | $m_b$ (t) | $k_s$ (kN/m) | $k_b$ (kN/m) | $\omega_b^2$ ((rad/s)$^2$) | $\omega_s^2$ ((rad/s)$^2$) | $T_b$ (s) | $T_s$ (s) | $\varepsilon$ | $\gamma$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.05 | 0.1 | 100 | 50 | 20 000 | 1000 | 6.667 | 200 | 2.432 | 0.444 | 0.033 | 0.667 |
| 2 | 0.05 | 0.2 | 100 | 50 | 20 000 | 1000 | 6.667 | 200 | 2.432 | 0.444 | 0.033 | 0.667 |
| 3 | 0.05 | 0.1 | 100 | 20 | 20 000 | 1000 | 8.333 | 200 | 2.175 | 0.444 | 0.042 | 0.833 |
| 4 | 0.05 | 0.1 | 100 | 50 | 10 000 | 1000 | 6.667 | 100 | 2.432 | 0.628 | 0.067 | 0.667 |

**Table 9.2.** Four examples of base isolation modelled as two-DOF systems: modal values, equations (D9.6)–(D9.9) and maximum displacements or base shear coefficient, exact (equations (D9.10)–(D9.12)) and approximate (equations (D9.13) and (D9.14))

| $\omega_1^2$ ((rad/s)$^2$) | $\omega_2^2$ ((rad/sec)$^2$) | $T_1$ (s) | $T_2$ (s) | $\pi_1$ | $\pi_2$ | $\xi_1$ | $\xi_2$ | $S_{d1}$ (m) | $S_{d2}$ (m) | $v_{b,\,max}$ (m) | $v_{s,\,max}$ (m) | $C_s$ | Case |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6.52 | 613 | 2.46 | 0.25 | 0.978 | 0.022 | 0.097 | 0.106 | 0.144 | 0.018 | 0.141 | 0.005 | 0.937 | 1 |
| **Approximate values** | | | | **1** | | **0.1** | | **0.142** | | **0.142** | **0.005** | **0.948** | **1** |
| 6.52 | 613 | 2.46 | 0.25 | 0.978 | 0.022 | 0.193 | 0.127 | 0.112 | 0.017 | 0.109 | 0.004 | 0.728 | 2 |
| **Approximate values** | | | | **1** | | **0.2** | | **0.110** | | **0.110** | **0.004** | **0.734** | **2** |
| 8.04 | 1241 | 2.21 | 0.18 | 0.965 | 0.035 | 0.095 | 0.161 | 0.145 | 0.011 | 0.140 | 0.006 | 1.164 | 3 |
| **Approximate values** | | | | **1** | | **0.1** | | **0.142** | | **0.142** | **0.006** | **1.185** | **3** |
| 6.37 | 313 | 2.49 | 0.36 | 0.956 | 0.044 | 0.093 | 0.114 | 0.145 | 0.024 | 0.139 | 0.009 | 0.926 | 4 |
| **Approximate values** | | | | **1** | | **0.1** | | **0.142** | | **0.142** | **0.009** | **0.948** | **4** |

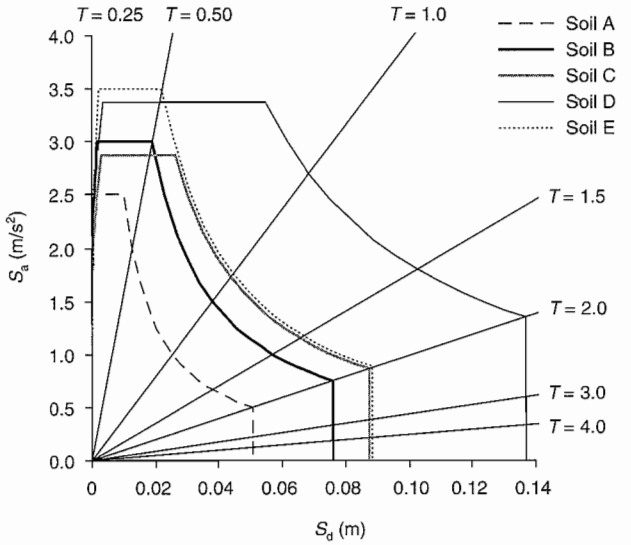



**Fig. 9.5.** ADRS representation of the normalised 5% type 1 elastic response spectra of Eurocode 8, for different soil categories

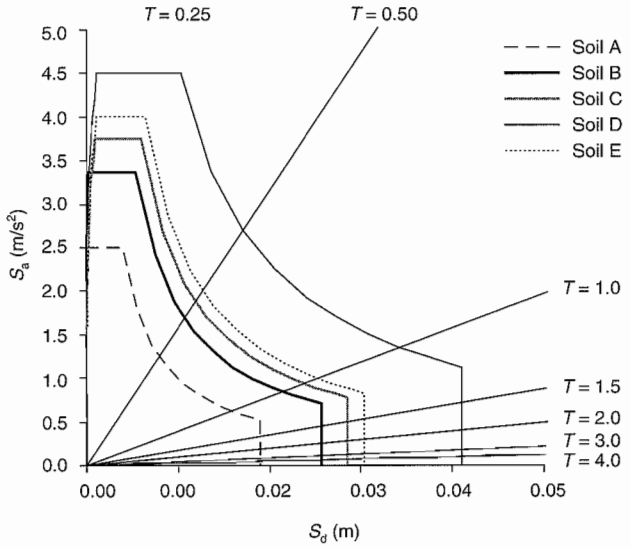



**Fig. 9.6.** ADRS representation of the normalised 5% type 2 elastic response spectra of Eurocode 8, for different soil categories

The type 1 and type 2 recommended spectra (see EN 1998-1) shown in Figs 9.5 and 9.6 apply for 5% damping, and are normalized to unit $a_g$ acceleration. In order to obtain the actual value of $S_d$ and $S_a$ for the seismic zone and damping of interest, they must be multiplied by the factor $a_g\eta$, where $a_g$ is the reference PGA (on type A ground, i.e. rock) and $\eta = \sqrt{[10/(5 + \xi\%)]}$ is the damping correction factor (see *equation (3.6)* in *clause 3.2.2.2* in EN 1998-1). The same spectral representation can be easily used either to evaluate the response parameters from the isolation period and damping or to derive the required period and damping for a desired base shear and isolation displacement.

To better explain the use of the diagrams in Figs 9.5 and 9.6, consider a structure isolated with an effective isolation period $T_{eff} = 2$ s and effective damping $\xi_{eff} = 10\%$, in a site characterized by a type 1, ground type C response spectrum and $a_g = 0.35g = 3.43$ m/s$^2$. From

the normalized spectra of Fig. 9.5, assuming that $\eta = 0.816$, then $v_{b,\max} = a_g \eta S_d(\omega_b, \xi_b) = 3.43 \times 0.816 \times 0.087 = 0.245$ m, $C_s = a_g \eta S_a(\omega_b, \xi_b) = 3.43 \times 0.816 \times 0.863 = 2.418$ m/s$^2$.

## 9.3. Design criteria

*Clauses 10.4(1), 10.6(2), 10.6(3), 10.5.1, 10.5.2, 10.5.3, 10.5.4*

Isolated buildings (*clause 10.3(1)*), like conventional ones, must comply with two limit states (*clause 2.2.1(1)*), which refer to seismic actions having the same return periods, and hence the same intensities as for conventional buildings, and are defined in terms of the 5%-damped elastic response spectra. However, the damping reduction coefficient $\eta$ assumes a greater importance, as practically all the isolation systems have some energy dissipation capacity, resulting in an overall damping larger than 5%. The minimum allowed value of 0.55 for $\eta$ (see *clause 3.2.2.2(3)* in EN 1998-1) implies that the maximum value of damping that can be considered is 28%.

Site-specific spectra must be utilized for importance class IV buildings, close to a potentially active fault with magnitude $M \geq 6.5$. It should be noted that these ground motions can generate unexpectedly large displacements at the isolation level, which can compromise the proper behaviour of the isolation devices.

Several general design provisions are given in *clause 10.5*, aimed at optimizing the behaviour of an isolated building. Most of them are crucial for the effectiveness of seismic isolation and for satisfactory behaviour of the isolated structure, and should be carefully considered when starting the design (both architectural and structural) of a building. They are mainly concerned with:

- the arrangement of the isolation devices in the structure (*clause 10.5.1*), to allow for their inspection, maintenance and replacement
- protection of the isolation devices against fire and chemical and biological attack
- the stiffness distribution in the isolation system, in order to minimize torsion effects (*clause 10.5.2*)
- the stiffness of the structural elements below and above the isolation interface (*clause 10.5.3*), which must be high enough to avoid differential movements
- the space around the superstructure (*clause 10.5.4*), which must allow it to move freely, so that seismic isolation can provide its full beneficial effect and no hammering occurs between isolated and fixed parts.

## 9.4. Seismic isolation systems and devices

*Clauses 10.5.1, 10.8, 10.10(7)*

Isolator units of different type and mechanical characteristics can be combined to fulfil the performance requirements of an isolation system, which can be summarized in the following points:

- low stiffness or resistance in the horizontal direction
- high dissipation capacity
- reliable behaviour with respect to non-seismic service actions
- re-centring capacity at the end of an earthquake.

Additional requirements are:

- durability
- easy installation and inspection
- limited cost
- invariability of mechanical behaviour.

Several isolation devices have been proposed and used in the last 30 years[91–93,98,99] in seismic isolation systems. To clarify their roles, a primary distinction can be made between 'isolators' and 'supplementary devices'. The devices in each category can be further classified according to their mechanical behaviour (force–displacement relationship).

### 9.4.1. Isolators

Isolators are the fundamental components of an isolation system, as they carry the weight of the construction and transmit it to the substructure. Essentially they are bearing devices which permit large relative translations (of the order of 200–400 mm) in at least one or both of the horizontal directions. Depending on their function as bearings, they may have to primarily satisfy the requirements for bearings. The most widely used types of isolators are elastomeric isolators and sliding bearings. Their idealized behaviour is shown in Fig. 9.7.

Elastomeric bearings are characterized by almost elastic behaviour. Some energy dissipation, however, takes place, and varies depending on the type of rubber compound used: 5–20% equivalent damping from low- (LDRBs) to high- (HDRBs) damping rubber bearings. Energy dissipation capabilities can be improved by inserting a lead plug (lead–rubber bearings, LRBs) or other highly energy-dissipating material in a cylindrical hole. In HDRBs and LRBs, secant stiffness and equivalent damping are considerably higher for low shear deformations (about 10–20%). This is beneficial to limit displacements under frequent actions such as wind. Minimum values of these two quantities are observed for shear deformations in the range of 100–150%, which is also the usual working range under design seismic actions. For shear deformations above this range, both the stiffness and the equivalent damping tend to increase sharply. This can be beneficial to limit displacements under anomalous or extreme earthquakes, but can be dangerous for the structural elements since the forces induced in the structure are higher than the design ones.

Elastomeric bearings can be made of natural or artificial rubber or a mixture of them. They are usually reinforced with steel shims, operating as confining elements, in order to increase their bearing capacity and reduce their vertical deformability. In this respect, two important geometrical characteristics are crucial in the design of these devices: the primary $S_1$ and the secondary $S_2$ shape factors. $S_1$ is the ratio between the cross-section and the lateral surface areas of each rubber layer, and affects the vertical stiffness, which varies almost proportionally to its square; $S_2$ is equal to the ratio between the diameter and the total thickness of rubber, and affects mainly the bearing capacity, in relation to buckling phenomena. The shear modulus of rubber, $G$, usually ranges from 0.35 to 0.4 MPa up to 1–1.4 MPa. The current trend is to use low $G$ moduli to reduce the total thickness of the device or to increase the isolation period, the horizontal stiffness of an isolator being equal to $GA/t_e$, where $A$ is the horizontal cross-sectional area and $t_e$ is the total rubber thickness. Well-manufactured devices have been shown to sustain shear deformations up to 500%, hence design shear deformations up to 150–200% could be safely permitted.

Highly beneficial properties of rubber bearings are their self-centring capability and the stability of their mechanical behaviour for a very large number of cycles. The occurrence of a strong earthquake does not require, in principle, any substitution or maintenance operations. Many studies on durability have been carried out. Procedures for accelerated ageing tests on the rubber compounds have been established and are easy to perform. The observation of rubber bearings installed in the past indicates that durability should not be a

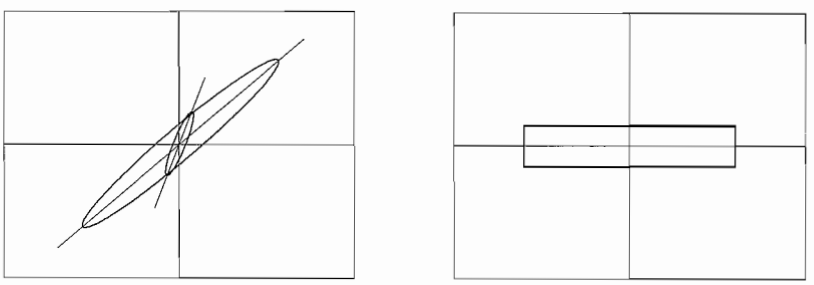



**Fig. 9.7.** Idealized force–displacement curves of (a) laminated rubber isolators and (b) sliding isolators

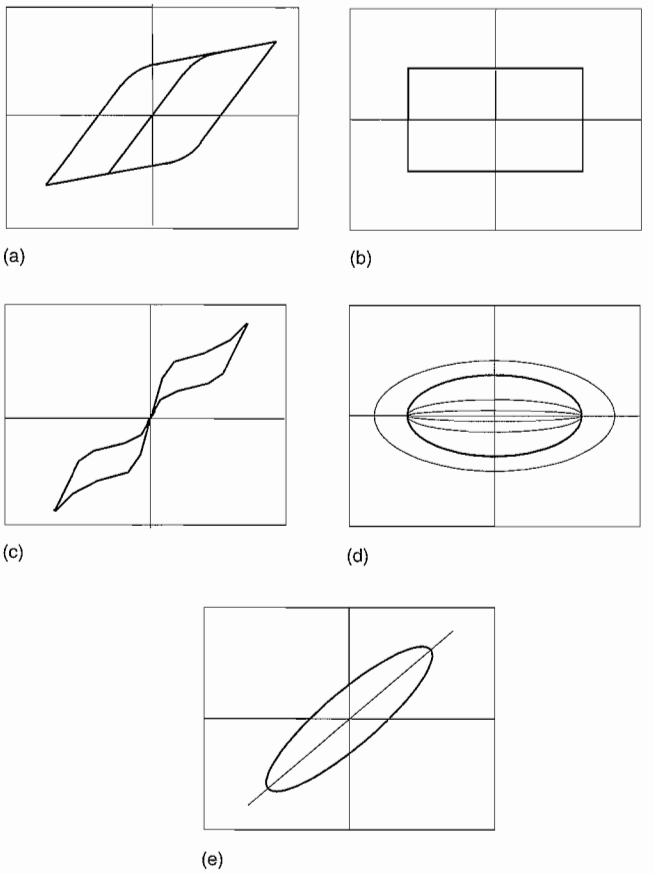



**Fig. 9.8.** Idealized force–displacement curves of supplementary devices based on (a) hysteresis of metals, (b) friction, (c) superelasticity of shape-memory alloys, (d) fluid viscosity and (e) visco-elasticity

problem under normal conditions, so that more than 50 years of useful life can be assumed under these conditions.

Sliding bearings exploit the low-friction characteristics between special materials (e.g. polished stainless steel and PTFE). The theoretical behaviour can be described in general terms as rigid–plastic, although velocity and pressure can induce variations (100% and more in steel–PTFE contact), thus modifying the theoretical cycle. The friction coefficient may also depend on temperature, humidity, contamination and ageing. Considerable reduction of the friction coefficient, from 5 to 10%, down to 0.5–2%, can be obtained by lubrication of the surfaces. Lubricated sliding bearings have often been used in Europe as isolators without relying on their energy dissipation capability. Different shapes of the surfaces (e.g. spherical instead of plane) allow various force-displacement relationships and even self-centring capability to be obtained.

Roller bearings made of cylinders or spheres provide very low resistance to horizontal movements, but are rarely used due to their cost.

### 9.4.2. Supplementary devices

Supplementary devices are installed separately from isolators to complete the isolation system. Very often they are made of simple energy-dissipating elements (EDEs). EDEs are sometimes embodied in isolators to improve their energy-dissipating capabilities and avoid the use of separate devices. Figure 9.8 shows the idealized force–displacement relationships for the most commonly used devices.

Metallic hysteretic devices exploit the high-energy dissipation capacity of metals such as mild steel and lead when stressed beyond the elastic limits. They are quite popular owing to

their reliability and the invariability of their mechanical characteristics in long-term service under the usual environmental conditions. On the other hand, after a strong earthquake the structure may need to be recentred. Mild steel EDEs, i.e. special pieces of steel purposely shaped for maximum efficiency in energy dissipation through bending, torsion and/or shear deformations, are often used, due to their low cost, dissipation effectiveness and invariability of mechanical properties. Their main fault is limited resistance to low-fatigue cycles. It is envisaged that they will be replaced after a strong earthquake.

Special alloys, such as shape-memory alloys, show very desirable characteristics, and 'intelligent behaviour' can be obtained. They can be used in both energy-dissipating devices and recentring devices, by exploiting their superelastic properties and their considerable low-fatigue resistance.

Friction dampers exploit the friction between suitably treated surfaces. They provide high potential at a low cost. However, two problems need to be solved before they can be considered as fully reliable. First, the static and dynamic friction coefficients should have, as far as possible, the same value, and should not depend on velocity, environmental conditions, and the long periods of time during which the two contacting surfaces do not move with respect to each other. Second, the normal force should not vary during their lifetime.

Visco-elastic dampers exploit the visco-elastic characteristics of special materials to obtain a cyclic behaviour in which the viscous component provides some energy dissipation capacity at the deformation rate produced by an earthquake. Materials normally used are special polymers or even high-damping elastomers. The visco-elastic material is normally stressed in shear, and various configuration of the devices can be adopted.

Viscous-fluid dampers exploit the viscosity of some fluids (usually oil or silicon) to achieve the typical elliptical viscous force–displacement behaviour. The reacting force and, therefore, the amplitude of the cycle depend strongly on speed. The reaction is practically zero when relative movements are very low, as in the case of thermal structural deformation. These dampers are therefore particularly suitable for long structures, such as bridges, where thermal movements are important and devices which give always fixed conditions cannot be used. Hydraulic jacks with calibrated orifices or valves are normally utilized, which, along with the use of fluids of different viscosities, make viscous-fluid dampers very versatile, as different force–displacement shapes can be obtained. The main problems of this type of dampers are related to their long-term reliability and their maintenance.

## 9.5. Modelling and analysis procedures

*Clauses 10.8(1), 10.9.1(3), 10.9.2, 10.9.3, 10.9.4, 10.9.5*

The requirement of no inelastic deformation in the structural elements (*clause 10.4(6)* in EN 1998-1) simplifies the modelling and the analysis of isolated structures, as all the structural elements can be assumed to behave linearly. On the other hand, the crucial role played by the isolation system requires specific care in its modelling and, in case it exhibits strongly non-linear behaviour, in the analysis procedures. First of all the variability of the mechanical properties of the seismic isolation system must be taken into account, as the acceleration transmitted to the structure, and hence the seismic forces, may be highly sensitive to it (*clause 10.8(1)*). The most unfavourable values of the mechanical properties presumably attainable during the lifetime of the structure should be used, consistently with the range of variation of the physical parameters (temperature, rate of loading, vertical loads, etc.) affecting them. Moreover the actual distribution of the isolator units with their relevant mechanical properties must be correctly taken into account in the model.

Different types of analysis can be used, according to the behaviour of the isolation systems, as summarized in Fig. 9.9.

Generally speaking, the behaviour of an isolation system under cyclic actions is more or less non-linear. Under certain conditions, an equivalent linear visco-elastic force–deformation relationship can be assumed for the isolation system and a simplified static

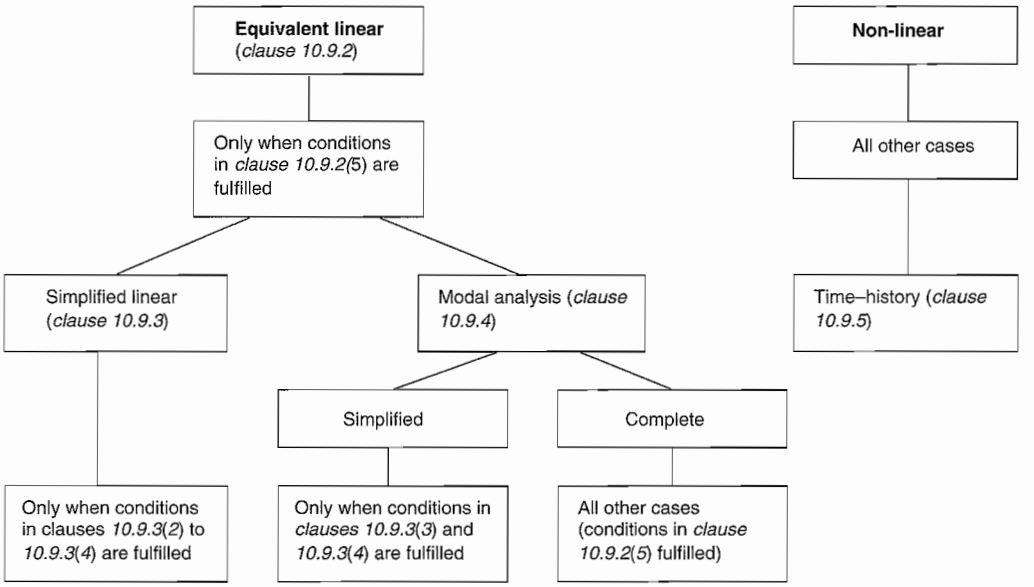



**Fig. 9.9.** Summary of the types of analyses for isolated structures and the conditions to be met for their application

analysis or a linear modal dynamic analysis can be adopted, using effective values of stiffness ($K_{\text{eff}}$) and damping ($\xi_{\text{eff}}$), which correspond to the values of these variables at the total design displacement $d_{\text{dc}}$ of the system. EN 1998-1 requires the following conditions to be met (*clause 10.9.2(5)*):

- The effective stiffness (secant) of the isolation system is at least 50% of the stiffness at $0.2d_{\text{dc}}$ (where $d_{\text{dc}}$ is evaluated at the stiffness centre of the isolation system).
- The effective damping of the isolation system is lower than or equal to 30%. In fact, high values of damping can introduce modal coupling and hence increase floor accelerations and base shear, an effect which is neglected in standard dynamic modal and static analysis, as illustrated in Naeim and Kelly.[93]
- The mechanical characteristics do not vary by more than 10%, due to the rate of loading effects and vertical load variations in the range of the design values.
- The increase of force in the isolation system for a displacement between $0.5d_{\text{dc}}$ and $d_{\text{dc}}$ is at least 2.5% of the total superstructure weight, to provide a minimum restoring effect and avoid accumulation of displacements.

The simplified linear analysis is the simplest approach for the calculation of seismic action effects on isolated structures. All the steps of this procedure are described in *clause 10.9.3* of EN 1998-1. Its theoretical basis is the approximate approach to the dynamics of seismic isolation described in Section 9.2 above. The structural system is assumed to behave like a simple oscillator, whose mass is the total mass of the superstructure and the stiffness is the total stiffness of the isolation system, for each direction of the seismic action. This type of analysis can be applied only when the conditions summarized in Table 9.3 are met.

Isolation displacements and seismic forces are evaluated with the simple equations in *clauses 10.9.3(1)* to *10.9.3(6)*. The seismic forces are then statically applied to the superstructure to evaluate the design quantities for the safety verifications (element stresses, interstorey drift, etc.). It should be emphasized that the distribution of the seismic forces along the height is no longer defined by a linearly varying distribution of the acceleration but rather by a constant distribution (*clause 10.9.3(6)*). This is consistent with the hypothesis that the superstructure moves like a rigid body, with negligible deformation (interstorey displacements much smaller than isolation displacement). Global torsion effects on the structure, which should be mainly due to accidental eccentricities, are approximately taken

**Table 9.3.** Summary of the conditions to be met for the application of the simplified linear analysis

| Clause in EN 1998-1 | Conditions |
|---|---|
| *10.9.3(2)* | Maximum eccentricity between mass and isolation stiffness centres: ≤ 7.5% |
| *10.9.3(3)* | Distance from faults producing $M \geq 6.5$: > 15 km |
| *10.9.3(3)* | Maximum plan dimension: ≤ 50 m |
| *10.9.3(3)* | Rigid substructure |
| *10.9.3(3)* | All devices located above elements of substructure that support vertical load |
| *10.9.3(3)* | Period range: $3T_f \leq T_{eff} \leq 3.0$ s |
| *10.9.3(4)* | Plan regularity and symmetry of the superstructure |
| *10.9.3(4)* | Negligible rocking rotation at the base of the superstructure |
| *10.9.3(4)* | Ratio between vertical and horizontal stiffness of isolation system: $K_v/K_{eff} \geq 150$ |
| *10.9.3(4)* | Maximum vertical vibration period: $T_v \leq 0.1$ s |

into account as in *clause 4.3.3.2.4* of EN 1998-1 for symmetrical conventional structures, by simply increasing the effects in individual structural members according to their distance from the centre of mass. For the calculation of the total design displacement of each isolator unit in a given direction, amplification factors are to be applied that depend on the position of the unit, on the total mass–stiffness eccentricity in the direction normal to the seismic action and on the torsional stiffness radius of the isolation system.

Although not always applicable, this kind of analysis is extremely useful in any case for preliminary design and global checks of the results of more sophisticated analyses.

If the isolation system may be modelled as linear, but some of the conditions of Table 9.3 are not met, the structural system should be analysed with a modal dynamic analysis (*clause 10.9.4*), where both the superstructure and the isolation system are modelled as linear elastic. A simplified model, with a rigid mass with three horizontal degrees of freedom simulating the superstructure, may still be used (*clause 10.9.4(2)*) when only the condition in *clause 10.9.3(2)* cited in Fig. 9.9 is not fulfilled, i.e. when the torsional displacement can produce significant differences in the displacement of the isolation system. These differences must be taken into account when evaluating the effective stiffness of the single isolator units.

A time–history analysis can always be carried out, for any type of structure and isolation system. It is compulsory if it is not possible to model the mechanical behaviour of the isolation system as equivalent linear (conditions in *clause 10.9.2(5)*). Obviously, only the isolation system needs to be modelled as non-linear, while a linear model is retained for the structure. The non-linear model should represent the actual constitutive law of the isolation system in the actual range of deformations and velocities related to the seismic design situation.

## 9.6. Safety criteria and verifications

*Clauses 10.4(1), 10.4(2), 10.4(3), 10.4(5), 10.4(6), 10.4(8), 10.3(2), 10.10(5), 10.10(6)*

While the safety verifications for the damage limitation state are essentially the same as for conventional buildings (*clause 10.4(3)*), with the additional requirement that all lifelines crossing the joints around the isolated structure should remain within the elastic range, the safety verification for the ultimate limit state (ULS) has some differences. The full isolation requirements result in checking that the design strengths equal the stresses obtained from the analysis for the substructure, and divided by 1.5 for the superstructure (*clause 10.10(5)*).

The crucial role played by the isolation devices is recognized in the safety verification. A magnification factor $\gamma_x$, to be defined in the National Annexes, must be applied to the design displacements of the devices, for their ULS verifications. The same factor has to be applied also to the ULS verification of the gas lines and other hazardous lifelines crossing separation joints to be designed to accommodate with a large safety margin the relative displacements they are subjected to.

## 9.7. Design seismic action effects on fixed-base and isolated buildings

The question of the economic advantages of structures protected by seismic isolation is often raised. This issue should be correctly addressed, looking at the overall expected costs, which include, in addition to the initial cost, the cost of repair and replacing damaged contents, as well as casualties and social costs. Nevertheless, it is also important to obtain an estimate of the initial costs, which is an important component of the overall expected cost.

The additional cost of seismic isolation is due to the cost of the devices and of the structural arrangement at the isolation interface. Savings can result from the reduction in the seismic forces acting on the structure. Since the possible savings are strictly related to the seismic forces, it is of interest to compare the design stresses on an isolated structure and on its fixed-base counterpart. Such a comparison can help the designer to make the basic choice of whether to utilize seismic isolation or not and, eventually, to optimize its application.

The stresses on a fixed-base framed concrete structure and on a similar isolated structure need to be compared in terms of base shear on the superstructure. Therefore, the ratio between the design spectral accelerations of a fixed-base structure $S_d(T_{fb}, q)$ multiplied by the effective mass ratio and that of a similar seismic-isolated structure $S_e(T_{eff})$ divided by the behaviour factor of 1.5 for the superstructure (*clause 10.10(5)*) is calculated, where $T_{fb}$ is the period of the fixed-base structure and $T_{eff}$ the period of the isolated structure.[100] The effective mass ratio $\lambda$ is taken as equal to 0.85, as prescribed for the equivalent linear static analysis, if the structure has at least two storeys and its vibration period is $T_{bf} < 2T_c$. It is taken equal to 1 for the isolated structure (*clause 10.9.3(6)*).

The comparison is referred to both the ULS and the damage limit state (DLS). For the ULS of the fixed-base structure, the design spectral ordinate depends on the behaviour factor $q$, which is related to the ductility class and to the regularity of the superstructure along the height. The comparison is referred to the higher value of $q$ for concrete structures. Thus, the case with $q = 4.5\alpha_u/\alpha_1$ (Ductility Class High and regular frames) is examined, assuming the default value: $\alpha_u/\alpha_1 = 1.3$. Reference is made to a typical rubber isolation system, having an equivalent viscous damping ratio equal to $\xi = 10\%$, resulting in a reduction factor of the spectral ordinate $\eta = 0.816$. The seismic force ratio is then given by

$$R_{ULS} = \frac{\lambda S_d(T_{fb}, q)}{S_e(T_{eff}, \xi_{eff})/1.5} = \frac{1.56}{q}\frac{S_e(T_{fb})}{S_e(T_{eff})} \quad \text{for } T_b \le T_{bf} \le 2T_C \text{ and } \xi_{eff} = 10\% \qquad \text{(D9.15)}$$

As far as the DLS is considered, no reduction factor $q$ has to be considered, and the seismic shear ratio becomes

$$R_{DLS} = \frac{\lambda S_e(T_{fb})}{S_e(T_{eff}, \xi_{eff})} = 1.04 \cdot \frac{S_e(T_{fb})}{S_e(T_{eff})} \quad \text{for } T_b \le T_{bf} \le 2T_C \text{ and } \xi_{eff} = 10\% \qquad \text{(D9.16)}$$

Looking at equations (D9.15) and (D9.16), it is clear that the period ratio plays a primary role in deciding the economic convenience of seismic isolation, as the spectral ratio depends mainly on it. Moreover, the advantages of seismic isolation are much more remarkable for the DLS, since no $q$ factor enters into the equation, which results in $R_{DLS}/R_{ULS} = 1.04q/1.56 \cong 4$.

Figures 9.10 and 9.11 show the $R_{ULS}$ values plotted with the isolated structure period along the abscissa. They refer to two extreme soil conditions, i.e. ground types A and D (*clause 3.1.2* in EN 1998-1), giving, respectively, the most and the least favourable conditions for seismic isolation. The higher the effective period, the more favourable the $R_{ULS}$ ratio becomes to seismic isolation. Focusing attention on the usual range of application of rubber isolation, 2.0 s $\le T_{eff} \le$ 3.0 s and considering an isolation ratio of at least 2, it can be seen that the seismic force ratio $R_{ULS}$ varies between 0.63 ($T_{fb}$ = 1.0 s, $T_{eff}$ = 2.0 s) and 3.0 ($T_{fb} \le$ 0.4 s, $T_{eff}$ = 3.0 s) for ground type A and 0.54 ($T_{fb}$ = 1.0 s, $T_{eff}$ = 2.0 s) and 1.5 ($T_{fb} \le$ 0.4 s, $T_{eff}$ = 3.0 s) for ground type D. It should be emphasized that further savings are implied by the fact that

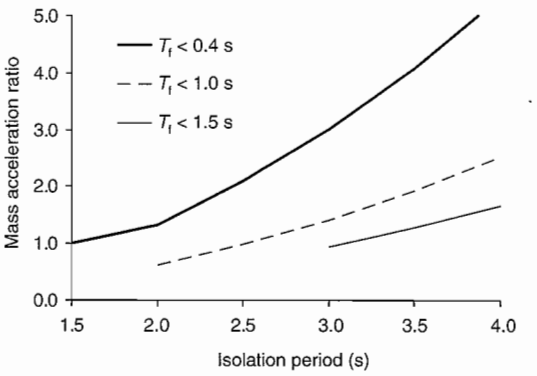



**Fig. 9.10.** Base shear ratio for ground type A (fixed base with $q = 5.85$, base isolated with $\xi = 10\%$)

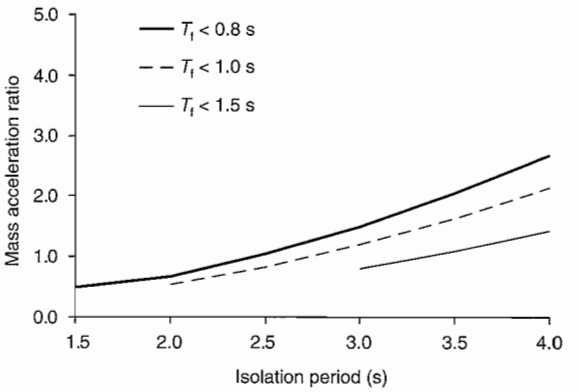



**Fig. 9.11.** Base shear ratio for ground type D (fixed base with $q = 5.85$, base isolated with $\xi = 10\%$)

neither capacity design nor seismic detailing is required for structures protected by seismic isolation (*clause 10.4(7)*).

As mentioned above, to obtain $R_{DLS}$, $R_{ULS}$ is multiplied by about 4. Values of $R_{DLS}$ much higher than 1 (ranging from 2.5 to 12 and from 2.2 to 6 for the situations above) are then obtained, emphasizing the significant advantages provided by seismic isolation in terms of non-structural damage control.

CHAPTER 10

# Foundations, retaining structures and geotechnical aspects

## 10.1. Introduction

### 10.1.1. Scope of the Designers' Guide to EN 1998-5

This part of the guide to Eurocode 8 addresses:

- the seismic design actions applicable to soils, foundations and natural or artificial slopes
- the design mechanical properties of soils in the presence of earthquake actions, i.e. their strength, stiffness and damping under cyclic loading
- appropriate models for stability and bearing capacity verifications, soil–foundation (and soil-retaining structure) interaction, including the evaluation of seismically induced deformations.

All aspects related to the structural design and dimensioning of foundations are dealt with in Chapters 4 and 5 of this guide, particularly as regards most of the provisions contained in *clauses 5.2, 5.3.1, 5.4.1.2, 5.4.1.3* and *5.4.2* of EN 1998-5.[2]

### 10.1.2. Relationship between EN 1998-5 and EN 1997-1 (*Eurocode 7: Geotechnical Design. Part 1: General Rules*)

According to the scope stated in *clause 1.1(1)* of EN 1998-5 '*complements Eurocode 7 which does not cover the special requirements of seismic design*', while in the scope of Eurocode 7 (clause 1.1.1(7)) it is stated that 'EN 1998 provides additional rules for geotechnical seismic design, which complete or adapt the rules of this standard'. For a better understanding of the mutual relationships and complementary aspects of the two documents, a few definitions and general concepts from EN 1997-1 are recalled below. Familiarity with EN 1997-1 is especially important when performing the simplified stability verifications referred to as 'pseudo-static' in EN 1998-5.

*Clause 1.1(1)*

#### *10.1.2.1. Common and separate definitions*

**'Ground' versus 'soil'**

In EN 1998-5 the term 'ground' is used according to the definition in EN 1997-1, clause 1.5.2.3, as '*soil, rock and fill in place prior to the execution of the construction works*'. Thus, a classification of ground types is introduced in EN 1998-1 for the purpose of establishing the dependence of the seismic action on the geotechnical characteristics of the construction site.

*Clauses 1.5.2(1), 1.5.2(2)*

**'Shallow' versus 'spread' foundations**

In this case, separate definitions are used in the two codes: in EN 1998-5, 'shallow' foundations include footings and raft foundations, while in EN 1997-1 the same are called 'spread' foundations, and include pads (isolated footings), strips and rafts.

### *10.1.2.2. Geotechnical categories, design values of geotechnical parameters and design approaches*

**Geotechnical categories**

The non-mandatory use of geotechnical categories is introduced in EN 1997-1 (see clauses 2.1(10) to 2.1(19)), to establish geotechnical design requirements. Such categories are not used in EN 1998-5, but it is noted that EN 1997-1 Geotechnical Category 3 applies to structures in highly seismic areas,* and '*should normally include alternative provisions and rules to those in this standard*' (i.e. EN 1997-1). This is interpreted to mean that geotechnical design aspects of structures in highly seismic areas will require specialist treatment, with tools that are not described in EN 1997-1.

**Characteristic and design values of geotechnical parameters**

The concept of characteristic values of the ground properties is not mentioned in EN 1998-5, but its use is understood and obviously intended to comply both with EN 1997-1 and with EN 1990 '*as far as possible, whilst remaining true to principles of sound geotechnical engineering*'.[101] The geotechnical interpretation of the concept has been one of the most controversial topics in the drafting of EN 1997-1. Some of the basic prescriptions and recommendations are given in the following paragraphs of clause 2.4.5.2 of that code:

> (1)P The selection of characteristic values for geotechnical parameters shall be based on derived values resulting from laboratory and field tests, complemented by well-established experience.
>
> (2)P The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the limit state.
>
> ...
>
> (7) The zone of ground governing the behaviour of a geotechnical structure at a limit state is usually much larger than a test sample or a zone of ground affected in an in situ test. Consequently the value of the governing parameter is often the mean of a range of values covering a large surface or volume of the ground. The characteristic value should be a cautious estimate of this mean value.
>
> ...
>
> (11) If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.
>
> (12)P When using standard tables of characteristic values related to soil investigation parameters, the characteristic value shall be selected as a very cautious value.

A useful discussion on characteristic values of geotechnical parameters can be found in Simpson and Driscoll.[101] The relationship of characteristic values to design values is governed by the general prescription of EN 1990, reiterated in EN 1997-1, namely that the design value $X_d$ of a geotechnical parameter is obtained as

$$X_d = X_k / \gamma_M \qquad \text{(D10.1)}$$

*A lower threshold of 0.30–0.35*g* design acceleration on type A ground can tentatively identify 'highly seismic areas'.

where $X_k$ is the characteristic value and $\gamma_M$ is the partial factor for the parameter, subject to national choice. However, the design values of geotechnical parameters can also be assessed directly.

**Design approaches**

EN 1997-1, in clause 2.4.7.3.4, introduces three alternative design approaches to geotechnical problems, denoted here as DA-1, DA-2 and DA-3, among which each country is allowed to make its national choice.* Each Design Approach introduces partial factors that on one hand affect directly the actions, or the action effects, and on the other hand the individual resistances, or the global resistance. For ground material properties, in particular, equation (D10.1) holds. In DA-1, furthermore, a double verification is prescribed using two different combinations of the coefficients.

After introducing within each Design Approach the appropriate partial factors to calculate the design action $E_d$ and the design resistance $R_d$, the safety verification with respect to a limit state of rupture or excessive deformation of a structural element or section of the ground (denoted STR and GEO, respectively, in EN 1997-1) simply requires that equation (D2.3) in Chapter 2 of this guide be satisfied.

Synthetically, the salient features of the EN 1997-1 design approaches are as follows:

- DA-1, first case (DA-1 C-1): partial factors are applied to actions but not to the ground strength parameters, i.e. the corresponding factors are equal to 1.0.
- DA-1, second case (DA-1 C-2): partial factors are applied to ground strength parameters ($\gamma_{\phi'}$ for tan $\phi'$, $\gamma_{cu}$ for the undrained shear strength $c_u$, etc.) but not to actions.
- DA-2: partial factors are applied to actions or directly to action effects, and to the global resistance, but not to ground strength parameters.
- DA-3: partial factors are applied only to structure-generated actions, but not to actions arising in the ground, and also to ground strength parameters (as in DA-1 C-2).

Thus, DA-3 coincides with DA-1 C-2 when structure-generated actions are absent, as may occur in slope stability verifications, for example.

In EN 1998-5, structure-generated actions, such as the inertial loads transmitted to the ground through the foundations, are combined according to the specific rules prescribed in EN 1998-1, *clauses 3.2.4* and *4.2.4*, and in EN 1990.[3] Although Design Approaches are not explicitly mentioned in EN 1998-5, the pseudo-static methods therein recommended for verifying the foundation bearing capacity and the stability of retaining walls assume design values of ground strength parameters in agreement with DA-1 C-2 and DA-3 (in the absence of structure-generated actions). Thus, while DA-1 C-2 and DA-3 are the Design Approaches that are the most compatible with EN 1998-5, other approaches, such as DA-1 C-1, could also be used by the designer, depending on the national choice. Should DA-1 C-1 be adopted, in the stability verification of a slope or of a retaining wall the characteristic values of the ground strength parameters would not be affected by partial factors but that of the ground unit weight ought to be multiplied by the corresponding partial factor ($\gamma_G$).

### *10.1.2.3. Ultimate limit state (ULS) and damage limitation state (DLS)*

Safety verifications in EN 1998-5 address ULSs, i.e. limit states of rupture or excessive deformation in the ground (GEO in EN 1997-1) or in structural elements (STRU in EN 1997-1). They also address damage, or serviceability, limit states.

Preventing the occurrence of GEO or STRU limit states is consistent with the no-collapse requirement set forth in EN 1998-1. DLSs are defined in the latter (clause 2.2.1) as those 'associated with damage beyond which specified service requirements are no longer met'.

*The synthesis of the EN 1997-1 Design Approaches given in the following is taken from Simonelli.[102]

Verifications with respect to DLSs are advocated in EN 1998-5 in the general requirements for:

- Slope stability (*clause 4.1.3.1*): it may not suffice to verify equation (D2.3) to check whether the limit states with '*unacceptably large permanent displacements of the ground mass*' is attained, and actual permanent displacements of the ground mass may have to be computed during an earthquake. Should such computations show that predicted permanent displacements are limited and have no adverse functional effects on the structure, the slope should be considered as safe, and its safety actually controlled by a damage limitation requirement.
- The foundations system (*clause 5.1*), which prescribes that seismically induced ground deformations be compatible with the essential functional requirements of the structure.
- Earth-retaining structures (*clause 7.1*), for which '*Permanent displacements ... may be acceptable if it is shown that they are compatible with functional and/or aesthetic requirements*'. Thus, considerations similar to those introduced for slope stability apply.

## 10.2. Seismic action

### *10.2.1. Design acceleration and importance factors*

*Clause 2.1*

In EN 1998-5, the most frequent representation of the seismic action occurs in the pseudo-static methods for stability verifications, where the action typically takes the non-dimensional form $\alpha S$, with $\alpha = a_g/g$, where $g$ is the acceleration of gravity. In this expression, $a_g$ is the design acceleration on type A ground, given by equation (D3.7), where $a_{gR}$ is the reference peak ground acceleration, to be found in the national seismic zonation map, $\gamma_I$ is the importance factor (see also Section 2.1 of this guide) and $S$ is the site-dependent soil factor, as defined in *clause 3.2.2.2* of EN 1998-1.

When dealing with stability verifications of sufficiently steep slopes, *clause 4.1.3.2(2)* prescribes in addition that for structures with $\gamma_I > 1$, the design acceleration on type A must be multiplied by a topographic amplification coefficient ($S_T$), with recommended values given in *Annex A* of EN 1998-5.

Importance classes and importance factors for buildings are dealt with in *clause 4.2.5* of EN 1998-1 (see also Section 2.1 of this guide), where the highest value of the importance factor $\gamma_I = 1.4$ is recommended for buildings, the integrity of which during earthquakes is of vital importance for civil protection. An importance factor $\gamma_I > 1$ will affect geotechnical seismic design through an increase of:

- inertial loads generated by the structure and transmitted to the foundations
- the so-called 'kinematic forces' acting on deep foundations (piles and piers) as a consequence of the ground deformation generated by seismic wave propagation, because such forces depend on the design acceleration
- earth pressures acting on both basement walls in a building and on adjacent retaining structures required to remain stable in order to ensure the integrity of the building structure
- destabilizing inertia forces in verifications of natural or man-made slopes, the stability of which is required for the integrity of the building structure
- the magnitude of the seismic shear stress to be used in evaluations of the liquefaction susceptibility of saturated, coarse-grained foundation soils.

The list applies almost unchanged (when $\gamma_I > 1$) to bridge structures, for which importance categories and recommended importance factors are given in clause 2.1 of EN 1998-2. This clause also states that 'in general bridges on motorways ... are considered to belong to the category of "average" importance, for which an importance factor $\gamma_I = 1$ applies'. Should, however, a country consider the serviceability of a motorway or highway system during earthquakes as essential for civil protection purposes, an increase in design actions (as

a consequence of applying $\gamma_I > 1$) will affect the totality of the geotechnical structures belonging to the system in question.*

A national choice of this type, not complying with the recommendation in EN 1998-2, has been made, for example, in Italy for 'motorways, national highways, and associated engineering structures',[102] following the release in 2003 of the most recent seismic norms, compatible with Eurocode 8.[103]

### 10.2.2. Topographic amplification factor

The topographic amplification factor ($S_T$) is introduced in *Section 4* of EN 1998-5, as a coefficient increasing the inertia forces for the seismic verification of the stability of slopes; more detailed information on it is provided in *Annex A* of EN 1998-5. Additional background is provided here on the magnitude of this factor. *Clause 4.1.3.2(2)*

The numerical values of $S_T$ were mostly derived from dynamic response analyses of the simple two-dimensional (2D) relief shapes described in *Annex A* of EN 1998-5, using a homogeneous elastic material, vertically propagating sinusoidal waves as excitation, and averaging computed amplitudes at representative points at the top of the analysed configurations. Representative amplification values applicable to the seismic action on top of the slopes are associated in *Annex A* with specific shapes and slope angles, as follows:

- $S_T = 1.2$ (or greater) for isolated cliffs and slopes with slope angle > 15°, as well as for ridges with 15° < slope angle < 30°
- $S_T = 1.4$ (or greater) for ridges with slope angle > 30°.

Strong motion records on topographic amplification are scanty, and detailed study of some of them has shown that site amplification near the top of hills and even mountains is more likely generated by local deposits of soils or layers of weathered rock than by the wave-focusing phenomena caused by locally convex geometry on the Earth's surface. To clarify the extent of the approximations introduced in *Annex A* of EN 1998-5, Table 10.1 shows the results of very accurate three-dimensional (3D) dynamic numerical analyses of four actual Italian sites with prominent topographic relief. The 3D models of these sites were excited by representative recorded accelerograms, and average amplification factors with respect to the excitation were obtained from the 5% damped, elastic response spectra of the motion calculated at several points on top. Table 10.1 compares the topographic amplification factors in *Annex A* of EN 1998-5 with those derived from the 3D analyses, and also from simpler 2D models. The values are in reasonable agreement, except for highly irregular and very steep 3D configurations.

### 10.2.3. 'Artificial' versus recorded time–history representations

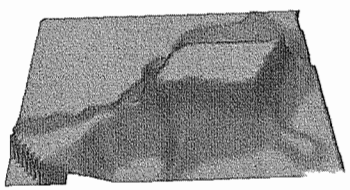
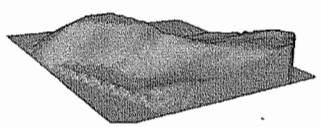
In the safety verifications contemplated in EN 1998-5, time–domain analyses come into play in non-linear dynamic calculations, needed to evaluate, for example, seismically induced permanent displacements of a slope or of a retaining work, or to assess the amplification response of the ground profile at a site. *Clauses 2.2(1), 2.2(2)*

Acceleration–time series generated so as to match the design elastic response spectrum with purely mathematical (stochastic) tools, disregarding the physics of the earthquake source and the wave propagation process, are commonly referred to as **artificial** accelerograms, as opposed to strong motion accelerograms **recorded** in real earthquakes.

The Eurocode 8 elastic spectra, similar to other codes, do not represent the response of a single-degree-of-freedom oscillator to any specific ground motion, because they result from the multiplication of an anchoring parameter (the design ground acceleration, yielded by a probabilistic seismic hazard analysis at national scale and subsequent zonation) by site-dependent spectral shape functions obtained by fitting selected real spectra from

*Including, possibly, artificial roadfills, which should be checked in case they undergo excessive settlements during earthquakes.

**Table 10.1.** Topographic amplification factors from 3D and 2D numerical analyses compared with Eurocode 8 topographic amplification factors, and classification of the site according to EN 1998-5, *Annex A*

| Site | Eurocode 8 site classification | Analysis | | |
|---|---|---|---|---|
| | | Eurocode 8 | 3D | 2D |
| Civita  | Isolated cliff | 1.2 | 1.75 (+46%) | 1.30–140 (+8% to +17%) |
| Altino  | Isolated cliff | 1.2 | 1.30 (+8%) | 1.22 (+2%) |
| Mt Titano (San Marino)  | Ridge with crest width significantly less than the base width and of average slope angle > 30° | 1.4 | 1.58 (+13%) | 1.18–1.32 (–6% to –16%) |
| Castellaro  | Ridge with crest width significantly less than the base width and of average slope angle < 30° | 1.2 | 1.25 (+4%) | 1.09–1.28 (–9% to +7%) |

After Paolucci.[105]

different records in different earthquakes.[106] Hence, artificial accelerograms will generally contain energy over the whole range of vibration periods, not seen in any single record. As a consequence, artificial accelerograms may represent velocities, displacements, and high-energy content that are unrealistic, and major problems arising from their use have already been pointed out for structural analysis applications.[107]

In analyses of permanent ground displacement, such as the equivalent rigid-block modelling of seismically induced slope movements, it is a critical requirement that the input accelerograms give rise to physically realistic velocities and displacements, because these are directly reflected in the calculated displacements (see Example 10.1 in Section 10.4.1.3). Hence, artificial accelerograms are not recommended for use in such analyses, especially in the near field of earthquake sources, where strong, low-frequency velocity and displacement pulses are likely to dominate the ground motions.

*Clauses 3.2.3.1.2* and *3.2.3.1.3* of EN 1998-1 set the basic physical requirements for selecting artificial and recorded (or simulated) accelerograms, and recognize the special requirements attached to inputs for dynamic slope stability verifications and soil amplification analyses. The main requirement for duration is that it is consistent with the (moment, or surface wave) magnitude $M$ of the earthquakes that control seismic hazard in the site region. At distances of between about 10 and 30 km, indicative values of strong motion durations on rock, according to the definition of Trifunac and Brady,[108] are 4–6 s for $M \cong 5$, 7.5–9.5 s for $M \cong 6$ and 12–15 s for $M \cong 7$.

*Clause 3.2.3.1.2 4(c)* of EN 1998-1, calling into play the fundamental period of structural response to set requirements on the spectral input levels, may not be applicable in dynamic slope stability analyses because the response of the structure may not be relevant.

## 10.3. Ground properties

### 10.3.1. Strength parameters

In general, the shear strength to be used in design should correspond to the least favourable drainage condition under the prescribed load. *Clause 3.1*

#### *10.3.1.1. Cohesive soils*

**Static shear strength**

Earthquake loading is of short duration, and drainage cannot occur in cohesive soils during its occurrence. The most dangerous condition is immediately after the loading, because the shear strength will generally increase afterwards with the progress of consolidation. If the development of excess* pore pressure $\Delta u$ in the soil can in first approximation be neglected, the governing shear strength will be the same as in the static case, i.e. *Clause 3.1(1)*

$$\tau_f = c_u \tag{D10.2}$$

where $c_u$ is the undrained shear strength determined from (undrained) static tests prior to the cyclic loading.†

**Cyclic degradation effects**

When a normally consolidated specimen of cohesive soil is subjected to undrained cyclic loading, positive $\Delta u$ is progressively generated, and a reduction in effective stress occurs that will decrease the shear strength and may even lead to cyclic failure beyond a certain shear stress level, or a certain number of loading cycles. In the case of saturated sand, a failure of this type is called liquefaction (discussed in Section 10.4.1.5).

*That is, in excess of the hydrostatic pressure $u_0$.

†An analysis using $c_u$ only is also called the '$\phi = 0$' method.

The shear strength reduction induced by cyclic loading in a normally consolidated (NC) clay can be estimated by the expression[109,110]

$$\frac{(c_u)_{cyc}}{(c_u)_{NC}} = \left(\frac{1}{1-\Delta u/\sigma'_c}\right)^{l-1} \tag{D10.3}$$

where $(c_u)_{cyc}$ is the shear strength induced by undrained cyclic loading, $(c_u)_{NC}$ is the undrained (static) strength of the soil prior to cyclic loading, $\sigma'_c$ is the effective (static) confining stress, $l$ is the experimental constant that can be estimated via the correlation $l = 0.939 - 0.002I_p$, and $I_p$ is the plasticity index of the soil.

Equation (D10.3) allows quantification of the undrained shear strength reduction, provided one can estimate the cyclically induced $\Delta u$, which depends on the number of loading cycles. For a typical normally consolidated clay with $I_p$ in the range 20–30, $l$ is close to 0.9, and the strength reduction can usually be neglected because it will not exceed 10% even for a pore pressure ratio $\Delta u/\sigma'_c$ as large as 0.6. Only for very plastic clays, similar to the Mexico City clay, with $I_p$ in excess of 200, would the same pore pressure ratio result in a 35% strength reduction after cyclic loading, according to equation (D10.3).

The cyclically induced decrease in shear strength is a temporary phenomenon: as drainage occurs followed by dissipation of excess pressure, the effective stress will increase again, and the shear strength with it, until it eventually regains its 'static' value.

**Rate of loading effects**

In general, the rate of loading causes an increase in the strength of a cohesive soil. Considering loading tests with times to failure of the specimen ranging from 100 s (slow test) to about 0.1 s, an increase of the order of 15% compared with static tests may be considered,[111] although the estimate is affected by the large scatter of experimental data. Such an increase is normally neglected in favour of safety.

### *10.3.1.2. Cohesionless soils*

*Clause 3.1(2)* In an uncemented, water-saturated sandy soil, the shear strength in undrained conditions can be expressed by the Coulomb failure criterion in terms of effective stress:

$$\tau_f = (\sigma_f - u)\tan\varphi' \tag{D10.4}$$

where $\sigma_f$ is the total stress normal to the considered shearing plane. The use of this relationship is suggested in *clause 3.1(2)* of EN 1998-5. Its use, however, requires that the excess pore pressure, $\Delta u$, generated by earthquake-like cyclic loading is evaluated, in order to obtain the effective normal stress $\sigma'_f = \sigma_f - u$. This evaluation may pose difficulties.

*Clause 3.1(1)* The alternative approach alluded to in *clause 3.1(1)* of EN 1998-5 is to make use of an experimental relationship between the undrained resistance of the soil subjected to a well-defined cyclic loading process, denoted as $\tau_{cy,u}$, and a parameter representative of the state of packing of the soil material, such as its relative density $D_r$.

Representations of the cyclic shear strength versus $D_r$ typically use results from cyclic triaxial laboratory tests, in terms of the so-called cyclic stress ratio, i.e. a normalized value of the constant amplitude shear stress, $(\sigma_{dl}/2\sigma'_0)_{20}$,* required to generate a double-amplitude axial strain of 5% in 20 load cycles. The latter strain level is assumed to represent failure of the sand caused by (earthquake-induced) liquefaction. Thus, for axial cyclic loading of constant amplitude, one may assume $\tau_{cy,u} = \sigma_{dl}/2$. The shear stress is then normalized with respect to the initial effective confining stress, $\sigma'_0$, to obtain the cyclic stress ratio.

*Reference is made here to the stress conditions imposed in a triaxial test, where the maximum shear stress equals one-half of the deviator stress $\sigma_d = \sigma_1 - \sigma_3$, i.e. the difference between the axial (vertical) and the horizontal principal stresses. The cyclic stress component acts in the vertical direction.

For relative densities up to about 70%, i.e. in the range of interest for most practical applications, the data indicate that there is a linear relationship between liquefaction resistance and relative densities that can be written as

$$\left(\frac{\sigma_{dl}}{2\sigma'_o}\right) = \text{const} \times D_r(\%) \tag{D10.5}$$

To make this expression amenable to design applications, one must find ways of converting the cyclic resistance obtained for a loading history of 20 constant amplitude cycles into the resistance applicable under irregular seismic loading. It can be shown that the introduction of the maximum shear stress, $\tau_{max,\,l}$, causing some specified level of shear strain in multi-directional irregular loading, leads to the approximate relation[111]

$$\frac{\tau_{max\,l}}{\sigma'_v}\left(\frac{\sigma_{dl}}{2\sigma'_o}\right)_{20}$$

and, hence, always in first approximation, to

$$\frac{\tau_{max\,l}}{\sigma'_v} = \text{const} \times D_r(\%) \qquad \text{for } D_r < 70\% \tag{D10.6a}$$

where $\sigma'_v$ denotes the effective vertical stress. Based on the compilation of a set of data on typical clean sands, the relationship

$$\frac{\tau_{max\,l}}{\sigma'_v} = 0.0042 \times D_r(\%) \tag{D10.6b}$$

has been proposed.[111]

### 10.3.2. Partial factors for material properties

The recommended values of the partial factors are those given in EN 1997-1 for DA-1 C-2. They can be taken from Annex A of the same code, Table A.4 (set M2). Reference is made to the previous summary illustration of the different Design Approaches contemplated in EN 1997-1.

*Clause 3.1(3)*

### 10.3.3. Stiffness and damping parameters

#### *10.3.3.1. Shear stiffness*

The main role played by the shear modulus $G$ or, equivalently, by the velocity of propagation of the seismic shear waves, $v_s$, in the ground, is in the classification of the construction site according to the ground types established in *clause 3.1.2* of EN 1998-1. This aspect will be dealt with later in more detail in connection with *clause 4.2.2* of EN 1998-5. Additional applications that require knowledge of the shear stiffness of the soil profile include the calculation of:

*Clauses 3.2(1), 3.2(2)*

- The dynamic soil–structure interaction parameters (*Section 6* and *Annex D* of EN 1998-5)
- The seismic site response, which may, for example, be necessary to define the seismic action for ground type $S_1$ (see note after *clause 3.1.2(4)* of EN 1998-1), or to obtain a refined estimate of the seismic coefficient $k_h$ to be used in pseudo-static stability verifications of earth-retaining structures higher than 10 m (*clause 2* of *Annex E* of EN 1998-5).

The linear elasticity relationship of equation (D3.1) gives the value of $G$ at very small shear strain levels, typically $10^{-6}$ or less. However, in the applications just listed it is essential that the $G$ values are compatible with the shear strain levels induced by the earthquake in the ground, typically ranging between $< 10^{-5}$ and $10^{-2}$ if soil failure does not occur. Some guidance on the strain dependence of $G$ is directly provided in EN 1998-5 (*Table 4.1*).

Geophysical tests for performing direct *in situ* measurements of $v_s$ are usually carried out only for projects where an accurate determination of the design elastic response spectrum is necessary, or at sites where the ground type identification by other methods would be very difficult. Ways of estimating the $v_s$ value from correlations with more common geotechnical parameters will be illustrated in connection with *clause 4.2.2* of EN 1998-5.

#### *10.3.3.2. Damping*

*Clauses 3.2(3), 3.2(4)*

In addition to dynamic soil–structure interaction, internal soil damping (also a strain-dependent quantity) comes into play in the very same applications listed in relation to the shear stiffness.

## 10.4. Requirements for siting and for foundation soils

### 10.4.1. Siting

#### *10.4.1.1. General*

*Clause 4.1.1*

Depending mainly on the social and economic importance of the project, and on the severity of the design seismic action, the '*assessment of the site of construction ... to determine the nature of the supporting ground*' prescribed in *clause 4.1.1(1)* of EN 1998-5 may be naturally envisaged as a multi-step process, in which a preliminary overall evaluation of the site hazards (if any) is made first, typically by an experienced geologist, even before carrying out soil borings or other *in situ* investigations. A final assessment, including the ground classification, is then performed after the *in situ* data have been gathered and analysed. In some European countries, e.g. in Italy, current earthquake regulations require that a geological report is prepared to provide guidance on the 'stability' of the site in general, and on the foundation types to be adopted in design.

#### *10.4.1.2. Seismically active faults*

*Clause 4.1.2(3)*

As implied in *clause 4.1.2(3)* of EN 1998-5, the assessment of the surface fault rupture hazard at a site usually requires specialist advice. One should be aware, though, that criteria used by geologists for evaluating the 'seismic activity' of a fault in relation to how recent tectonic movements associated to it are not uniform and depend on the risk protection level envisaged for the structure (or infrastructure). Different authorities competent on this issue may also apply somewhat different criteria.

In Europe, surface breakage caused by co-seismic fault slip is a relatively rare event, observed in recent times in some zones of the Mediterranean region, mostly in Italy and Greece. Seismological evidence suggests that in regions where seismogenic activity is confined within the upper 20 km or so of the Earth's crust, co-seismic surface rupture tends to occur only for earthquakes with – moment ($M_w$) or surface wave ($M_S$) – magnitudes larger than about 6.5. However, this should not to be taken as an absolute criterion.

Useful information on seismically active faults in Europe may be found in the European Catalogue of Seismogenic Sources,[112] under the heading of 'Sources derived from geophysical/geological data'. However, the scale of the mapping of the involved faults is in general not compatible with the scope of the clause under discussion.

*Clause 4.1.2(1)*

Discussions on *clause 4.1.2(1)* of EN 1998-5 by the CEN/TC250/SC8 subcommittee did not result in making the indication '*in the immediate vicinity of tectonic faults*' quantitatively more stringent in terms of setbacks of new construction from a surface fault trace. A useful term of reference in this respect is Californian legislation, the Alquist-Priolo Earthquake Fault Zoning Act of 1972,[113] which prescribes that

> before a project can be permitted, cities and counties must require a geologic investigation to demonstrate that proposed buildings will not be constructed across active faults. An evaluation and written report of a specific site must be prepared by a licensed geologist. If an active fault is found, a structure for human occupancy cannot be placed over the trace of the fault and must be set back from the fault (generally 50 feet).

'Late Quaternary' in *clause 4.1.2(2)* of EN 1998-5 may be interpreted to encompass either the Holocene (last 10 000 years), or a longer time span beginning after the end of the latest glaciation (when applicable). *Clause 4.1.2(2)*

### *10.4.1.3. Seismic slope stability*

'*Unacceptably large displacements*', mentioned in *clause 4.1.3.1(2)* of EN 1998-5, may result from the attainment of either a ULS or a DLS. In either case, the seismically induced displacements in the slope may have to be evaluated. *Clause 4.1.3.1(2)*

For the quantification of the seismic action, the topographic amplification factor mentioned in *clause 4.1.3.2(2)* and in *Annex A*, have already been discussed. *Clause 4.1.3.2(2)*

Irrespective of whether finite-element, rigid-block or pseudo-static methods are used for stability analysis, the design actions, resistances, and strength parameters should presumably be consistent with the national choice of the geotechnical Design Approach of EN 1997-1. It has been mentioned in the introduction to this chapter that DA-1 C-2 or DA-3 (in the absence of structural loads) is compatible with EN 1998-5. EN 1997-1 (clause A.3.3.6) also requires that a partial factor on ground resistance, the value of which is left to national choice, is applied for slopes. The recommended values for such a resistance partial factor are unity for DA-1 and DA-3, and 1.1 for DA-2. *Clause 4.1.3.3*

If a refined dynamic response analysis of a slope is performed by, for example, finite elements, a correct description of strain-softening effects in the ground materials may be critical because they control the formation of shear bands, and their development into a localized slip surface. Additional recommendations on general aspects of slope stability analysis may be found in clause 11.5 of EN 1997-1.

**Pseudo-static method of analysis**

The application of the pseudo-static method of analysis entails the following steps: *Clauses 4.1.3.3(3), 4.1.3.3(4), 4.1.3.3(6), 4.1.3.4(4)*

(1) After selecting a slip surface (most frequently a circular arc), the prescribed horizontal and vertical seismic inertia forces are applied statically, in addition to the other permanent loads, to the centre of gravity of the ground mass enclosed between the slip surface and the ground surface.
(2) Through rigid body equilibrium considerations, a safety factor is computed as the ratio of stabilizing to destabilizing forces acting on the ground mass.
(3) The same operation is repeated many times, changing the slip surface and re-computing the safety factor, until a minimum value is found that is assumed as the effective safety factor.

Consistent with equation (D2.3), the check on the limit state condition is satisfied by verifying that the safety factor thus obtained is not smaller than unity.

It is essential not to overlook the limitations stated in *clause 4.1.3.3* of EN 1998-5 to the application of the pseudo-static method, i.e. *Clauses 4.1.3.3(3), 4.1.3.3(8)*

- the geometry of the topographic profile and the ground profile must be reasonably regular
- the ground materials of the slope, if water saturated, should not be prone to developing a significant pore pressure increase, which may lead to loss of shear strength and to stiffness degradation under cyclic loading.

The latter limitation will be discussed in more detail in Section 10.4.1.4.

The key parameter in the pseudo-static method is the **seismic coefficient** ($k_h$), i.e. the fraction of the design acceleration ratio, $S\alpha$, used to determine the design inertia forces. This fraction, set equal to 0.5 in *equation (4.1)* of EN 1998-5, has traditionally been selected on an empirical basis, drawing on the observed performance of slopes of embankments and earth dams and related back-calculations. The choice of the actual value of $k_h$ can be better interpreted using methods for evaluating permanent slope displacements (e.g. see Newmark[114]). By computing the response of simplified slope models (rigid block) excited by *Clause 4.1.3.3(5)*

acceleration histories recorded in many different earthquakes, the most likely permanent slope displacements caused by different earthquake magnitudes have been estimated as a function of the ratio $a_c/a_{max}$ of the 'critical' slope acceleration to the maximum value of input acceleration. The value of $a_c/g = k_c$ is the critical seismic coefficient of the slope, i.e. the coefficient yielding pseudo-static inertia forces that reduce the factor of safety to 1.0.

Parametric studies have been performed for embankments with heights ranging from about 15 to 80 m, considered also applicable to earth slopes with comparable depths to bedrock, and generally conservative for smaller depths. Results show that if a pseudo-static slope analysis with $k_h$ equal to a prescribed fraction of the peak acceleration yields a factor of safety greater than 1.0, then the displacements are likely to be limited. Within this perspective, the value $k_h = 0.5$ of EN 1998-5 appears to ensure that permanent slope displacements corresponding to a factor of safety greater than 1.0 would not exceed some tens of centimetres, even for earthquakes with a magnitude as high as 8.25,[115] and would obviously be smaller at smaller magnitudes. Stated differently, since the magnitudes of most damaging earthquakes in the highest seismicity regions of Europe are between 5.5 and 7.0, pseudo-static stability verifications with $k_h = 0.5$ resulting in factors of safety greater than 1.0 would ensure that permanent slope movements are negligible.

Therefore, as the value of $S\alpha$ increases, the designer may find it no longer convenient to use the pseudo-static method. The slope might tolerate larger displacements without attaining a failure LS, which would in any case require additional analyses to check the magnitude of the displacements.

The amount of conservatism that may be introduced by pseudo-static verifications compared to a fully dynamic method of analysis will be illustrated in a later section, in connection with the design of retaining structures.

**Dynamic slope analysis by the rigid-block model**

*Clause 4.1.3.3(7)* The simplified dynamic method for slope displacement analysis referred to was originally formulated by Newmark,[114] and further developed and refined in several successive studies (e.g. see Franklin and Chang[116]).

The application of the rigid-block model strictly requires that the shear strength along the slip surface be of purely frictional type, and suffer no degradation with time as a result of pore pressure increase induced by cyclic loading. The calculation of seismically induced permanent slope displacements using the rigid-block model is carried out through the following steps:

- A static stability analysis of the slope is performed first, which will identify the least safe slip surface, with the ground mass involved, and the effective static safety factor $F_s$ (> 1).
- The soil mass enclosed between the sliding surface and the ground surface is assimilated to a rigid block free to slide with friction on a rough plane (support) inclined at an angle $\theta$ with respect to the horizontal; for a non-planar slip surface, $\theta$ may be taken as the inclination of the resultant of the tangential forces acting on the slip surface, derived from the static analysis.[116]
- The critical (horizontal) seismic coefficient for the rigid block is calculated as

$$k_c = (F_s - 1)\frac{\tan\theta}{1 + \tan\phi\tan\theta} \qquad \text{(D10.7)}$$

  where $\phi$ is the angle of shearing resistance on the sliding surface. The interpretation of the critical coefficient is straightforward: the block cannot slide as long as the acceleration $\ddot{x}_0(t)$ of the support is less than $k_c g$, and starts sliding when such threshold is exceeded.
- A horizontal acceleration history $\ddot{x}_0(t)$ satisfying the requirements of *clause 4.1.3.3(7)* is selected as support excitation; the vertical component of the excitation can often be neglected because many parametric studies have shown that its influence is modest (due also to lack of correlation among vertical and horizontal acceleration components). The

downslope block displacements **in the direction parallel to the inclined support** are calculated by integrating the following equation of motion:

$$\ddot{x}(t) = \left(\frac{\ddot{x}_0(t)}{g} - k_c\right) g \frac{\cos(\theta - \phi)}{\cos\phi} \qquad \text{for } \frac{|\ddot{x}_0|}{g} \geq k_c$$
$$\ddot{x} = 0 \qquad \text{for } \frac{|\ddot{x}_0|}{g} < k_c \qquad \text{(D10.8)}$$

This amounts to integrating twice with respect to time the accelerogram portions that exceed the thresholds $\pm k_c g$, so that the resulting displacements are progressively cumulated up to the end of the motion.

An example of analysis of a real slide is presented below to illustrate the application of the method.

## Example 10.1: calculation of seismically induced displacements in a real landslide

### *Landslide description*

The landslide considered (Fig. 10.1), classified as a debris–earth block slide according to Varnes,[117] was located in the Udine region in north-east Italy. The permanent displacement calculation was carried out to check the effectiveness of the improvement measures applied to stabilize the slide, originally exhibiting a static safety factor $F_s = 1$. A number of vertical drains were inserted to lower the water table to the level shown in Fig. 10.1, so as to prevent further sliding of the unstable near-surface portion of soil and rock debris.

### *Seismicity data and input ground motions*

The landslide site lies in a region of high seismic activity, with the most recent destructive shocks (the Friuli earthquakes) occurring in May and September 1976, with magnitudes 6.4 and 6.0, respectively. Peak accelerations up to $0.35g$ on rock were recorded at the Tolmezzo–Ambiesta accelerograph station (Fig. 10.2), some 15 km from the landslide site. In the most recent seismic zonation of Italy,[104] the 475 year design acceleration on rock in the landslide area is $a_{gR} = 0.25g$. The looser near-surface materials in the landslide could justify the adoption of a soil factor $S$ between 1.2 (ground type B) and 1.4 (ground type E); which would bring the design acceleration $a_g S$ within the range $0.30$–$0.35g$.

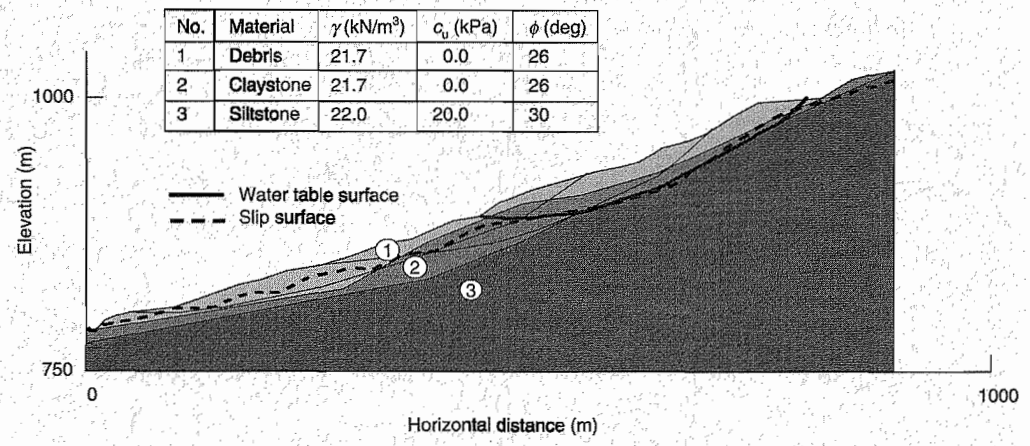



**Fig. 10.1.** Cross-section of a real landslide, located in the province of Udine, north-east Italy, including a description of ground materials and properties

The horizontal accelerations recorded at Tolmezzo–Ambiesta, illustrated in Fig. 10.2, with peak values of $0.32g$ (north-south) and $0.34g$ (east-west) were used as the excitation, without introducing an additional soil factor $S > 1$. The strong motion durations are between about 5 and 8 s, according to the most common definitions. Since the sign of recorded accelerations depends on the recording instrument, it is advisable to perform the sliding block analysis twice for a given accelerogram: once with accelerations as recorded, and a second time with inverted sign. Experience shows that the results tend to differ significantly. The sign inversion procedure was followed in the current case.

***Landslide parameters for rigid block analysis***

Earlier investigations had disclosed extreme heterogeneity and variability of the near-surface slide materials and of their strength parameters; the values illustrated in Fig. 10.1 represent very cautious estimates, affected by large uncertainties. For these reasons, and since the choice does not affect the significance of the example, no partial factor was introduced to reduce the shear strength in the static analysis of the slide and in the calculation of the critical seismic coefficient. Static stability analysis following a classical method[118] gave the following results:

- static safety factor $F_s = 1.44$
- critical seismic coefficient $k_c = 0.13$
- equivalent inclination of supporting plane $\theta = 18.2°$.

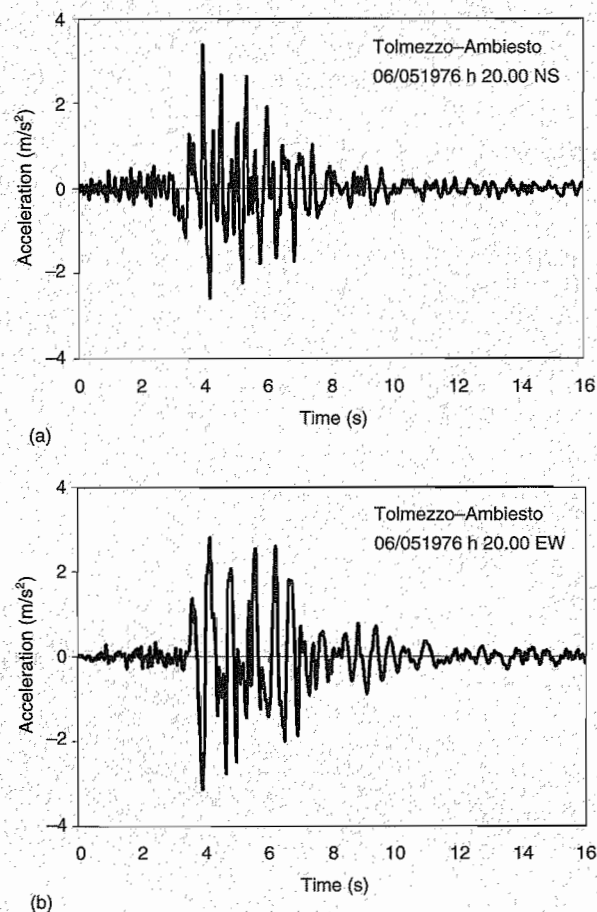



**Fig. 10.2.** Horizontal accelerations on rock, recorded at the station closest to the epicentre, during the 6 May 1976, $M_w$ 6.4 Friuli mainshock, selected as the excitation for dynamic rigid block analysis of the landslide in Fig. 10.1

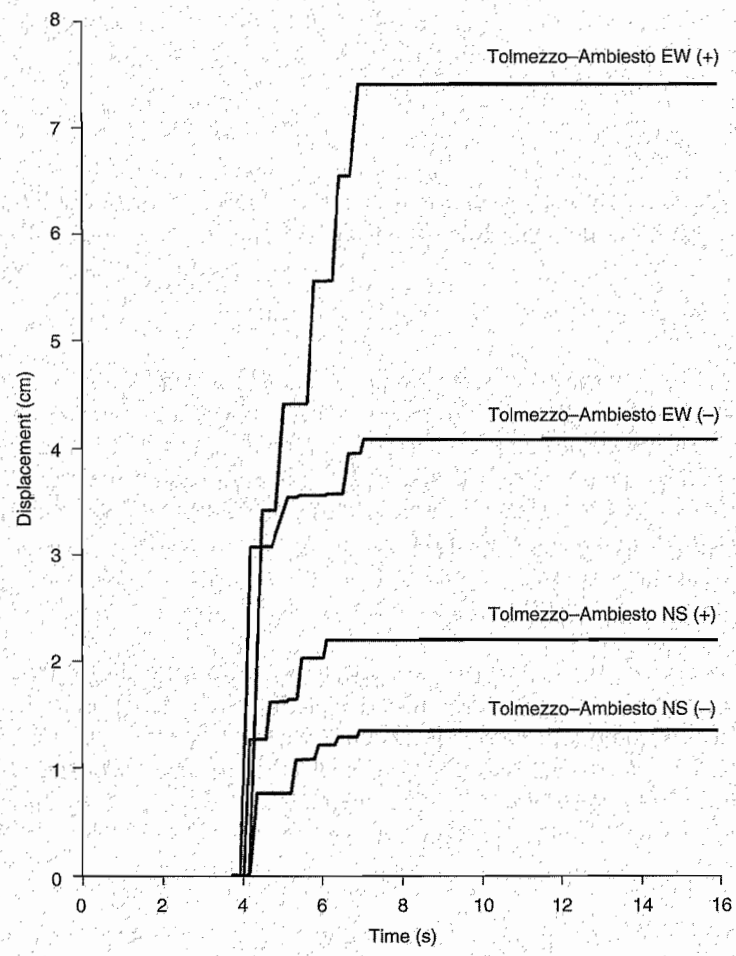



**Fig. 10.3.** Histories of permanent downslope displacements of the rigid-block model of the slide of Fig. 10.1, generated by the input accelerations of Fig. 10.2: (+), using accelerations as recorded; (–), accelerations with the signs inverted. The displacements are in the direction inclined at $\theta = 18.2°$ with respect to the horizontal

Since the value of $F_s$ is relatively high, one would expect limited permanent slope displacements. On the other hand, a negative factor is that the acceleration peaks of the excitation are much higher than $k_c$.

***Results***

Figure 10.3 depicts the permanent displacements in the downslope direction (i.e. in the direction inclined at $\theta = 18.2°$ with respect to the horizontal) calculated for the equivalent rigid block: their values, ranging between about 1 and 7.5 cm, are very small or negligible for all practical purposes. The strong sensitivity of the final displacement values to the details of the excitation waveform, including the anticipated influence of the sign inversion, lends support to the requirement in *clause 2.2(2)* of EN 1998-5 concerning the time–history representation of the seismic action.

*10.4.1.4. Pore water pressure increments due to cyclic loading*

The need to take pore pressure increments into account in saturated soils subject to contractive behaviour under cyclic loading is stressed in *clauses 4.1.3.3(9)* and *4.1.3.4(1)* of EN 1998-5. If appropriate cyclic (triaxial or shear) laboratory tests cannot be performed, empirical correlations may be used to estimate $\Delta u$.

*Clauses 4.1.3.3(9), 4.1.3.4(1)*

For constant-amplitude cyclic stress loading of a cohesionless soil, the pore pressure ratio $U = \Delta u/\sigma'_0$ can be conveniently computed as a function of the number of cycles $N_l$ to liquefaction through the following correlation[119]

$$U = \frac{2}{\pi}\arcsin\left(\frac{N}{N_l}\right)^{1/2b} \tag{D10.9}$$

where $b$ is an experimental constant with a typical value of 0.7. The number of cycles to liquefaction may be computed via the expression

$$N_l = A\left(\frac{\tau_c}{\sigma'_0}\right)^{\beta}(D_r)^{\delta} \tag{D10.10}$$

In equation (D10.10) $\tau_c$ is the cyclic stress amplitude, $\sigma'_0$ is the initial (vertical) effective stress prior to the earthquake, $D_r$ is the relative density (as a fraction $\leq 1$, not a percentage), and $A = 0.0503$, $\beta = -4.35455$ and $\delta = 4.80243$ are empirical constants. These constants were determined by fitting one of the best available sets of data, from shaking table experiments on liquefaction of large samples of saturated Monterey (California) No. 0 sand.[120] Equation (D10.10) holds for relative densities between 0.54 and 0.9, in the range of stress ratios $0.1 \leq \tau_c/\sigma'_0 \leq 0.3$, and for $N_l \leq 100$. The standard deviation of the regression is $\sigma_{\log Nl} = 0.09$ and $R^2 = 0.96$, indicating that the fitting of the data provided by equation (D10.12) is reasonably accurate.

Application of equations (D10.9) and (D10.10) to actual seismic design problems requires the irregular cyclic stress history generated by the earthquake to be estimated at the depth of interest in the soil, and reduced to an equivalent number of cycles of constant amplitude. A very simple approach[111] assumes that the uniform equivalent shear stress amplitude $\tau_e$ ($= \tau_c$) at shallow depth $z$ in a soil deposit (or in a slope) can be assessed by *equation (4.4)* of EN 1998-5, or by a slightly modified form of it, i.e.

$$\tau_e = 0.65\alpha S\sigma_{v0}(1 - 0.015z) \tag{D10.11}$$

where $\sigma_{v0}$ is the total overburden stress acting at depth $z$ (metres) before the earthquake. The following estimates of the number $N$ of significant equivalent cycles (at 0.65 of the peak value) as a function of earthquake magnitude $M$ may be used: for $M = 5.5$, 6.0, 6.5, 7.0 and 7.5, the corresponding values are $N = 3.5$, 4.0, 6.5, 10 and 14.5, respectively.[121] The coefficient of variation of these estimates is close to 0.5; this variability should be taken into account through sensitivity analyses.

*Clause 4.1.3.4(2)* A simple way of quantifying the reduced, large-strain shear strength referred to in *clause 4.1.3.4(2)* of EN 1998-5 is through the expression

$$(\tan\phi')^* = \left(1 - \frac{\Delta u}{\sigma'_v}\right)\tan\phi \tag{D10.12}$$

where $(\tan\phi')^*$ is the reduced effective strength. The ratio $\Delta u/\sigma'_v$ can be estimated using equations (D10.9)–(D10.11).

### *10.4.1.5. Potentially liquefiable soils*

*Clauses 4.1.4(1), 4.1.4(2)* The term 'liquefaction', introduced by Japanese specialists in 1953, has been employed to describe different phenomena affecting saturated, predominantly coarse-grained, soils. All such phenomena are characterized by the occurrence of permanent deformations, but they may be induced by different types of loading (monotonic, transient or cyclic), and the amount of deformation involved may substantially differ. A common feature is the loss of soil strength generated by pore water pressure build-up (in excess of the hydrostatic values) in undrained conditions, i.e. under constant-volume shear deformation. These conditions are imposed by the duration of the loading processes, which is too short for the excess pressures in the fluid to start dissipating.

From a design viewpoint, evaluating the liquefaction hazard and its potential consequences requires the following steps to be separately checked:

(1) **Susceptibility**, i.e. whether or not the soil under consideration is prone to liquefaction (e.g. if the fines content is high, liquefaction can be ruled out *a priori*).
(2) **Hazard**, i.e. whether or not liquefaction can occur under the design earthquake (e.g. the soil may be prone to liquefaction but the shear stresses generated by the design earthquake may not be severe enough to trigger the phenomenon).
(3) **Risk**, i.e. the possibility that, should liquefaction occur, the foundations suffer damage, and the extent of such damage.

**Susceptibility**

Both *clauses 4.1.4(7)* and *4.1.4(8)* of EN 1998-5 address step 1 above. The 15 m depth limit is supported mostly by the database of well-documented liquefaction phenomena, which contains no field observation at greater depths. Also, the normalized seismic shear stress $\tau_e/\sigma'_{v0}$ decreases appreciably at larger depths. *Clauses 4.1.4(7), 4.1.4(8)*

The exclusion cases listed under *clause 4.1.4(8)* refer to:

- the severity of ground motion, meaning that below some acceleration threshold even loose soils exhibit elastic response, and no permanent volume deformations occur
- the content of plastic fines (soils containing a relatively high proportion of non-plastic fines may also liquefy)
- high penetration resistance, as encountered in dense sands.

While still widely performed in some countries, liquefaction susceptibility verifications relying exclusively on grain size distribution criteria have been proven by field evidence to be unsafe (except in cases with a high content of plastic fines); a better policy, as shown below, is to correct the *in situ* soil resistance appropriately by an amount depending on the fines content.

**Hazard**

Once the susceptibility of the soil and site conditions has been checked in the previous step, verification of the liquefaction hazard is undertaken by comparing, on one hand, the specific action effect ($L$), or **seismic demand**, with the soil resistance against liquefaction ($R$), or **capacity**. Hence, the ratio

$$F_L = \frac{R}{L} \tag{D10.13}$$

in which both $R$ and $L$ are expressed in the form of cyclic shear stresses, can be conveniently viewed as the safety factor against liquefaction. The interpretation of equation (D10.13) is very simple: at depths where $R < L$, the hazard of liquefaction is likely, and unlikely where $R > L$.

$R$ is expressed as the cyclic shear stress amplitude, $\tau_{cy}$ (previously discussed in connection with *clause 3.1*) required to cause liquefaction of the *in situ* soil in a number of cycles compatible with the magnitude of the reference earthquake. Dividing $\tau_{cy}$ by the initial effective vertical stress, $\sigma'_{v0}$, yields the **cyclic resistance ratio** $(\tau_{cy}/\sigma'_{v0})_l$.

On the other hand, the action effect generated by the design earthquake in the soil is represented by an equivalent constant-amplitude cyclic shear stress, $\tau_e$, which may be evaluated in a simplified way by an expression such as equation (D10.11). If $\tau_e$ is also normalized by dividing it by $\sigma'_{v0}$, one obtains the **cyclic stress ratio** $(\tau_e/\sigma'_{v0})$.

Introducing these definitions, equation (D10.13) takes the form

$$F_L = \frac{(\tau_{cy}/\sigma'_{v0})_l}{\tau_e/\sigma'_{v0}} \tag{D10.14}$$

Ideally, the cyclic resistance ratio ought to be evaluated by obtaining from the construction

site undisturbed specimens of the soil susceptible to liquefaction, and performing cyclic laboratory tests on them that simulate the design earthquake loading. This would typically require recourse to highly specialized *in situ* sampling techniques, such as deep freezing of the soil, which are expensive and can be deployed in only a few critical projects. At a lower level of reliability, the tests may be executed on specimens reconstituted in the laboratory at the desired relative density. In a sophisticated approach of this type, the stress ratio $\tau_e/\sigma'_{v0}$ would be computed more accurately than by using equation (D10.11), e.g. by performing one-dimensional wave propagation analyses for the *in situ* soil profile, excited by acceleration histories representative of the design earthquake.

In the few cases where this approach can be followed, verification of safety against liquefaction simply requires checking that $FL > 1$, after applying the appropriate partial factor to $\tau_{cy}$.

*Clauses 4.1.4(3), 4.1.4(9)*

To bypass the significant difficulties posed by the sampling and cyclic testing of undisturbed specimens of saturated sandy soils, empirical methods for estimating $(\tau_{cy}/\sigma'_{v0})_1$ on the basis of standard *in situ* tests were developed (e.g. see Seed[121]) that have become standard practice for assessing the liquefaction hazard. Adoption of such practice is prescribed as a minimum requirement for the verification of the liquefaction hazard in *clause 4.1.4(9)* of EN 1998-5. It relies on comparing the *in situ* measurements of the standard penetration test (SPT) blowcount with the limiting values measured at sites that have suffered liquefaction in past earthquakes. These past observations are cast in the form of a curve expressing $(\tau_{cy}/\sigma'_{v0})_1$ as a function of the normalized SPT resistance, $N_1(60)$.

*Clauses 4.1.4(4), 4.1.4(5)*

*Clauses 4.1.4(4)* and *4.1.4(5)* of EN 1998-5 state that normalization of the measured $N_{SPT}$ blowcount should be performed by the expression

$$N_1(60) = N_{SPT}\sqrt{\frac{100}{\sigma'_{v0}}}\frac{ER(\%)}{60} \tag{D10.15}$$

where $\sigma'_{v0}$ is in kilopascals, and ER is the ratio of the actual impact energy to the theoretical free-fall energy in the SPT test. The value of this ratio has been 60% in traditional US practice, but is somewhat higher in the current practice of some European countries: for instance, in Italy, the energy ratio ER is taken as 70–75%, yielding a correction factor ER/60 equal to 1.20– 1.25.

*Clause 4.1.4(10)*

For a given soil depth ($z$) of analysis, the cyclic stress ratio in the denominator of equation (D10.16) is obtained by associating with the abscissa $N_1(60)$, as given by equation (D10.15), the ordinate $\tau_e$ given by *equation (4.4)* of EN 1998-5. The latter is a slightly conservative version of the foregoing equation (D10.11), with the depth reduction term $(1 - 0.015z)$ omitted. This normalized cyclic stress ratio, i.e.

$$\frac{\tau_e}{\sigma'_{v0}} = \frac{\tau_e}{\sigma'_{v0}}[N_1(60)] \tag{D10.16}$$

can now be represented in the chart on the left of *Fig. B.1*, in *Annex B* of EN 1998-5. Plotted on the same graph is also a curve representing the cyclic resistance ratio for clean sands (empirically derived from observations at liquefaction sites in past earthquakes) as a function of $N_1(60)$, i.e.

$$\left(\frac{\tau_{cy}}{\sigma'_{v0}}\right)_{1,\,M_s=7.5} = f[N_1(60)] \tag{D10.17}$$

where the index '$M_S = 7.5$' warns that the curve as it stands holds only for earthquakes of such magnitude or close to it. Also, the resistance curve on the left side of *Fig. B.1* is applicable only to sands with fines content less than 5%.

To facilitate computations, the following polynomial fit to the empirical curve $f(.)$ in equation (D10.17) is convenient, for $N_1(60) \leq 30$:[122]

$$f(.) = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \tag{D10.18}$$

where $x = N_1(60)$, $a = 0.048$, $b = -0.1248$, $c = -0.004721$, $d = 0.009578$, $e = 0.0006136$, $f = -0.0003285$, $g = -1.673 \times 10^{-5}$ and $h = 3.714 \times 10^{-6}$.

Adaptation of equation (D10.17) to earthquake (surface wave) magnitudes $M_S$ other than 7.5 can be achieved by the following empirical correction:

$$\left(\frac{\tau_{cy}}{\sigma'_{v0}}\right)_{l,\, M_s} = CM\left(\frac{\tau_{cy}}{\sigma'_{v0}}\right)_{l,\, M_s = 7.5} \qquad \text{(D10.19)}$$

where CM values as a function of magnitude are given in *Table B.1* in *Annex B* of EN 1998-5. Such correction factors, greatly reducing the liquefaction likelihood at the lower magnitudes, are affected by significant uncertainty; it is advisable in applications to try other corrections as well.[122] The reduced likelihood of liquefaction with decreasing magnitude stems from the reduction in the number of cycles of ground motion caused by the shorter duration.

Albeit less susceptible, silty sands, with a fines content (FC) above 5%, may also suffer liquefaction, as illustrated by the different curves in the graph on the right side of *Fig. B.1* in *Annex B* of EN 1998-5. Depending on the value of FC of the considered sand, one may use the appropriate curve in the latter figure, or interpolate for intermediate FC values. A procedure more readily amenable to automatic computation is to modify the $N_1(60)$ values in such a way as to use **only** the basic curve for clean sands, based on the fact that the different curves in *Fig. B.1* in *Annex B* of EN 1998-5 have essentially the same shape and can be obtained from each other through rigid translation. Thus, the $N_1(60)$ blowcount measured in silty sands can be transformed into an 'equivalent resistance' $N_1(60)_{cs}$, where 'cs' denotes clean sand, through the expression[122]

$$N_1(60)_{cs} = a + b\, N_1(60) \qquad \text{(D10.20)}$$

The values of $a$ and $b$ are a function of FC, as follows:

$a = 0, b = 1.0$ for $FC \le 5\%$

$a = \exp[1.76 - (190/FC^2)], b = [0.99 + (FC^{1.5}/1000)]$ for $5\% \le FC \le 35\%$

$a = 5.0, b = 1.2$ for $FC \ge 35\%$

To sum up, checking the liquefaction hazard for a susceptible soil layer requires performing the following operations: *Clause 4.1.4(3)*

(1) Carry out SPT tests or, alternatively, static cone penetration tests (CPTs), as appropriate, in the foundation soils in the depth range where susceptibility to liquefaction has been assessed.

(2) Normalize the measured $N_{SPT}$ into $N_1(60)$ by means of equation (D10.13) and, if FC is greater than 5%, transform $N_1(60)$ into $N_1(60)_{cs}$ through equation (D10.20) (alternatively, refer to the appropriate cyclic resistance curve in *Fig. B.1* in *Annex B* of EN 1998-5). *Clause 4.1.4(4), 4.1.4(5), 4.1.4(10)*

(3) Associate with the $N_1(60)_{cs}$ value the corresponding cyclic stress ratio via *equation (4.4)* of EN 1998-5 and equation (D10.16) above.

(4) For the same $N_1(60)_{cs}$, compute the cyclic resistance ratio to liquefaction through equations (D10.17)–(D10.19), using the CM values of *Table B.1* in *Annex B* of EN 1998-5.

(5) Compute the safety factor $F_L$ in equation (D10.14), and check that $F_L \ge 1/\lambda$ to satisfy *clause 4.1.4(11)* of EN 1998-5. The recommended minimum value $1/\lambda = 1.25$ of $F_L$ is meant to take into account the uncertainties of the field-based procedure, in which no partial factor is applied to the *in situ* resistance $N_1(60)$. *Clause 4.1.4(11)*

Alternatives envisaged in *Annex B* of EN 1998-5 (*clauses B.3* and *B.4*) refer to the possibility of using cyclic resistance charts based on CPT and S wave propagation velocity measurements instead of SPT. While the CPT is in most cases preferable to the SPT to quantitatively describe *in situ* soil profiles, the corresponding empirical cyclic resistance curves proposed in the literature are not as well constrained by field observations as in the SPT case.

**Example 10.2: liquefaction hazard evaluation**

The example refers to a actual site near the coastline in north-east Sicily, Italy, where an uppermost layer of loose silty sand over 15 m thick, with FC estimated at 15%, was found on the sea bottom. To determine the cyclic liquefaction resistance, a reference magnitude $M_S = 6.1$ was selected, coinciding with that of the strongest event observed in the region

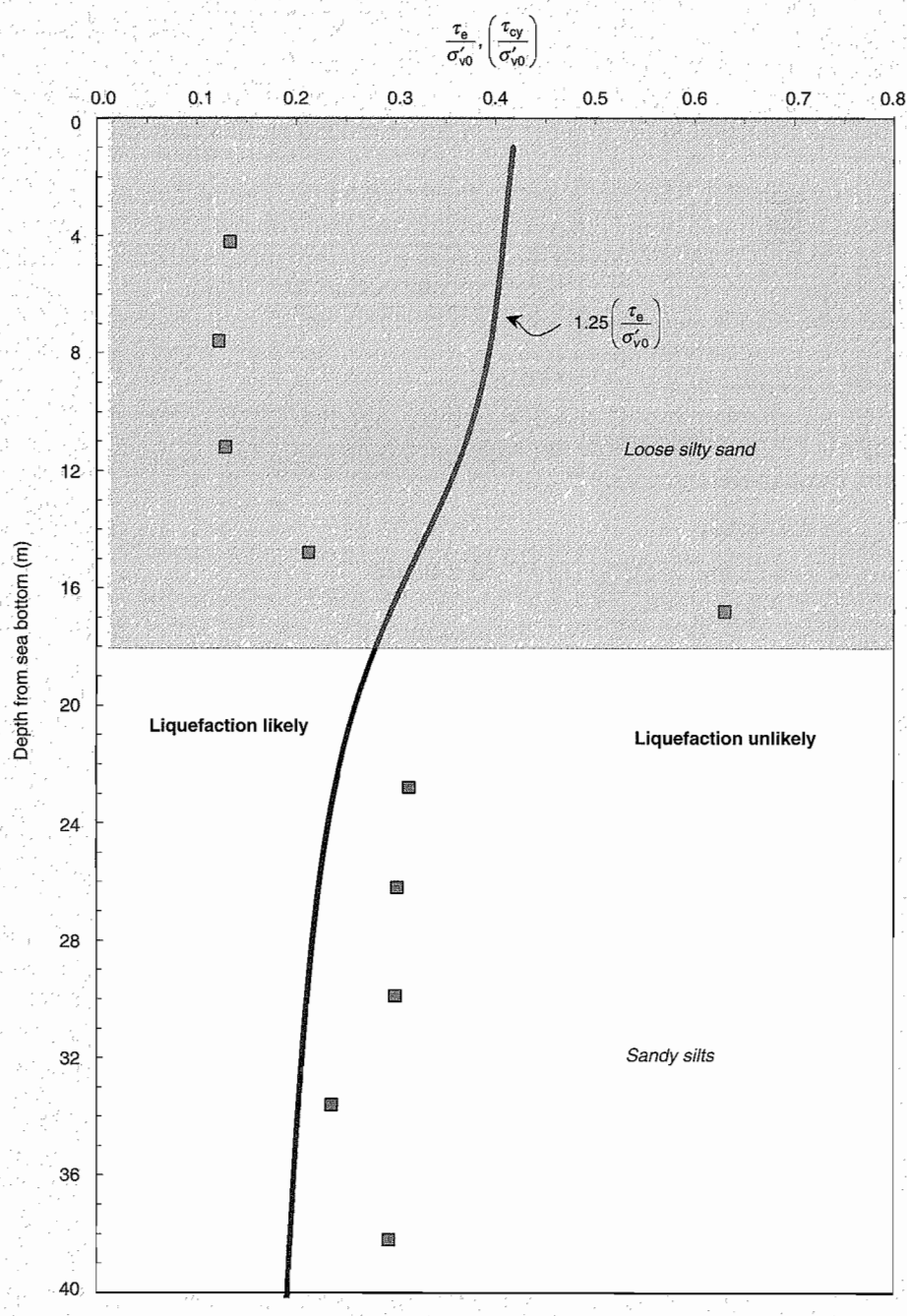



**Fig. 10.4.** Evaluation of liquefaction potential at a site with a near-surface layer of silty sand, having FC = 15%. Solid symbols denote values of cyclic resistance ratio $(\tau_{cy}/\sigma'_{v0})_l$ corresponding to *in situ* $N_{SPT}$ values and to a reference $M = 6.1$ earthquake magnitude, while the curve shows the applicable cyclic stress ratio as a function of depth, multiplied by the safety factor 1.25

(Gulf of Patti earthquake, 1978). A design acceleration $Sa_g = 0.25g$ is applicable to this site according to the zonation in Decree 3274.[104] The results of applying the verification procedure prescribed by EN 1998-5 on the soil profile of one geotechnical boring are illustrated in Fig. 10.4. This directly displays the two terms in the definition of $F_L$, equation (D10.14): the earthquake loading term, with the 25% increase corresponding to the factor of safety, is represented by the continuous curve, while the cyclic liquefaction resistance corresponding to the measured $N_{SPT}$ values is shown by the solid symbols. Liquefaction under the reference earthquake may in this case be expected in the silty sand layer, but is unlikely to occur in the underlying, denser, sandy silts as a consequence of both increased penetration resistance and increased FC.

**Measures for mitigating risks caused by soil liquefaction**

*Clauses 4.1.4(12), 4.1.4(13)*

The type of damage caused by earthquake-induced liquefaction to building structures and foundations has been documented for more than 200 years: for instance, spectacular field evidence associated with widespread liquefaction occurrences, including sand volcanoes and landsliding, is extensively (and graphically) described in Sarconi,[123] in an account on the devastating Calabria, southern Italy, earthquakes of 1783.

A major recent earthquake where liquefaction damage occurred on a large scale, especially to port and coastal installations, was that of Kobe, Japan, in 1995. Liquefaction was massive over two man-made islands in the Kobe port (Kobe and Rokko Islands), in areas reclaimed with soil that had not been compacted. This may have been in part an economic decision, supported by the assumption that the particular soil used in the reclamation, locally known as 'Masado' and originating from the decomposition of the granite rocks exposed on the slopes of the Rokko mountains adjacent to the city, exhibited grain size distribution features that should have made it resistant to liquefaction, although *in situ* $N_{SPT}$ values were as low as 5–10.[124] In the areas where the reclaimed soils were improved by sand drains or rod compaction piles, liquefaction was less significant but not completely absent. Also notable, and rather unusual, was the fact that gravels and cohesive soils were observed to be ejected with water from cracks in the liquefied ground.

Measures to prevent liquefaction in the foundation soil should primarily be aimed at maintaining the functions of the supported structure after an earthquake. In addition to the specific purpose of the measures adopted (listed below) and their technical implementation aspects, the dimensions of the intervention (area and depths to be treated) are important factors to be taken into account. The options available as to the specific purpose of the intervention are:

(1) To increase the soil density, in order to reduce the occurrence of excessive pore water pressure, by means of:
   - dynamic compaction methods, such as heavy tamping, or vibro-compaction, or
   - solidification, e.g. achieved by injecting a fluid mixture into the soil (jet grouting).
(2) To dissipate pore water pressure by increasing the soil permeability, e.g. through placement of drains (as in vibroflotation), or by replacement of *in situ* soils with coarse sand, gravel, etc.
(3) To modify the *in situ* stress conditions through an increase of the confining pressure, by lowering the water table level.

These measures can be used individually or in combination.

The effectiveness of dynamic compaction methods is assessed by comparing the measured post-treatment $N_{SPT}$ (or CPT) values with the initial ones; perusal of *Fig. B.1* of EN 1998-5 shows that for severe earthquake motions and low or moderate FC, safety from the liquefaction hazard requires achieving $N_1(60)$ values larger than about 25–30.

In heavy tamping, soil densification is achieved by letting a concrete or steel plate (up to tens of tonnes in weight) fall freely from heights that may reach 30–40 m. The aim is to

cause repeated liquefaction in the soil with successive impacts; after the dissipation of excess pore pressures generated by the impact has occurred, the soil particles will settle in denser, more stable configurations. Heavy tamping is a simple and rapid procedure, but its effectiveness depends heavily on the soil FC, and actually tends to vanish when the FC exceeds 15–20%.

Placement of drains, for example the 'gravel piles' normally associated with vibro-compaction (vibroflotation) techniques is an effective and fairly economic measure. Simplified methods and charts are available for the design of the spacing of the drains.[119]

Partial solidification can be obtained by injecting a cement–water mixture into the soil (jet grouting; chemical compounds, which are much more expensive, are rarely used). This measure aims at strongly increasing the shear stiffness of the soil, in order to ensure that the earthquake-induced shear strains will be small and essentially in the elastic range. This in turn prevents the tendency of a loose soil to contract under cyclic loading and, hence, to develop excess pore water pressure. Treatment by jet grouting, which is more expensive than dynamic compaction and placement of drains, can reach large depths, and is an important alternative when dynamic compaction cannot be performed either because of environmental constraints, such as nearby buildings affected by vibrations, or because of the FC of the soil.

Permanent lowering of the water table typically requires inserting into the soil an impermeable diaphragm wall enclosing the perimeter of the construction area and permanent dewatering below the foundation depth by pumping. It is clearly an expensive measure, justified only for certain projects, and is typically used in conjunction with other methods.

#### *10.4.1.6. Soils that may suffer excessive settlements under cyclic loads*

**Experimental indications for estimating settlement**

*Clauses 4.1.5(1), 4.1.5(2), 4.1.5(3)*

Vibration has long be recognized as an effective way of densifying cohesionless soils; experiments based on cyclic simple shear and shaking table tests with acceleration and strain amplitudes in the range of those expected in strong motion earthquakes have shown that shear strain amplitude, $D_r$, and the number of loading cycles are the primary factors governing compaction of dry and saturated drained cohesionless soils.

Early cyclic shear tests on small specimens of dry sand subject to sinusoidal excitation[125] indicated that, for shear strain amplitudes of about 0.3%, the permanent vertical strain accruing in 10 cycles of loading is about 0.5% for a medium-dense sand ($D_r = 60\%$), and about 1.0% for a loose-to-medium sand ($D_r = 45\%$). It may be recalled that 10 cycles of loading at the indicated constant amplitude would correspond to a rather severe earthquake, of magnitude about 7. This indication was confirmed by results of unidirectional horizontal shaking table tests on uniform dry sand, showing vertical settlements between 1 and 2% for 0.3$g$ peak base accelerations in the worst cases, and considerably less for more favourable situations; it was also found to be in agreement with settlements of 0.5–1.0% observed in unsaturated sand fills subjected to strong ground shaking in the San Fernando, California, earthquake of 1972.[126]

Shaking table laboratory investigation of the effects of two-dimensional and three-dimensional excitation were also carried out several years ago,[127] with independent random excitation histories having peak horizontal accelerations up to 1.0$g$ and vertical accelerations (with sinusoidal motion) up to about 0.3$g$ at a typical frequency of 6 Hz. With induced stresses and strains assumed to be comparable to those developed in the field at 1.5–3.0 m depth, the results showed that the settlement under combined motion approximately equals the sum of the settlements independently induced by one-dimensional motion, and that this holds true for a wide range of densities. For a medium-dense sand, the vertical settlement under combined excitation was found to be about 0.25% for an excitation lasting about 4 s, and 0.3% for a shaking duration of 10 s. These results may be considered a reliable basis for making judicious extrapolations to field conditions.

### 10.4.2. Ground investigations and studies

*Clause 4.2.1, 4.2.1(2)*

As stated in clause *4.2.1.(2)* of EN 1998-5, CPTs, where they can be performed, are especially suitable for providing a detailed and continuous quantitative description of a soil profile. CPTs are especially effective in detecting small-scale vertical (and lateral) heterogeneities, for instance loose sands that may be prone to liquefaction, interlayered in other soils. Correlations are available between the measured cone tip resistance $q_c$ and different parameters of geotechnical earthquake engineering interest such as relative density, and $S$ wave propagation velocity $v_S$ or the elastic small deformation shear modulus $G_0$. A useful correlation for the latter takes the form

$$\frac{G_0}{q_c} = c_1 \left( \frac{q_c}{(\sigma'_{v0} p_a)^{0.5}} \right)^{c_2} \tag{D10.21}$$

where $q_c$ is the measured cone tip penetration resistance (in the same units as $G_0$), $p_a$ is the reference pressure (= 1 bar = 100 kPa), and $c_1$ and $c_2$ are empirical constants.

For uncemented siliceous sands it was found[128] that the modulus to point resistance ratio decreases with increasing compressibility of the soil, and the values $c_1 = 290.57$ and $c_2 = -0.75$ have been proposed. For sand and gravel deposits of Pleistocene age, with FC less than 20%, gravel content between 13 and 95%, and $D_{50}$ between 1 and 20 mm, the values $c_1 = 144.04$ and $c_2 = -0.631$ have been determined,[129] with a coefficient of determination $R^2 = 0.81$.

### 10.4.3. Ground type identification for the determination of the design seismic action

*Clauses 4.2.2(1), 4.2.2(2), 4.2.2(4)*

In addition to fulfilling the needs specified in EN 1997-1 (Chapter 3) concerning design under non-seismic actions, geotechnical ground investigations within the context of seismic design are carried out at a construction site to:

- detect, within the depth range of interest, soil layers that may be prone to seismically induced liquefaction or excessive settlements due to compaction (already discussed)
- obtain basic data for the ground-type identification required in *clause 3.1.2* of EN 1998-1
- detect marked irregularities in the buried morphology at shallow depths, such as strong lateral variations in bedrock depth, which may significantly affect the dynamic site response and, hence, the design seismic action.

The ground type can be identified through the values of three different geotechnical parameters, as per *clause 3.1.2* of EN 1998-1, which ought to be measured or estimated over a depth interval of 30 m below the foundation level. The ranges of the parameter values that identify the different ground types are given in *Table 3.1* of EN 1998-1.

The leading parameter, at least conceptually, is the shear wave velocity $v_s$, because it is generally required for calculating the weighted-average velocity value $v_{s,30}$ through equation (D3.1). In more specific cases, the $v_s$ profile may also be necessary to estimate the natural periods of vibration of a soil deposit at the construction site, e.g. to evaluate local amplification or soil–structure interaction effects. It is stated in EN 1998-1 that '*The site should be classified according to the value of the average shear wave velocity,* $v_{s,30}$, *if this is available. Otherwise the value of* $N_{SPT}$ *should be used.*' The two auxiliary geotechnical parameters are the SPT blowcount and, limited to cohesive soils, the undrained shear strength $c_u$.

*Clauses 4.2.2(5), 4.2.2(6)*

In both EN 1998-1 and EN 1998-5 (*clause 4.2.2(5)*), *in situ* measurements of $v_s$ are recommended in few cases, while for many applications the $v_s$ values can be estimated through widely used geotechnical correlations, mainly from $N_{SPT}$ values, but also from $q_c$ values of CPTs (see equation (D10.21)), in many cases more readily performed than SPTs.

*In situ* measurements of the profile of the S wave velocity ($v_s$) are reliably performed by well known geophysical methods, which include:

- The cross-hole test, which is apt to yield the most accurate determinations of seismic propagation velocities (down to depths of 100 m or more), but requires the execution of

**Table 10.2.** Values of the age factor $f_A$ for equation (D10.22)

| Geological age | $f_A$ |
|---|---|
| Holocene (last 10 000 years) | 1.0 |
| Pleistocene (600 000 years) | 1.3 |

**Table 10.3.** Values of the soil-type factor $f_G$ for equation (D10.22)

| Soil type | $f_G$ |
|---|---|
| Clay | 1.00 |
| Sand | 1.10 |
| Gravel | 1.45 |

at least two boreholes, preferably three; in a more sophisticated version, tomographic techniques can be applied using multiple sources and receivers, allowing the seismic velocity distribution to be resolved in two, or three, dimensions rather than along a single one-dimensional vertical profile.

- The down-hole test, less accurate than the previous one, allowing typical measurement depths of a few tens of metres to be reached; it requires a single borehole, but is significantly affected by the background noise level at the surface.
- The seismic CPT test, in which a seismic probe is inserted in the tip of the static cone penetrometer, and velocity measurements are performed at discrete depths, using essentially the same configuration as in the down-hole test, i.e. a seismic source at the surface, and a seismic sensor at depth.
- Tests based on the dispersive propagation properties of surface waves: in these tests the velocities are determined by iteratively solving an inverse mathematical problem using the dispersion curve characteristic of the soil profile. The seismic signals may be artificially generated, as in the SASW (spectral analysis of surface waves) type of survey, or may be just the 'natural' surface waves present in the seismic background noise, as in the ReMi (refraction microtremor) method.[130]

The latter type of tests are somewhat less accurate with respect to the in-hole tests, but have the significant advantage of not requiring the execution of boreholes, and as such they may represent the only option available when operating in congested urban areas.

Statistical correlations have been developed, mainly in Japan, between $N_{SPT}$ values and the small deformation shear modulus $G_0$, or the shear wave velocity of propagation $v_s$. Among the most widely used is the correlation between $v_s$ and $N(60)$ proposed by Ohta and Goto,[131] which has the form

$$v_S = C(N(60))^{0.17} z^{0.20} f_A f_G \qquad \text{(D10.22)}$$

where $v_S$ is in metres per second, and $C$ is a constant (= 68.5), $z$ is the depth of the SPT measurement (metres), $f_A$ is the age factor of soil deposits and $f_G$ is the soil-type factor. The latter factors take the values shown in Tables 10.2 and 10.3.

The correlation equation (D10.22) has been tested in different regions and has been found to give generally better results for the younger soil deposits. The use of equation (D10.22) is illustrated in one of the following examples, showing how the identification of ground types can be performed according to the prescriptions given in *clause 3.1.2* of EN 1998-1.

## Example 10.3: ground-type identification at an actual construction site

### *Description of the problem*

Illustrated first is a site on very thick Quaternary sediments, located in a city in northern Italy. The area is one of low seismicity, but the project involved a large investment, and an accurate identification of the ground type for the selection of the design seismic action, according to the prescriptions given in *clause 3.1.2* of EN 1998-1, was considered important.

### *Ground data*

A single S wave velocity profile was available, obtained by down-hole measurements made in the same borehole where SPTs had been performed, as illustrated in Fig. 10.5. 'R' (for 'refusal'), starting at a depth of 15 m, denotes the presence of large gravel and pebbles in the predominantly sandy soils that characterize the site. The typical $v_s$ values are mostly between 200 and 300 m/s, except in a few ranges where they increase somewhat with increasing gravel content. The upper and lower bound estimates based on geological age, provided by equation (D10.22), are in reasonable agreement with the averaged measurements.

### *Identification procedure and results*

Quantitatively, the identification procedure is carried out in the form shown in Table 10.4. Adding the values in the last column gives a total transit time $t_{30} = 0.111$ s; hence, the

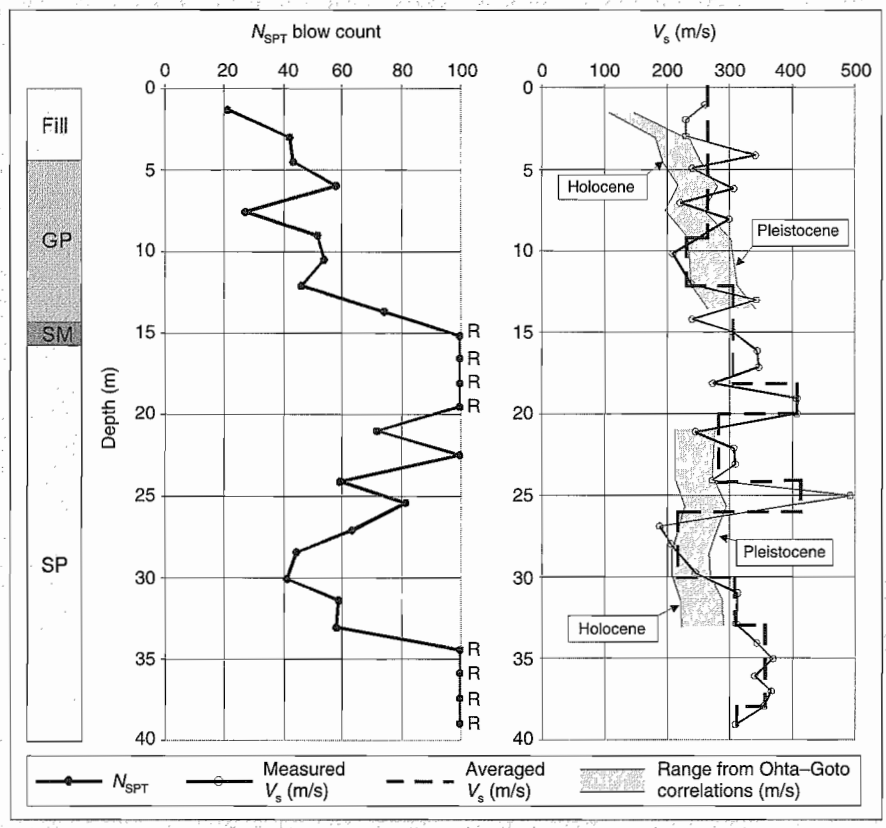



**Fig. 10.5.** Data used for ground-type identification at a deep alluvium site in a northern Italian city. From the left: soil profile (unified soil classification), SPT profile, $v_s$ profile from down-hole measurements and $v_s$ bounds estimated with equation (D10.22)

**Table 10.4.** Data on stratification for Example 10.3

| Depth of soil layers from ground surface (m) | Layer thickness, $h_i$ (m) | Average $v_s$ in the $i$th layer (m/s) | S wave transit time in the $i$th layer, $t_i = h_i/v_{s,i}$ (s) |
|---|---|---|---|
| 0.0–9.2 | 9.2 | 264 | 0.035 |
| 9.2–12.1 | 2.9 | 229 | 0.013 |
| 12.1–18.2 | 6.1 | 304 | 0.020 |
| 18.2–20.1 | 1.9 | 408 | 0.005 |
| 20.1–24.1 | 4.0 | 281 | 0.014 |
| 24.1–26.0 | 1.9 | 408 | 0.005 |
| 26.0–30.0 | 4.0 | 213 | 0.019 |

weighted average velocity value of equation (D3.1) is $v_{s,30} = 30/0.111 = 270$ m/s, and the ground is identified as type C, according to *Table 3.1* of EN 1998-1.

It may be noted that:

- For simplicity, the 30 m soil depth is calculated from the ground surface, rather than from the foundation level.
- The layer subdivision in Table 10.4 is based on significant changes in the average $v_s$ values, rather than on the soil profile description.
- Since two-thirds of the $N_{SPT}$ values in the 30 m range exceed 50, had the identification been exclusively based on penetration resistance, the ground would more likely be assigned to type B from *Table 3.1* of EN 1998-1. However, estimating the $v_s$ values with equation (D10.22) would give the more correct type C classification, as shown in Fig. 10.5.

## Example 10.4: a further case of ground-type identification at an actual site

### *Description of the problem*

A further example of ground-type identification is provided in Fig. 10.6 for another site, also located on a very thick sequence of Quaternary sediments.

### *Ground data*

With respect to the case of Example 10.3, more abundant and accurate data are available for ground-type identification, consisting of two different sets of SPT and cross-hole velocity tests at locations about 100 m apart, as shown in Fig. 10.6. The cross-hole measurements could not be reliably performed at depths of less than 10 m, due to lack of bonding between soil and borehole casing, not an uncommon occurrence at shallow depths. Hence, the assumption that the 30 m depth range starts at the depth of –10 m, since this is comparable to the depth of the foundations level.

### *Identification procedure and results*

For the data plotted on the left-hand-side in Fig. 10.6, direct inspection of the $N_{SPT}$ and $v_s$ profiles reveals that the ground belongs to type C, since most of the velocities are in the range between 180 and 360 m/s, and nearly all the blowcount values are in the range between 15 and 50. In this case, the two sets of parameters would give a consistent identification.

For the data for the second location, plotted on the right-hand side of Fig. 10.6, the use of the penetration resistance would be misleading because in a soil of this type, as we have already seen, a 'refusal' indication does not necessarily mean that the soil is very stiff. On the other hand, direct use of the measured $v_s$ values would correctly yield a type C identification as at the previous location, although the trends of the two velocity profiles are quite different.

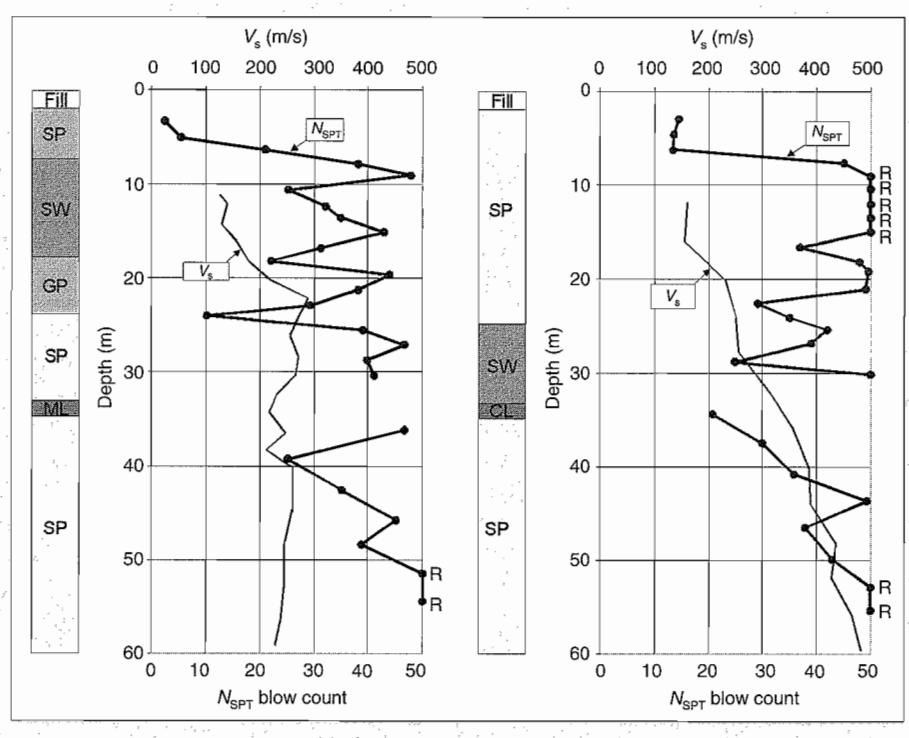



**Fig. 10.6.** Data used for ground-type identification at a deep alluvium site in a northern Italian city: soil profiles (unified soil classification), SPT and $v_s$ profiles from cross-hole tests between two adjacent boreholes at two nearby locations

### *10.4.3.4. Dependence of dynamic soil parameters on the strain level*

The '*calculations involving dynamic soil properties under stable conditions*' referred to in *clause 4.2.3(1)* of EN 1998-5 typically include seismic site response analyses aimed at refining the determination of the design seismic action (elastic response spectrum), especially for 'difficult' soil profiles such as types E, S1 or S2, and calculations related to dynamic soil–foundation interaction, e.g. soil–pile interaction, or soil–retaining wall interaction. *Clause 4.2.3(1)*

The modulus and velocity reduction factors of *Table 4.1* of EN 1998-5, as well as the increase in the damping ratio, are especially appropriate for ground type C, within the depth range (20 m) in which the highest earthquake-induced shear strains are likely to develop. To facilitate selection by the designer, the factors are expressed as a function of the design severity of surface ground motion, rather than of the seismic shear strain amplitude, as commonly done when illustrating results of laboratory tests. Most of the values in *Table 4.1* are derived from back calculations of field observations at sites equipped with vertical accelerometer arrays, with instruments placed at closely spaced depths in the uppermost few tens of metres.[132] More information on curves describing the shear modulus and damping ratio dependence on strain amplitudes may be found in standard geotechnical references (e.g. see Gazetas[133]). *Clauses 4.2.3(2), 4.2.3(3)*

### *10.4.3.5. Internal soil damping*

A damping ratio of 0.03 is not as small as it may seem, and is appropriate for moderate ground shaking (less than 0.10*g*): properly conducted cyclic laboratory tests would give for most soils values of the order of 0.01 or less in the range of very small strains ($10^{-5}$ or less). For more severe ground shaking, higher damping ratios are provided in *Table 4.1* of EN 1998-5. *Clauses 4.2.2(7), 4.2.3(3)*

## 10.5. Foundation system

### 10.5.1. General requirements – seismically induced ground deformation

*Clause 5.1(1)*

One key requirement, stated in *clause 5.1(1)* of EN 1998-5, is that the earthquake-induced permanent deformations in the foundations remain small. Since foundations are placed underground and it is difficult and time-consuming to inspect and repair them, excursions of the soil–foundation system into the plastic deformation range are to be avoided even in severe earthquakes (with design accelerations as high as, say, 0.3–0.4*g* on type A ground). Moreover, permanent deformations are very difficult to estimate, even for simple types of foundations.

In order to restrain the horizontal displacements of the foundations, so that their response remains essentially elastic with small residual displacements, allowable total horizontal displacements may be assumed as a design target. The Japanese standards for the design of highway bridge foundations[134] recommend 1% of the width of the foundation as the allowable horizontal displacement, limited to 50 mm for large (> 5 m) foundations. For foundation piles with a diameter not exceeding 150 cm the recommended limit is 15 mm.

Verifications of the horizontal displacements may often be omitted for shallow foundations (isolated footings and rafts) because, until the initiation of sliding, the ground shear deformation caused by the horizontal forces is the primary cause of the foundation horizontal displacements; such deformation is smaller than that arising in other types of foundations, and very rarely has been observed to exert an adverse influence on the superstructure.

In very rare earthquakes, it is conceivable that inelastic behaviour in the foundation system may occur, for instance in the event that the soil develops a condition of cyclic mobility or even liquefaction. In such exceptional cases, the maximum displacement of the top of the foundations should not reach the range where the response of the foundation system becomes clearly non-linear when the capacity of the yielding elements of the superstructure is reached. In such cases, the quoted Japanese standards recommend that foundations of bridge piers designed for ductile behaviour adopt an approximate design target of 0.02 rad at the top of the foundations.[134]

Effective ways to ensure that the requirements of *clause 5.1(1)* are met include:

- adopting for the foundation system of buildings the optimal, box-type configuration discussed in Section 5.10 in relation to *clause 5.8.1(5)* of EN 1998-1
- ensuring that such a system, in addition to satisfying the safety requirements set forth in *Section 5* of EN 1998-5, has an adequately high factor of safety against bearing capacity failure for static loads.

While this aspect will be better elucidated in Example 10.5 (see p. 238), on the basis of numerical simulation results, it is expected that satisfying the basic stability verifications prescribed in *Section 5* of EN 1998-5 will also ensure that the permanent ground deformations will remain small.

*Clause 5.1(2)*

The properties of *in situ* improved or even substituted soil referred to in *clause 5.1(2)* are typically determined by dynamic (SPT) or static (CPT) penetration tests.

### 10.5.2. Rules for conceptual design

*Clause 5.2(2)*

Concerning *clause 5.2(2)(c)* of EN 1998-5, observational data consistently show that the amplitude of earthquake ground motions, including the peak accelerations, decrease with depth, and may become less than half of the surface value within a depth range that may vary from a few to several tens of metres from the surface. An example is illustrated in Fig. 10.7. Such amplitude variation can be explained by recalling that, under the realistic assumption of P and S earthquake waves propagating in a direction perpendicular to the ground surface, total reflection occurs at the surface, which causes doubling of the amplitude even in a perfectly homogeneous and elastic ground. In a layered soil profile, the motion amplitude generally undergoes additional amplification on approaching the ground surface.

An example of 'appropriate study' such as advocated by *clause 5.2(2)(c)* would consist of:

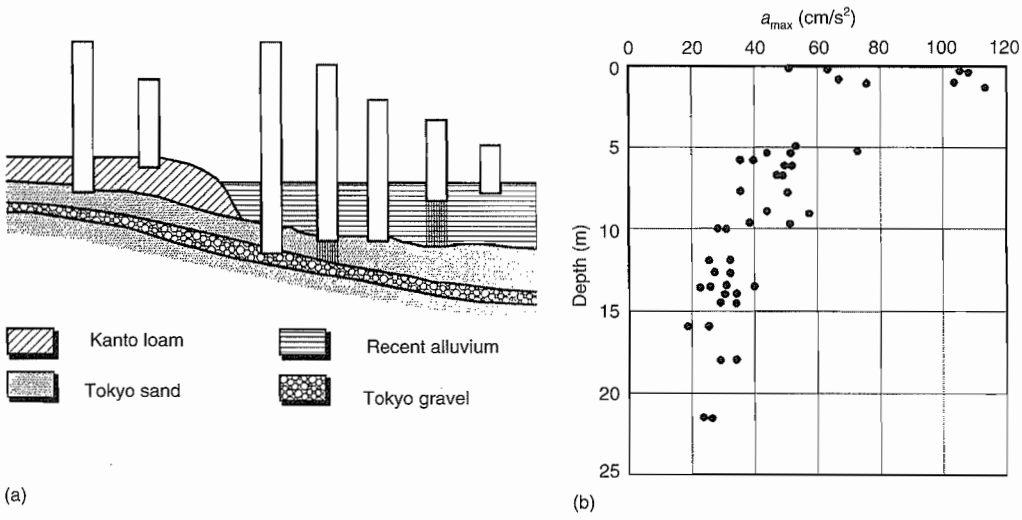



**Fig. 10.7.** (a) Simplified cross-section of near-surface geology in the Tokyo area, showing the different foundation depths of buildings, where ground accelerations were recorded in the $M = 6.4$ Higashi–Matsuyama earthquake of 1968. (b) Maximum acceleration values ($a_{max}$) as a function of depth from ground surface recorded in this earthquake. (After Ohsaki and Hagiwara[135])

- modelling the free-field soil profile as a set of plane and parallel layers extending in depth until reaching a ground formation with, say, $v_s > 500$ m/s
- taking into account the modulus reduction and damping increase as a function of the amplitude of the seismic shear strains
- computing the response of a such a model to, say, three to five acceleration histories complying with the appropriate requirements for the design seismic action
- averaging the depth distributions of the computed maximum accelerations, to provide a sound basis for deciding the reduction of $a_g$ as a function of the depth of the base of the building.

It may be recalled that, in principle, together with the reduction of peak ground motion values with depth, the presence of rotational components of ground motion arising from kinematic interaction effects – usually neglected – would have to be taken into account when a significant embedment comes into play.

### 10.5.3. Transfer of action effects to the ground

*Clauses 5.3.2(1), 5.3.2(2), 5.3.2(3), 5.3.2(4)*

Concerning the transfer of the horizontal force to the ground, engineering judgement was used in *clause 5.3.2(3)* of EN 1998-5 to allow a fraction of it to be resisted by at most 30% of the fully mobilized earth pressure on the front face of the foundation. To gain a more quantitative insight, one may subdivide the total design horizontal force $V_{Ed}$ into the shear force, $V_{Bd}$, acting at the bottom horizontal base of a footing or foundation slab, and into the horizontal force $V_{Sd}$ acting on the vertical front faces of the foundation (where passive earth pressure is mobilized), using the following expressions:[134]

$$V_{Bd} = \frac{1}{1+\beta_H} V_{Ed} \qquad \text{(D10.23a)}$$

$$V_{Sd} = \frac{\beta_H}{1+\beta_H} V_{Ed} \qquad \text{(D10.23b)}$$

where $\beta_H = k_H D_f / 2 k_s B$ is the allotment ratio of horizontal forces, $k_H$ is the coefficient of horizontal subgrade reaction (kilonewtons per cubic metre) (over the sides of the foundation), $D_f$ is the effective embedment depth (metres), $k_s$ is the coefficient of shear subgrade reaction

(kilonewtons per cubic metre) (at the base of the foundation) and $B$ is the width of foundation (metres).

The coefficient $k_H$ may be computed as $k_H = k_{H0} A_H^{1/2}/a_0$, where $k_{H0}$ is typically obtained from an *in situ* plate bearing test with a rigid disc of diameter $a_0$, and $A_H$ is the loading area (square metres) of the foundation perpendicular to the load direction. The coefficient of shear subgrade reaction, $k_s$, may be estimated as $k_s = \lambda k_V$, where $\lambda = 1/3$ and $k_V$ is the coefficient of vertical subgrade reaction (determined by an empirical expression similar to that just given for $k_H$).

Unlike the Japanese standards,[134] which do not rely on side friction as a supporting mechanism for horizontal forces, EN 1998-5 allows for such a mechanism, provided appropriate construction measures are taken to guarantee an effective frictional contact.

Expressions similar to equations (D10.23) are given[134] also on the sharing of the total moment between the base and the sides of the foundation.

## 10.5.4. ULS verifications for shallow or embedded foundations

### *10.5.4.1. Verification of sliding resistance*

*Clauses 5.4.1.1(2), 5.4.1.1(3), 5.4.1.1(4), 5.4.1.1(5), 5.4.1.1(6)*

The design friction resistance $F_{Rd}$ in *equation (5.1)* of EN 1998-5 is controlled by the interface friction angle at the base of the footing, for which reference is made to EN 1997-1, clauses 6.5.3(8) and 6.5.3(11). The latter provide two different sets of expressions depending on whether drained or undrained conditions occur. For drained conditions, the two alternative expressions (in the notation of EN 1998-5) are

$$F_{Rd} = N_{Ed} \tan \delta_d \tag{D10.24a}$$

$$F_{Rd} = \frac{N_{Ed} \tan \delta_k}{\gamma_M} \tag{D10.24b}$$

where $\delta_k$ is the characteristic value of the angle of interface friction and, according to EN 1998-5, $\gamma_M = \gamma_{\phi'}$. Attention should be paid to the fact that the values provided by EN 1997-1 are those of the design friction angle $\delta_d$ in equation (D10.24a) above, not the characteristic values $\delta_k$ of equation (D10.24b). The same applies to the case of drained conditions. Thus, for cast-*in-situ* concrete foundations one would have: $\delta_d = \phi'_{cv,d}$, or (to a first approximation) $\delta = \delta_k = \phi'_{cv}$, to be used in *equation (5.1)* of EN 1998-5.

*Clause 5.4.1.1(7)*

*Clause 5.4.1.1(7)* of EN 1998-5 is equivalent to formulating a performance criterion for footings; for a reasonable limit to the amount of sliding (cf. comments on *clause 5.1(1)* (see p. 236) related to allowable total horizontal displacements).

### *10.5.4.2. Verification of bearing capacity*

*Clause 5.4.1.1(8)*

The general expression and criteria provided in informative *Annex F* of EN 1998-5 to verify the seismic bearing capacity of the foundation have been derived from yield limit analysis of a strip (i.e. two-dimensional foundation), on the basis of rigid-plastic theory.[136] Hence, the ultimate bearing capacity (expressed by the limiting surface of *equation (F.1)* in EN 1998-5 with the equality sign) is not related to settlements. Nevertheless, it is of considerable practical importance to approximately assess the amount of permanent deformation corresponding to the attainment of the limit capacity, as discussed below.

The application of the criteria provided in *Annex F* is illustrated by the following example.

## Example 10.5: verification of the footing of a viaduct pier against bearing capacity failure

### *Description of the problem and input data*

The large shallow footing of the pier of a viaduct, located on the high speed train line under construction in 2004 between Milano and Torino in northern Italy, is to be verified against bearing capacity failure. The dimensions of the foundation, shown in Fig. 10.8, are:

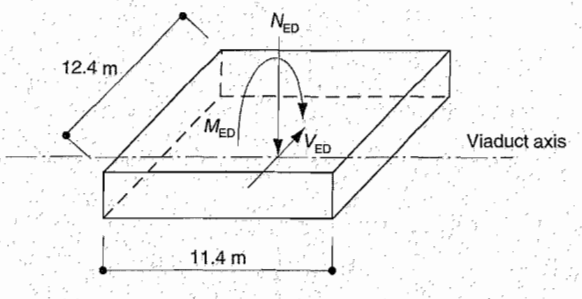



**Fig. 10.8.** The geometry of the shallow foundation of a viaduct pier with the design forces acting on it

**Table 10.5.** Design soil profile for Example 10.5

| Depth from ground surface (m) | Material | Unit weight (kN/m$^3$) | Angle of shearing resistance $\phi'$ (°) | Undrained shear strength (kN/m$^2$) |
|---|---|---|---|---|
| 0–14 | Gravel | 20.5 | 38 | 0 |
| > 14 | Gravel and sand | 19.5 | 37 | 0 |

- plan dimensions: 11.4 × 12.4 m
- elevation of foundation base: +186.85 m a.s.l.
- elevation of ground surface: +191.15 m a.s.l.
- thickness of footing: 2.5 m.

The design soil profile for the depths of interest is given in Table 10.5.

The most unfavourable combination of loads acting on the foundation, obtained from the analysis of the viaduct structure, is: $N_{Ed} = 37550$ kN, $V_{Ed} = 2368$ kN, $M_{Ed} = 35641$ kN m.

The seismic action is determined knowing that the construction site lies in a low-seismicity zone (level 4 according to the current seismic zonation of Italy), with hard ground design acceleration $a_{gR} = 0.05g$. The ground at the site is considered type B (deposit of dense to very dense sand and gravel), for which a soil factor $S = 1.25$ is assumed (the current Italian code uses soil factor values slightly different to those of Eurocode 8). For a unit coefficient of importance, this gives a design ground acceleration: $a_g S = 0.05 \times 1.25 = 0.0625g$.

### *Verification against bearing capacity failure*

According to *equation* (*F.6*) in EN 1998-5, the ultimate bearing capacity, $N_{max}$, per unit length of the strip foundation under a vertical centred load is

$$N_{max} = \frac{1}{2}\rho g\left(1 \pm \frac{a_v}{g}\right) B^2 N_\gamma \qquad \text{(D10.25)}$$

where:

$a_v$ = vertical ground acceleration = $0.5 a_g S = 0.03125g$
$N_\gamma$ = the bearing capacity factor, given by

$$N_\gamma = 2\left[\tan^2\left(45° + \frac{\phi'_d}{2}\right) e^{\pi \tan \phi'_d} + 1\right] \tan \phi'_d = 30.21$$

where the design value of the angle $\phi'_d$ of shearing resistance is given by $\phi'_d = \tan^{-1}(\tan \phi'/\gamma_M) = 32°$, $B = 12.4$ m and $\rho g = 20.5$ kN/m$^3$.

Substituting the previous values gives

$$N_{max} = \tfrac{1}{2} \times 20.5 \times (1 \pm 0.03125) \times 12.4^2 \times 30.21 = \begin{matrix} 49100 \\ 46124 \end{matrix} \text{ kN/m}$$

and, taking into account the actual length of the foundation and using the smaller value, yields the total bearing capacity:

$$N_{max,\,tot} = 46124 \times 12.4 = 571\,938 \text{ kN}$$

For medium-dense to dense sands, *Table F.2* of *Annex F* gives $\gamma_{Rd} = 1.0$. Hence, substituting in *equation (F.2)* of EN 1998-5 yields

$$\bar{N} = \frac{\gamma_{Rd} N_{Ed}}{N_{max,\,tot}} = \frac{37\,550}{571\,938} = 0.06565$$

For a purely cohesionless soil, the dimensionless inertia force is given by (*equation (F.7)* of EN 1998-5)

$$\bar{F} = \frac{a_g S}{g \tan \phi'_d} = \frac{0.05 \times 1.25}{\tan 32°} = 0.10$$

and the value of $\bar{N}$ satisfies the required *condition (F.8)* of *Annex F*:

$$0 < \bar{N} \le (1 - m\bar{F})^{k'} = (1 - 0.96 \times 0.10)^{0.39} = 0.9614$$

Furthermore,

$$\bar{V} = \frac{\gamma_{Rd} V_{Ed}}{N_{max,\,tot}} = \frac{2368}{571\,938} = 0.00414$$

$$\bar{M} = \frac{\gamma_{Rd} M_{Ed}}{B N_{max,\,tot}} = \frac{35\,641}{12.4 \times 571\,938} = 0.00503$$

Substituting all the previous values and the appropriate numerical parameters into *equation (F.1)* now yields

$$\frac{(1 - 0.41 \times 0.08)^{1.14} (2.90 \times 0.00414)}{(0.06565)^{0.92} [(1 - 0.96 \times 0.08)^{0.39} - 0.06565]^{1.25}} + \frac{(1 - 0.32 \times 0.08)^{1.01} \times (2.80 \times 0.00503)^{1.01}}{(0.06565)^{0.92} [(1 - 0.96 \times 0.08)^{0.39} - 0.06565]^{1.25}} - 1 \le 0$$

Since the left-hand side equals –0.66, the inequality is satisfied, and safety against bearing capacity failure is verified with a wide margin.

*Clause 5.4.1.1(9)*

As already discussed in the section devoted to soil strength parameters, strength and stiffness degradation mechanisms will mainly affect either soft cohesive soil with a very high plasticity index and high water content, or loose saturated cohesionless soils. In both cases, shallow foundations would not normally be used.

*Clause 5.4.1.1(11)*

The various critical factors indicated in *clause 5.4.1.1(11)* are clarified in the following by illustrating the results of dynamic non-linear simulations on a simple soil-footing model.

### Example 10.6: non-linear dynamic analyses of a simple soil-footing model

#### *Description of the problem*

The dynamic response of the simple soil–foundation–structure system illustrated in Fig. 10.9 is to be computed, taking as the excitation a vertically propagating signal given by a recorded acceleration history and using a fully non-linear constitutive model for the ground material, allowing for work hardening and cyclic plasticity, as well as a coupled

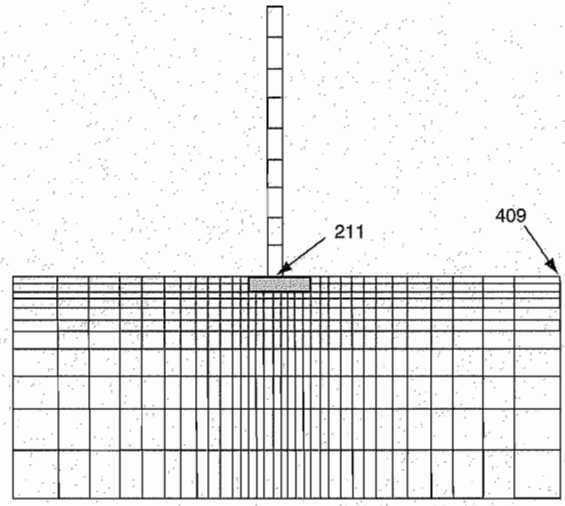



**Fig. 10.9.** Finite-element model of the soil–foundation–structure system of Example 10.6, used to illustrate salient effects of non-linear dynamic response. The numbers refer to mesh nodes at which response histories are calculated (see Fig. 10.12)

description of the solid and fluid phases of the ground material. The main objective is to compute realistic settlements and rotations of the foundation, and to evaluate how these can be related to pseudo-static stability verifications. The task was carried out with the finite-element code of Gefdyn,[137] adopting the constitutive model of Hujeux[138] which makes use of 16 soil parameters. A complete description, including calibration of the soil parameters, has been given elsewhere.[139]

### *Input data*

The input data are:

*Footing*:

- width $B = 4$ m
- embedment $D = 1$ m
- design static vertical load $q_d = 150$ kPa
- safety factor with respect to general failure under vertical loads $F_s = 15$
- vertical static settlement $\delta = 15$ mm.

Note the large static safety factor, not untypical for ordinary shallow foundations on dense soil.

*Structure*:

- fundamental period of vibration $T_0 = 0.5$ s
- height $H = 16$ m
- equivalent height $\zeta = 2.5$ B.

*Soil*:

Hostun RF sand from France, in medium-dense conditions ($D_r = 65\%$), was used. Some typical parameter values for this material are:

- elastic shear modulus $G = 250$ MPa
- elastic bulk modulus $K = 542$ MPa
- angle of shear resistance $\phi = 35.23°$
- permeability coefficient $\kappa = 4 \times 10^{-5}$ m/s.

The soil was modelled as homogeneous, and both undrained and effective stress, i.e. two-phase, analyses were conducted. The soil model parameters were calibrated using the laboratory test data available for the Hostun RF sand, including modulus degradation curves and curves of resistance to liquefaction.[139]

### *Seismic action*

The numerical code used treats the earthquake excitation as an incident acceleration signal, with a prescribed incidence angle. Vertical incidence was considered for simplicity. Also, the code makes use of non-reflecting boundary conditions.

The records selected for excitation are representative of high-seismicity regions of Italy and Greece, although they are not the strongest recorded there. The three acceleration histories shown in Fig. 10.10 were originally used, but only results pertaining to the Gemona 06 NS record, which has a maximum acceleration of $0.33g$ and was obtained at a medium-stiff soil site, are shown here.

The 5% damped response spectrum of this record is compared in Fig. 10.11 with the Eurocode 8 ground type C spectrum. While the record itself was not selected with the purpose of closely matching the code spectrum, the requirement of close agreement among the spectral ordinates at the fundamental period of the structure (0.5 s) was imposed.

### *Initial and boundary conditions*

All the analyses were performed in two stages, namely an initial static analysis under the weight of the structure and foundation, aimed at establishing realistic initial stress conditions for the dynamic analysis, and a subsequent dynamic analysis under the described earthquake excitation applied uniformly at the bottom boundary of the model. Impervious boundaries were assumed for the effective stress analyses.

### *Results*

Selected results showing the time evolution of some response measures at representative points of the model are shown in Fig. 10.12. Foundation displacements (node 211) are calculated relative to the base excitation (node 199), and relative to free-field displacements (node 409), whereas the foundation rocking is obtained by calculating the difference of the vertical displacements at the two sides of the foundation and dividing it by $B$.

For the effective stress conditions the base shear is reduced with respect to the drained conditions, because the pore water has a significant damping effect, and also the permanent deformations are less important. The high value of the effective stress vertical settlement with respect to the free field is a model artefact rather than a true physical effect, because the soil dilates in the vicinity of the impervious lateral boundary. The 'true' free-field vertical settlement ought to be derived from an independent one-dimensional analysis of the soil profile response.

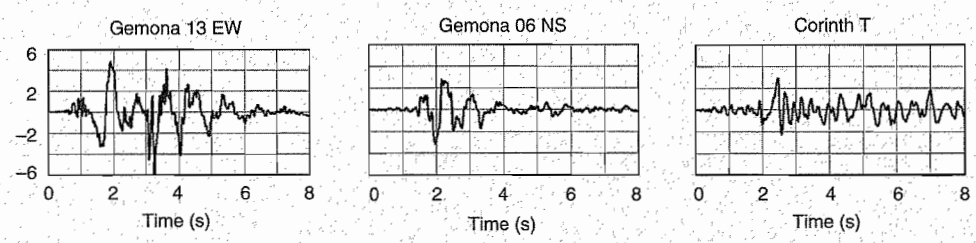



**Fig. 10.10.** Horizontal recorded acceleration histories (in m/s$^2$) used as the excitation (incident waveforms) for the model of Fig. 10.9. The Gemona records were obtained during the strongest shocks of the 1976 Friuli earthquake sequence

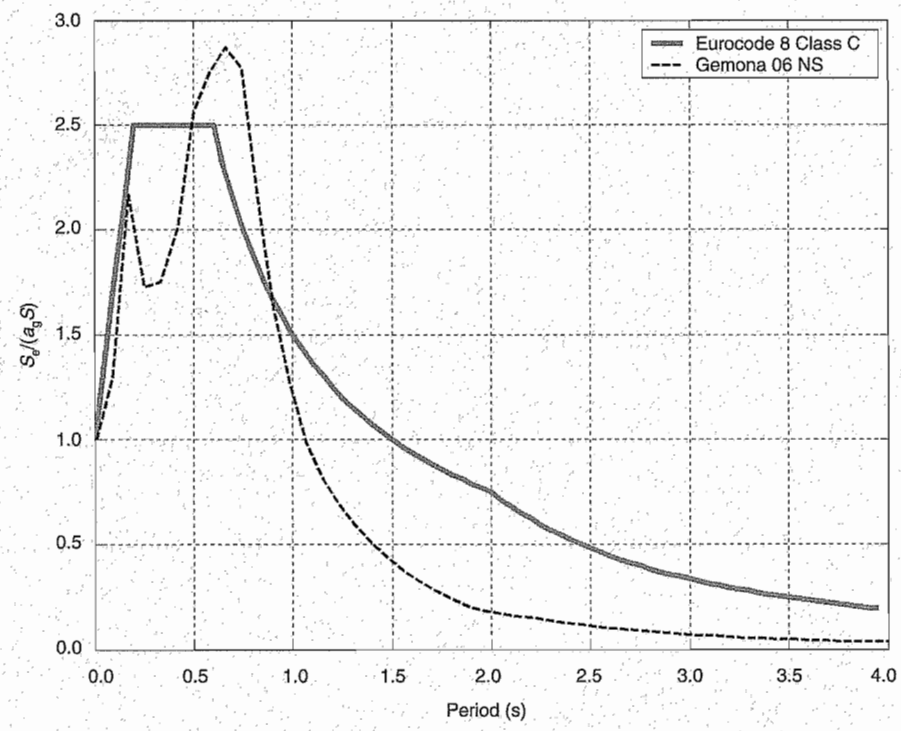



**Fig. 10.11.** Acceleration response spectral shape of the Gemona 06 NS record, shown in Fig. 10.10, compared with the Eurocode 8 elastic spectral shape ($S_e(T)/a_gS$) for ground type C

Since the failure of the system is attained for very high load eccentricities, the foundation rotation attains critical permanent values exceeding 5 mrad in drained conditions, while the permanent horizontal sliding is practically negligible.

To establish a relationship between the results of the dynamic analysis and those of the pseudo-static stability verifications prescribed by EN 1998-5, it should first be noted that when the load eccentricity is considered, the bearing capacity due to the pseudo-static application of the seismic action is strongly reduced. The curves shown in the chart of Fig. 10.13, derived from the general expression *equation* (*F.1*) of the limiting load surface in EN 1998-5, are particularly useful for the evaluation of such a reduction. Each curve in the chart of Fig. 10.13 separates, for the indicated value of the non-dimensional eccentricity $\zeta/B$, the region of the $F_s$ values required for the safe design of the foundation under a seismic coefficient $k_h$ (lying above the curve) from the region of the unsafe ones (lying below the curve).

For the simple structure studied here, the controlling values for the pseudostatic evaluation are

- $\zeta/B = M_{Ed}/V_{Ed}B = (1/k_h)(e/B) = 2.5$
- the static factor of safety $F_s = 15$.

Inserting these values into the chart gives a limit seismic coefficient $k_h = 0.15$, i.e. the bearing capacity the foundation will be exceeded when the base shear transmitted to the foundation, $V/N_{Ed}$, exceeds 0.15. To check this, the graphs in Fig. 10.14 from the previous finite-element dynamic analysis are provided, which show that permanent foundation rotations are progressively accumulated when the base shear exceeds the 0.15 threshold value. Foundation uplift was not allowed for in the analysis.

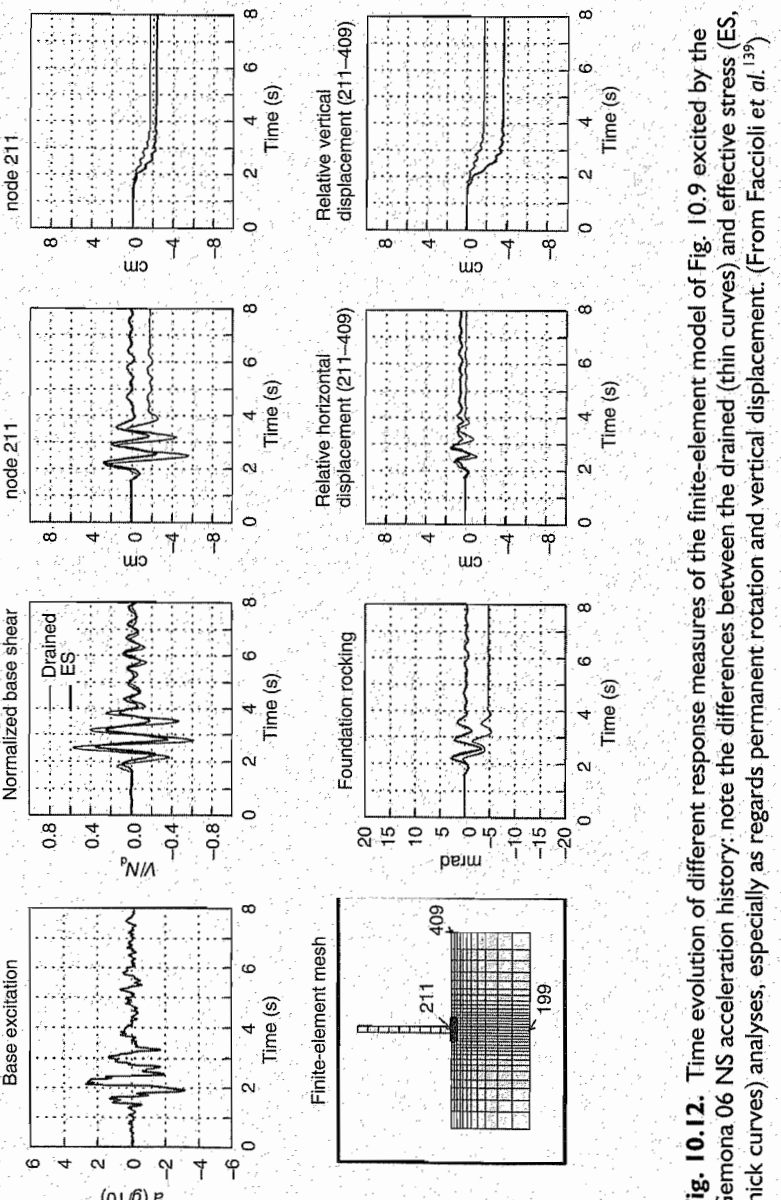



**Fig. 10.12.** Time evolution of different response measures of the finite-element model of Fig. 10.9 excited by the Gemona 06 NS acceleration history: note the differences between the drained (thin curves) and effective stress (ES, thick curves) analyses, especially as regards permanent rotation and vertical displacement. (From Faccioli *et al.*[139])

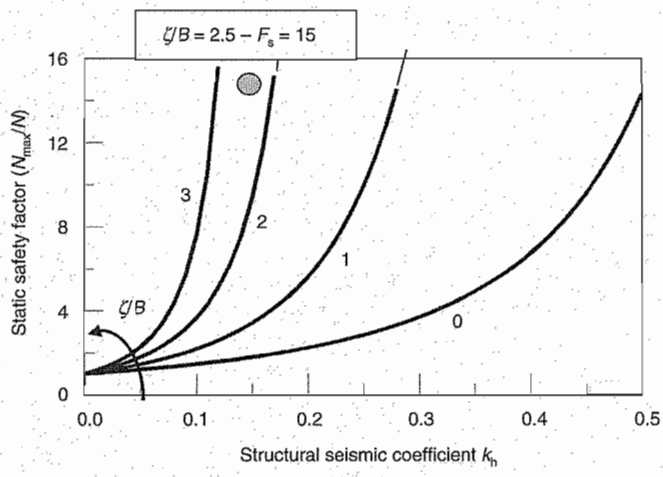



**Fig. 10.13.** Chart for simplified evaluation of seismic effects on the bearing capacity of a shallow strip foundation of width $B$, designed with a static safety factor $F_s = N_{max}/N$. Each curve depends on the value of the non-dimensional eccentricity $\zeta/B = M_{Ed}/(V_{Ed}B)$, and separates the safe domain (above the curve) from the unsafe one (below). The circle with the appended numerical values of $\zeta/B$ and $F_s$ indicates the location of the simple foundation model of Fig. 10.10 in the chart. (After Pecker and Paolucci[140])

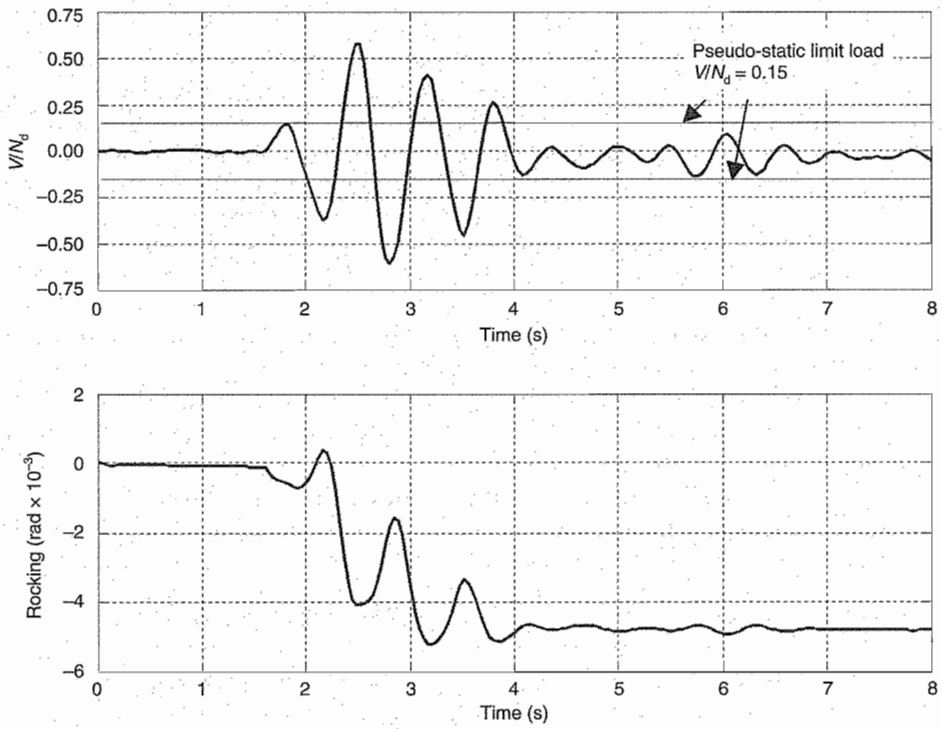



**Fig. 10.14.** Time response of base shear and foundation rocking response to the Gemona 06 NS acceleration history of Fig. 10.10: note the accumulation of permanent rotations each time the base shear exceeds the bounds represented by the pseudo-static limit value of the seismic coefficient. (From Faccioli *et al.*[139])

More general conclusions, also based on additional numerical simulations on both medium-dense and dense sands[139] not discussed here, may be drawn as follows:

- The seismic response of shallow foundations on medium-dense and dense sands is satisfactory, provided the foundations are designed with an adequate static safety factor.
- In the case of medium-dense soils, unacceptable permanent settlements tend to occur only when the pseudo-static limit load is exceeded during several cycles of the excitation.
- The behaviour of shallow foundations resting on dense sands is good even for seismic loads exceeding significantly the pseudo-static limit load, and the small induced permanent displacements are not expected to have harmful effects on the superstructure.
- The principal permanent deformation effects are expected on rocking, when the (transient) load eccentricities are significant.
- If the total settlement of the soil–foundation system is to be taken into account, and not only that of the foundation relative to the free field, the free-field permanent displacements caused by the soil non-linear behaviour can be very important and must be evaluated, especially in the case of thick sand deposits (approximately > 20 m).

### 10.5.5. Piles and piers

#### *10.5.5.1. Introduction*

*Clause 5.4.2(3)*

To start with, a quantitative criterion is needed to decide when a pile may be considered to be flexible, rigid (i.e. like a pier) or semi-flexible. For this purpose, for soils in which the elastic modulus vanishes at the surface, the **elastic length** of the pile is introduced in the form[141]

$$T = (E_p J_p / k)^{0.2} \qquad \text{(D10.26)}$$

where $k$ is the so-called **gradient of the soil modulus** (defined below) and $E_p J_p$ is the flexural stiffness of the pile. In cohesionless soils, $k$ takes values between about 2000 kN/m$^3$ for loose saturated conditions and about 20 000 kN/m$^3$ for dense conditions above the water table. For normally consolidated cohesive materials, $k$ may vary between about 200 and 2000 kN/m$^3$. For a soil in which the elastic modulus $\neq 0$ at the pile head, i.e. when the Young modulus can be expressed in the form $E = E_0 + kz$, the elastic length $T$ can be written as $T = (E_p J_p / E_0)^{0.25}$.

Next, a non-dimensional length is defined as $Z_{max} = L_p / T$, where $L_p$ is the actual length of the pile. Based on these definitions, the following classification is introduced:

- if $Z_{max} > 5$, the **pile is flexible**, i.e. its behaviour is not affected by the length, and collapse is always caused by a flexural failure, with creation of a plastic hinge
- if $5 > Z_{max} > 2.5$, the **pile is semi-flexible**, i.e. its behaviour is affected by the length, and collapse may be caused either by flexural failure, or by attainment of the ultimate resistance of the soil
- if $Z_{max} < 2.5$, the **pile is rigid**, i.e. it behaves like a pier – in this case, the flexural deformation can be neglected with respect to rigid rotation, and the collapse always occurs because the ultimate resistance of the soil is attained.

Attention will be focused on flexible piles in the following. The soil–pile interaction for rigid piles (piers) may be analysed, making reference to limiting equilibrium solutions, such as those proposed by Broms.[142,143] However, finite-element analyses should be employed when pier head displacements and rotations need to be accurately evaluated.

*Clause 5.4.2(1)*

Concerning more specifically the seismic verification of pile foundations, the following two basic conditions are envisaged:

(1) the pile needs to be verified **only** under the effect of inertia forces transmitted from the superstructure onto the head of the pile, which represents the most common situation, and

(2) in addition to the inertia forces of point 1, the pile needs to be verified also under the effect of kinematic forces when certain – relatively infrequent – conditions occur, as specified in *clause 5.4.2(6)* of EN 1998-5.

In both cases an important distinction should be made, due to its bearing on the method of analysis and the design decisions. The pile–soil interaction may be treated essentially as an elastic problem, as described in more detail below, if the horizontal soil pressures are far from their ultimate value, and the horizontal displacement of the head of the pile is limited (indicatively less than about 10–12 mm). This is typically the case for cast-*in-situ*, large diameter (> 1 m) concrete piles embedded in reasonably good soil.

The effects of the inertia forces acting at the pile head are strongly reduced at depths greater than 3.0–3.5 times $T$, so that, in practice, the pile response depends to a large extent on the soil properties within the same depth range, provided the soil profile does not become strongly irregular at a greater depth. Therefore, elastic pile–soil interaction remains a viable assumption as long as the shallow soil layers near the pile head do not suffer rupture or significant plastic deformation. Otherwise, the elastic theory is no longer applicable, and one has to resort to a fully non-linear approach, such as the so-called ***p–y* curves** discussed below. The need to take the non-linearity of the soil–pile interaction into account is also strongly influenced by the pile diameter: when small-diameter (e.g. 0.2–0.3 m) driven piles, or micro-piles, are used, elastic theory is frequently not applicable.

#### *10.5.5.2. Absence of kinematic action effects*

The easiest option for seismic pile analysis and verification is provided by a pseudo-static approach, and the pile resistance against failure under transverse earthquake-generated loads may be checked following the basic indication of EN 1997-1 (clause 7.7.3(3)), i.e. *Clauses 5.4.2(1), 5.4.2(6), 5.4.2(3)*

> The calculation of the transverse resistance of a long slender (i.e. flexible) pile may be carried out using the theory of a beam loaded at the top and supported by a deformable medium characterised by a horizontal modulus of subgrade reaction.

For low-seismicity zones, such that $a_g S \leq 0.1g$, and in the presence of a ground profile type C or D, or even ground type E, the soil–pile behaviour is expected to be essentially elastic. Then, detailed response calculations on the single piles would generally not be needed, and the designer may rely on standard solutions and charts for pile moments and shear forces derived from the elastic model of a beam-on-Winkler type soil (e.g. see Matlock and Reese[141]), making reference to appropriate values of 'operational' secant values of the soil horizontal modulus of subgrade reaction. When large-diameter piles are employed with a ground profile not worse than C, the elastic soil–pile analysis may be extended to values of $a_g S$ larger than $0.1g$.

When elastic soil–pile interaction can be assumed, a full dynamic approach may also be used, in which the soil reaction to the pile horizontal displacement is evaluated, e.g. starting from the exact dynamic solution for a thin horizontal soil annulus around the pile. One obtains in this way frequency-dependent shear and bending moment along the pile, as well as frequency-dependent, equivalent spring and dashpots for each mode of vibration of the pile.[144] After taking pile group effects into consideration, the concentrated parameters can be inserted at the base of the structure to account for the soil–foundation interaction effects, if a dynamic analysis of the structure itself is carried out. In such cases the equivalent parameters may be evaluated at, for example, the fundamental frequency of vibration of the structure.

Pile group effects may be neglected for small pile groups (e.g. up to three piles), while for larger groups the recommendations given further below may provide a reference.

Computational tools are available to determine dynamic pile–soil–pile interaction effects and pile group effects,[145] but the results in terms of dynamic transfer functions, e.g. of pile bending moments as a function of frequency, exhibit strong peaks in correspondence to some of the natural frequencies of the system, whose practical significance is difficult to assess.

When it is anticipated that unavoidable soil yielding and/or failure of the shallow layers govern the soil–pile interaction (indicative of $a_g S > 0.1g$ and a ground profile of type C, D, E, S1 or S2) but the structure to be designed is of ordinary importance, more detailed calculations than in the previous case may be needed, depending on the pile diameter and the soil resistance at shallow depth, so as to introduce the non-linearity in the response of the soil–pile system. In these cases, taking as the point of departure the equation of the elastic beam under static flexural action,

$$E_p J_p \frac{d^4 y}{dz^4} = -p \tag{D10.27}$$

in which $y$ is the horizontal pile deflection, $z$ is the depth and $p$ is the soil reaction, the soil modulus is introduced as $E_s = p/y = k_s D$, where $k_s$ is the **coefficient of horizontal subgrade reaction** and $D$ is the pile diameter. The equation to be solved, e.g. by finite-element or finite-difference methods, then becomes

$$E_p J_p \frac{d^4 y}{dz^4} + E_s y = 0 \tag{D10.28}$$

with appropriate boundary conditions (which bring the applied loads into play).

Several formulations have been proposed for the dependence of $E_s$ (basically a semi-empirical parameter) on depth, displacement amplitude and soil properties (see Jamiolkowski and Garassino[146] for a review). In the case under discussion, i.e. when $E_s$ is a (strongly) non-linear function of $y$ and $z$, it is preferable to refer directly to the so called $p$–$y$ curves, i.e. to the full relationship between soil reaction and pile deflection. In this way, the soil modulus under a given acting load is defined by:

- a simple mathematical expression derived from the $p$-$y$ curve, such as:

$$\frac{1}{E_s} = \frac{1}{E_{si}} + \frac{1}{p_{lim}} \frac{y}{D} \tag{D10.29}$$

  which is well adopted for sands and soft normally consolidated clays
- an initial tangent soil modulus, $E_{si}$, generally varying with depth
- an ultimate unit soil resistance, $p_{lim}$ (or $p_u$), also a function of depth.

The numerical solution of equation (D10.28) in conjunction with equation (D10.29) obviously requires an iterative procedure.

An expression of the type $E_{si} = k_i z^n$ is often used to determine the initial tangent modulus for cohesionless and NC soil deposits, where $k_i$ can be estimated or computed on the basis of $D_r$ or $c_u$.

As regards the pile group effects in the presence of seismic action, it is recommended in Gazetas:[133]

- to assume the coefficient of horizontal subgrade reaction of each grouped pile practically equal to that of an isolated pile, when the pile-to-pile distance is not less than $2.5D$
- to decrease the coefficient of horizontal subgrade reaction of each single pile in the group by multiplying it by a reduction factor $\mu$ which depends on the ratio $L/D$, where $L$ is the distance between adjacent pile centres, as follows:

$$\mu = 1 - 0.2\left(2.5 - \frac{L}{D}\right) \qquad L < 2.5D \tag{D10.30}$$

#### *10.5.5.3. Ultimate lateral soil resistance*

*Clause 5.4.2(2)* The determination of the ultimate soil resistance for laterally loaded piles is a complex problem of the ULS of an elastic plastic medium, for which rigorous closed-form solutions are not available.[146]

For piles embedded in cohesionless soils the simple expression

$$p_u = a\tan^2\left(45° + \frac{\phi'}{2}\right) D\sigma'_{v0} \tag{D10.31}$$

has been proposed,[142,143] where $a$ is an adjustment factor taking values between 3 and 4. Equation (D10.31) makes it very clear that at shallow depths the ultimate resistance tends to become very small, due to the dependence on the effective vertical stress.

For piles embedded in cohesive materials under undrained static loading conditions, one may refer to the following empirical expressions:[147]

$$p_{uw} = \left(2 + \frac{\sigma'_{v0}}{c_u} + 2.83\frac{z}{D}\right) c_u D \qquad z \le z_{crit} \tag{D10.32a}$$

$$p_{uf} = 11 c_u D \qquad z > z_{crit} \tag{D10.32b}$$

The value of $z_{crit}$ is determined for $p_{uw} = p_{uf}$. It may be recalled that, for normally consolidated materials, $c_u$ is typically a linear function of depth.

Spurred on by offshore platform design needs, for which wave loading becomes a key factor when the pile foundations are embedded in soft clays or loose sands, a number of modifications have been proposed to the static $p$–$y$ curves for piles to take cyclic loading conditions into account. These are especially significant for soft and stiff clays, for which typical strain-softening effects may substantially reduce the ultimate soil resistance under static loading.[146] It is, however, not so evident whether the cyclic modifications developed for wave-loading effects are directly applicable to earthquake loading, except perhaps for cases of very soft clays with high water content and long durations of strong ground motion.

#### *10.5.5.4. Presence of kinematic action effects*

*Clause 5.4.2(6)* of EN 1998-5 stipulates the conditions under which the kinematically induced bending moments need to be taken into account when checking the safety of piles. As in the previous case, the simplest way is to follow a pseudo-static approach, through the following steps:[148]

*Clause 5.4.2(6)*

- the actions generated by the soil motion (kinematic interaction) are idealized as equivalent static soil deformation relative to the depth of the pile tips
- such actions are conveniently specified based on the peak displacement distribution of the free-field response, obtained from, for example, a site-specific one-dimensional seismic response analysis of the soil profile
- the distribution of peak displacements with depth in the soil profile, for a given excitation accelerogram, can be obtained when the maximum relative displacement occurs between the top and the bottom of the deep foundation
- the displacement distribution thus obtained is imposed statically at the supports of the springs of the beam-on-elastic foundation model, in addition to the inertia loads acting on the pile head.

In regular soil profiles the free-field maximum horizontal displacement distribution as a function of depth frequently corresponds to the fundamental mode of vibration of the soil column. For a homogeneous and elastic soil layer overlying bedrock, the displacement profile in the fundamental horizontal mode of vibration is $y(z) = y_0(\gamma_r v_{s,r}/\gamma_s v_{s,s})\cos(\pi z/2H)$, where $H$ is the thickness of the layer, the origin of the $z$ axis is at ground surface, the first term in parentheses is the seismic impedance contrast between bedrock and soil, and $y_0$ is the displacement amplitude on top of exposed bedrock. The fundamental mode corresponds to the frequency $f_0 = v_s/4H$.

If the free-field response to selected accelerograms is computed, and the displacement histories are derived at various depths from those of acceleration, strain-compatible values of the soil properties must be used, in accordance with the criteria already discussed in Section 10.4.3.4. General compatibility between such free-field properties and those entering in the soil resistance for the pile analysis should be ensured.

## 10.6. Soil–structure interaction

*Clause 6.1*

Earthquake ground motions give rise to two types of interaction mechanisms between the structure, the foundation and the soil, namely:

- **Kinematic interaction**: stiff, slab-like or deeply embedded foundation elements cause foundation motions to be different from free-field motions because of wave-scattering phenomena, wave inclination or embedment. Kinematic effects typically manifest themselves through a suppression of the higher-frequency components in the motion of the foundation with respect to that of the free-field, the frequency cut-off being typically a function of the foundation size and the value of $v_s$ in the soil;
- **Inertial interaction**: inertia forces developed in the structure cause displacements of the foundation relative to the free field. Frequency-dependent foundation impedance functions are introduced to describe the flexibility of the foundation support, as well as the so-called radiation damping associated with soil–foundation interaction.

In the absence of large, rigid foundation slabs or of deep embedment, inertial interaction tends to be more important.

To quantify inertial interaction effects, a structure with a surface foundation can be accurately represented by an equivalent, **fixed-base** one-degree-of-freedom oscillator with period $\tilde{T}$ and damping ratio $\tilde{\zeta}$, which represent the properties of an oscillator that is allowed to translate and rotate at its base. Hence, $\tilde{T}$ and $\tilde{\zeta}$ differ from the corresponding properties $T$ and $\zeta$ of the fixed-base oscillator; so, the interaction effects are conveniently expressed by the quantities $\tilde{T}/T$ (lengthening of the period) and $\tilde{\zeta}$. Expressions for these 'flexible-base' parameters are widely quoted in the literature (e.g. see Stewart *et al.*[149]).

The list in *clause 6.1* of EN 1998-5 restricts the necessity of taking dynamic soil–structure interaction effects into consideration to a few cases in which particular combinations of structure geometry and soil conditions occur. Such restrictions are generally supported by observations at real instrumented building sites. Indications from a particularly comprehensive survey of such observations[150] are that:

- free-field and foundation-level ground motions appear to be comparable, i.e. kinematic interaction effects are negligible, except for deeply embedded foundations
- the ratio of structure-to-soil stiffness, $h/(v_s T)$, where $h$ is the effective structure height, is the factor with the greatest influence on the period lengthening and the foundation damping factor.

When the stiffness ratio tends to zero, these two parameters are about 1.0 and 0, respectively, whereas for the largest observed stiffness ratio (about 1.5, for a nuclear reactor building) the period lengthening reached a value of about 4 and the damping ratio about 30%. For stiffness ratios < 0.15, which correspond to many practical cases, the inertial interaction effects are negligible. However, the empirical data exhibit a very large scatter, and clear-cut practical recommendations are not readily obtainable from them.

## 10.7. Earth-retaining structures

### 10.7.1. General design considerations

*Clauses 7.1(1), 7.2(6)*

Many complex and interacting phenomena, in addition to the inherent variability and uncertainty of soil properties, are involved in the dynamic response of earth-retaining structures to earthquake ground motions (e.g. see Kramer[151]): even within the restricted context of providing guidance to designers in the interpretation and application of the provisions of EN 1998-5, it is not possible to discuss all the relevant aspects that may affect seismic design.

To put the task of seismic safety verifications of earth-retaining structures into better focus, it is recalled that:

- Cases of collapse of retaining structures with significant physical and economic consequences have been rarely documented in modern destructive European earthquakes: e.g. see Baratta[152] for the damage to the ports of Messina and Reggio Calabria caused by the disastrous Messina Straits earthquake of 1908 (where very little modern retaining structure was involved) and, more recently, the damage inflicted to the Ulcinj harbour quay walls by the 1979 Montenegro earthquake (caused by backfill liquefaction). By contrast, severe damage to port installations has been caused in Japan in several destructive earthquakes by the collapse of quay walls and embankments, the last in Kobe in 1995. The main damage was caused by gross instability (due to liquefaction) of the non-compacted backfill soils behind and below gravity-type retaining structures, often consisting of caissons.
- Even under static conditions, predicting actual forces and deformations of earth-retaining structures is a difficult task. For this reason, wall deformations are seldom considered explicitly in design and 'the typical approach is to estimate the forces acting on a wall and then to design the wall to resist those forces with a factor of safety high enough to produce acceptably small deformations'.[151] Simplified approaches are typically used to evaluate static, and pseudo-static, loads on retaining walls. *Clause 7.3.2.3* and *Annex E* of EN 1998-5 explicitly refer to the most commonly used simplified approach, i.e. the Mononobe–Okabe method.
- Simplified pseudo-static approaches, such as the above-mentioned Mononobe–Okabe method, rest on crude simplifications of the actual dynamic soil–structure interaction problem; their level of approximation and conservatism is often hard to assess. While there were no alternatives to using these simplified approaches when efficient numerical tools were not available, the situation has drastically changed nowadays, especially for important retaining structures. In such cases designers are encouraged to use non-linear finite-element or finite-difference analysis techniques, paying appropriate attention to the crucial aspects of the analysis. Examples 10.7 and 10.8 provide a comparison of results obtained by the simplified pseudo-static approach and by a full non-linear dynamic analysis.

Implicit in the design of a retaining structure must be the understanding of the failure modes ('limit' modes according to Eurocode 7) that can affect the soil–structure system. Clause 9.7 of Eurocode 7 provides a detailed graphical description of all the failure modes that need to be taken into account, i.e. overall stability, foundation failure (for gravity walls), rotational failure of embedded walls, vertical failure of embedded walls and modes for structural failures.

Permanent displacements, albeit of limited extent, always occur in the so-called **yielding walls** in practice, i.e. walls that move an amount sufficient to develop minimum active (on the outer side) and maximum passive (on the inner side) earth pressures. *Clauses 7.1(2), 7.3.2.1(2), 7.3.2.2(3)*

*Clauses 7.2(3)* to *7.2(6)* of EN 1998-5 stress the need of ensuring wide margins of safety against the build-up of pore pressure and the occurrence of liquefaction in the backfill, because, as already noted, this has been the main cause of retaining wall failures in many earthquakes. *Clauses 7.2(3), 7.2(4), 7.2(5), 7.2(6)*

### 10.7.2. Basic models

The basis of models used in pseudo-static approaches for computing the earth pressures are readily available in standard geotechnical engineering references. In particular the Mononobe-Okabe method is derived form the static Coulomb theory, by introducing the additional forces due to the pseudo-static accelerations and by obtaining the pseudo-static soil thrust through equilibrium of the forces acting on the soil wedge.[151] *Clause 7.3.2.1(1), 7.3.2.1(3)*

*Clause 7.3.2.1(3)* of EN 1998-5 refers to non-yielding retaining walls, which cannot move sufficiently to develop active and passive earth pressures.

### 10.7.3. Seismic action

*Clause 7.3.2.2(3)* The concept expressed by *clause 7.3.2.2(3)* of EN 1998-5 is in many ways similar to that of ductility, and the associated reduction factor of elastic forces, used in structural analysis. The reasoning, illustrated in Section 10.4.1.3 when discussing pseudo-static inertia forces to be used in stability verification of slopes in connection with *clause 4.1.3.3(5)* of EN 1998-5, can be also applied for retaining structures. By increasing the value of the seismic coefficients, $k_h$ and $k_v$, the permanent displacements corresponding to a safety factor greater than 1 are progressively decreased, obviously at the expense of increasing costs.

*Clause 7.3.2.2(4)* For retaining structures, the use of a horizontal seismic coefficient equal to about 50% of the actual design acceleration may at least in part be traced back to Japanese practice, documented notably by the Overseas Coastal Area Development Institute of Japan[153] (see the graph on p. 188 therein). By fitting the data points obtained from back-analyses of many real cases,[153] it was recommended that a value of $k_h g$ equal to the maximum acceleration recorded by old SMAC accelerographs (which have been out of use for several years) is adopted; these devices suffered from severe band limitation in the recording of high frequencies of ground motion due to excessive damping. Taking such limitations into account and converting into the maximum accelerations ($a_{max}$) that would be recorded by modern (analogue) accelerographs, simple reasoning suggests that the recommendation by the Overseas Coastal Area Development Institute of Japan[153] would become roughly equivalent to taking $k_h = 0.3$–$0.5$ times $a_{max}$, where the lower value in the range corresponds to smaller magnitudes and the larger value to larger ones (indicative of $M > 5.5$). The previous relationship between $k_h$ and $a_{max}$ would be valid for $a_{max} < 0.4g$.

The designer should be aware that the magnitude of the pseudo-static seismic action prescribed in *clause 7.3.2.2(4)* of EN 1998-5 (**in the absence of specific studies**) can lead to conservative design of the retaining structure. Therefore, he or she may want to make an independent evaluation of the wall permanent displacement, for the purpose of choosing a different value of the design action through a more accurate estimate of the reduction factor $r$, amounting to a specific study. In addition to finite-element analyses, applicable to any type of retaining structure, permanent displacements of gravity walls can be estimated by simplified methods similar to the Newmark sliding block approach discussed in Section 10.4.1.3, in connection with *clause 4.1.3.3(7)* of EN 1998-5 on slope stability analysis (see Example 10.1). The best known of such methods[154] provides a simple expression for the wall permanent displacement, calibrated from numerical simulations, which is a function of the design peak ground acceleration and velocity and of the yield (or critical) acceleration of the wall. The method can be directly applied to compute the response of a specific wall to appropriately selected accelerograms.

The dynamic analysis discussed in Example 10.8 will illustrate the extent of the amplification of the seismic action over a height in excess of 10 m.

### 10.7.4. Design earth and water pressure

*Clause 7.3.2.3* Appropriate attention must be paid to the fact that the (factored) value of the angle of shearing resistance in terms of **effective stress,** $\phi'$, is used in the expressions (*equations (E.2)–(E.4)*) in EN 1998-5 for the Mononobe–Okabe earth pressure coefficient, irrespective of the drainage conditions assumed in the stability verification. In fact, expressing the rupture criterion in effective stresses (i.e. $\tau_f = (\sigma_f - u)\tan\phi'$), does not prevent one from using it in undrained conditions, provided the pore pressure $u$ on the failure surface can be estimated.

*Clause 7.3.2.3(7)* The formulae in *Annex E* in EN 1998-5 are precisely designed to handle the different situations that may arise as regards the drainage conditions through the equivalent angle $\theta$ and the actual or buoyant unit weight of the soil $\gamma^*$ and by considering the following different situations:

- **Dry backfill** (*equation (E.5)*).

- **Saturated impermeable backfill**: the pore water is not free to move with respect to the soil during the seismic action, and the backfill responds dynamically like a single undrained medium, acted upon by inertia forces proportional to the total unit weight of the soil. *Clauses 7.3.2.3(8), 7.3.2.3(9), 7.3.2.3(10)*
- **Saturated permeable backfill**: the pore water is free to move with respect to the soil, and the effects of the seismic action on the soil and on the water are essentially independent. In this case the inertia forces are proportional to the buoyant unit weight of soil and a hydrodynamic water thrust develops, in addition to the hydrostatic one and to the soil thrust. *Clauses 7.3.2.3(11), 7.3.2.3(12)*

#### *10.7.4.1. Limitations of validity for the earth pressure formulae in EN 1998-5 (equations (E.2)–(E.4))*

The Mononobe–Okabe expressions given in *Annex E* of EN 1998-5 for the earth pressure are applicable, provided that the seismically induced excess pore pressure $\Delta u$ remains limited and the backfilling soil is far from a liquefaction condition. If $\Delta u$ increases significantly, the earth pressure formulae require modifications to account for the influence of $\Delta u$ on the evaluation of $\theta$ and $\gamma^*$. For a moderate pore pressure increase, a reduced effective angle of shearing resistance such as given by equation (D10.12) (proposed by Bouckovalas and Cascone[155]) may be used. Hence, when problems of this type are encountered, pseudo-static methods such as that of the Mononobe–Okabe approach should not be used in conjunction with shear strength parameters expressed in terms of total stress.

For a constant $\phi'$, the value of the active earth pressure coefficient $K$ provided by *equation (E.2)* in EN 1998-5 increases (non-linearly) with increasing $k_h$, until reaching an upper limit when the term $(\phi' - \beta - \theta)$ under the square root in *equation (E.2)* in EN 1998-5 becomes negative. This denotes a limit state beyond which the pseudo-static equilibrium of forces acting on the soil wedge of the basic model of *clause 7.3.2.1(1)* of EN 1998-5 can no longer be maintained, and the seismic earth pressure can no longer be evaluated using the Mononobe–Okabe method. This limitation, typically attained under very strong ground motions, stems from the fact that in the Mononobe–Okabe approach the shear resistance is assumed to be uniform, isotropic and constant, whereas 'the behaviour of a sliding mass is affected by such factors as strength anisotropy, progressive failure and strain localisation (a shear resistance angle mobilised along a failure plane reduces from a peak value $\phi_p$ to a residual value $\phi_{res}$).[148] Modifications have been proposed to the Mononobe–Okabe method to take into account the effects of strain localization into a shear band and associated strain softening in the shear band, but it would seem preferable to have recourse to finite-element analyses with realistic constitutive soil descriptions.

#### *10.7.4.2. Earth pressure for rigid structures*

The background for *equation (E.19)* in EN 1998-5 is the analysis of a homogeneous elastic soil layer of depth $H$ trapped between two rigid walls separated by a distance $L$.[156] It can be shown that for $L/H > 4$ the pressures acting on one wall will actually be independent from those acting on the other. In this case, and for the frequencies of excitation encountered in most practical problems, dynamic amplification is negligible, and the pressures derive from the elastic solution of a constant acceleration applied throughout the soil. *Equation (E.19)* in EN 1998-5 is applicable only to a homogeneous elastic soil, without any consideration for the fluid phase or the development of excess pore pressure. The point of application of the dynamic force at mid-height of the wall is actually a simplification of a more rigorous estimation, giving about $0.6H$ from the base. *Clause 7.3.2.1(3)*

### Example 10.7: simplified seismic analysis of a flexible earth-retaining structure with the pseudo-static approach

#### *Description of the problem*

This example illustrates how to compute with a pseudo-static analysis the earthquake effects on a simple retaining structure consisting of a reinforced-concrete pile wall,

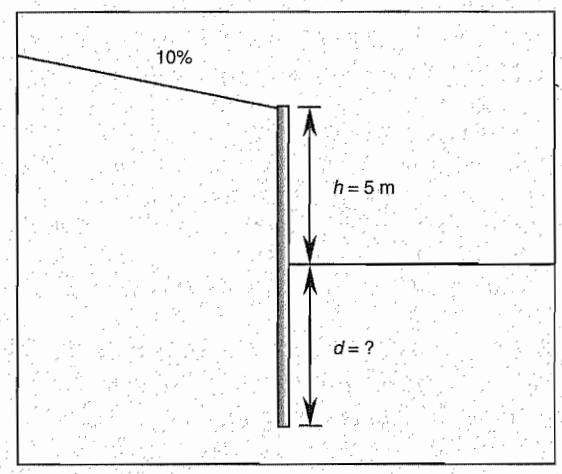



**Fig. 10.15.** The geometry of the pile wall and slope of Example 10.7

without any anchor or strut. The main function of the wall is that of a retaining structure near a train line; no building structure is present in its vicinity.

Since the stability of the structure is in this case ensured only by the passive soil resistance mobilized in front of the wall, the first step in design is the evaluation of the embedment depth necessary to prevent wall failure under static conditions. As the wall is flexible, and the earth pressure is influenced by its deformability and displacement, the soil–structure interaction should be considered. This may be done with the help of readily available computer programs, which model the wall as an elastic beam on a Winkler-type soil, represented by elastic–perfectly plastic springs. The same programs also allow seismic stability verification using the pseudo-static approach to be undertaken, as shown in detail below.

### *Input data*

The input data are (Fig. 10.15):

*Wall*:

- pile diameter $D = 1$ m
- pile spacing $I = 1.5$ m
- required height of wall above ground $h = 5$ m
- Young modulus of concrete $E_c = 28$ GPa
- Poisson coefficient of concrete $\nu_c = 0.15$.

*Soil*:

- unit weight $\gamma = 20$ kN/m$^3$
- angle of shearing resistance $\phi'_k = 32°$ (characteristic value)
- slope angle of the backfill surface $\beta = 5.7°$ (= 10%)
- Young modulus of soil $E_s = 25$ MPa
- Poisson coefficient of soil $\nu_s = 0.25$.

The value of the Young modulus of soil implies the mobilization of (significant) soil deformations, compatible with the structure considered. It can be assumed to amount to 1/5 to 1/6 of the small deformation ($\sim 10^{-6}$) modulus $E_0$.

### *Design seismic action*

The design seismic action is specified as follows:

- acceleration on type A ground $a_{gR} = 0.15g$ (from the national seismic zonation map)

- soil factor $S = 1.25$
- importance factor $\gamma_I = 1.0$
- design acceleration $= 0.15 \times 1.25 = 0.19g$.

An alternative choice for the design seismic action will also be considered, in which the design acceleration will be taken as $a_g = 0.10g$, consistent with the results of the dynamic analysis illustrated in Example 10.8 and with *clauses 7.3.2.2(6)* and *E.2* of EN 1998-5.

The soil factor $S$ takes the value recommended by the current earthquake code of Italy for ground types B, C and D, and does not coincide with those recommended in EN 1998-1 for either type 1 or type 2 spectra.

***Static design of the wall: determination of the design embedment depth***

Following approach 1 (DA-1) of EN 1997-1 (clause 2.4.7.3(4)), the design is governed by combination 2 (CA-2). The pile wall behaviour is therefore analysed with the soil shear strength reduced to

$$\tan\phi'_d = \frac{\tan\phi'_k}{1.25} \Rightarrow \phi'_d = \tan^{-1}\left(\frac{\tan\phi'_k}{1.25}\right) = 26.6°$$

with the angle of friction between the soil and the wall taken, for static design purposes, as

$$\delta_d = \tan^{-1}\left(\frac{\tan\delta_k}{1.25}\right) = 17.1°$$

The embedment depth $d$ is determined through a soil–structure interaction analysis where, as already mentioned, the wall is modelled as an elastic beam and the soil as a series of horizontal elastic–plastic springs. Computer programs commonly employed to perform these calculations require, as input data, the horizontal component of earth pressure coefficients, in the active and passive case, both for the retained soil at the back of the wall ('uphill') and for the soil in front of the wall ('downhill'). The coefficients can be determined, as functions of $\phi_d$, $\delta_d$ and $\beta$, by well-established methods. The charts provided in Annex C of EN 1997-1 have been used for this purpose.

The active ($K_A$) and passive ($K_P$) earth pressure coefficients, all referred to forces inclined at an angle $\delta_d$ with respect to the normal to the wall, are:

- coefficient of active earth pressure for retained soil $K_A^{up} = 0.367$
- coefficient of passive earth pressure for retained soil $K_P^{up} = 6.047$
- coefficient of active earth pressure for soil in front of the wall $K_A^{down} = 0.339$
- coefficient of passive earth pressure for soil in front of the wall $K_P^{down} = 4.602$.

The corresponding values of the horizontal components are:

- $K_{AH}^{up} = K_A^{up} \cos\delta_d = 0.35$
- $K_{PH}^{up} = K_P^{up} \cos\delta_d = 5.76$
- $K_{AH}^{down} = K_A^{down} \cos\delta_d = 0.323$
- $K_{PH}^{down} = K_P^{down} \cos\delta_d = 4.383$.

The elastic–plastic spring stiffness values which describe the non-linear soil behaviour are usually determined within the programs on the basis of the previous coefficients and of the elastic soil parameters.

Several interaction analyses were performed for parametric purposes using the previous earth pressure parameters, varying the embedment depth of the wall. The horizontal displacement, $U$, of a representative point, i.e. the top of the wall, was computed, and a curve of displacement versus embedment depth $d$ plotted (Fig. 10.16). The point where this curve shows a sharp increment in the slope is used to determine the design embedment depth of the wall. For this example, $d = 7$ m was chosen, and the total height of the wall is therefore $H = 12$ m.

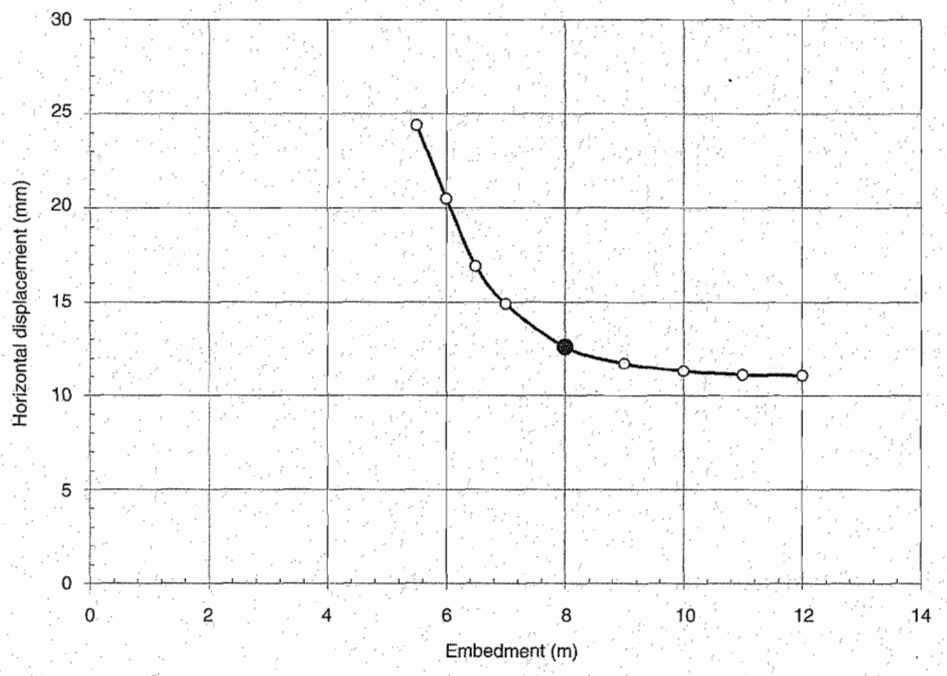



**Fig. 10.16.** Horizontal displacement, *U*, of the top of pile wall versus the embedment depth *d* (see Fig. 10.15). The solid black dot indicates the design embedment

### *Seismic analysis using the pseudo-static approach*

The first step in the seismic analysis is to evaluate the seismic coefficients for the site and the relevant structure.

With the first choice of the design seismic action ($a_g = 0.19g$) the horizontal seismic coefficient, in the absence of topographic amplification ($S_T = 1.0$), is (see *clause 7.1* of EN 1998-5)

$$k_h = S\frac{a_{gR}}{g}\frac{1}{r} = 1.25 \times \frac{0.15}{2} = 0.0938$$

where $r = 2$ is the reduction factor for the design ground acceleration ratio, related to the structure flexibility. The vertical seismic coefficient, assuming a ratio of vertical-to-horizontal design accelerations larger than 0.6, is (see *clause 7.2* in EN 1998-5) $k_v = \pm 0.5 k_h = \pm 0.0469$.

With the second choice of the design seismic action ($a_g = 0.10g$), the horizontal and vertical seismic coefficients are $k_h = 0.10/2 = 0.05$ and $k_v = \pm 0.025$.

#### *Calculation of the earth pressure and seismic force due to the backfill*

The overall (static plus dynamic) earth pressure coefficient is computed with the Mononobe–Okabe formula for active states (*equation (E.2)* in EN 1998-5):

$$K_A^{*up}(\phi'_k, \delta_k, \beta, k_h, k_v) = 0.455$$

The horizontal component is

$$K_{AH}^{*up} = K_A^{*up} \cos\delta_k = 0.434 \qquad \text{(active state, seismic condition)}$$

The overall (static plus dynamic) force acting from the landward side is given by *equation (E.1)* in EN 1998-5, and is inclined at an angle $\delta_k$ to the horizontal. In this particular case, where water is absent, the corresponding overall horizontal force is

$$E_{DH} = \frac{1}{2}\gamma(1 \pm k_v)K^*_{AH}H^2$$

where $H$ is the wall height. The maximum value of the overall horizontal force, when $k_v$ is taken as positive, is

$$E_{DH} = 0.5 \times 20 \times (1 + 0.0469) \times 0.434 \times 13^2 = 768 \text{ kN}$$

To obtain the horizontal component of the seismic force $E_H$ alone, the horizontal component of the static force $E_{SH}$ must be subtracted from the overall horizontal force $E_{DH}$ ($E_H = E_{DH} - E_{SH}$):

$$K^{up}_{AH}(\phi'_k, \delta_k, \beta) = 0.296 \qquad K^{up}_{PH}(\phi'_k, \delta_k, \beta) = 7.12$$

As the seismic force is assumed to act at mid-height of the wall, $E_H$ is applied to the structure as a horizontal, uniformly distributed load with intensity

$$p = E_H/H = 124/13 = 10.8 \text{ kPa}$$

It is emphasized that the backfill soil is modelled in the seismic analysis using the **static coefficients of earth pressure** computed with the **characteristic value** of the shear strength, i.e. $K^{up}_{AH}(\phi'_k, \delta_k, \beta) = 0.296$ and $K^{up}_{PH}(\phi'_k, \delta_k, \beta) = 7.12$, and, in addition, applying the static-equivalent seismic load as a **horizontal uniformly distributed load** (i.e. $p = E_H/H = 124/13 = 10.8$ kPa).

*Calculation of passive earth pressure and seismic force for the supporting soil in front of the wall*

In front of the wall $\beta = 0$, and the horizontal static earth pressure coefficients, computed with the characteristic strength parameter values, would be:

- active state, static condition: $K^{down}_{AH}(\phi'_k, \delta_k) = 0.274$
- passive state, static condition: $K^{down}_{PH}(\phi'_k, \delta_k) = 5.81$.

The static passive horizontal force at limit equilibrium state would be

$$E_{SH} = \frac{1}{2}\gamma K^{down}_{PH} d^2$$

In the presence of the seismic action, instead, the earth pressure coefficient for the passive state is given by the Mononobe–Okabe formula for passive states (*equation (E.4)* in EN 1998-5): $K^{*down}_P(\phi'_d, k_h, k_v) = 2.46$. *Equation (E.4)* in EN 1998-5 is derived under the assumption that under seismic conditions, for passive states, there is no shearing resistance between the soil and the wall, i.e. $\delta = 0$. Therefore, the coefficient computed with this equation already represents a horizontal force, and no further projection is needed to obtain the horizontal component, that is, $K^{*down}_{PH} = K^{*down}_P$.

The overall (static plus dynamic) force acting horizontally in front of the wall is therefore

$$E_{DH} = \frac{1}{2}\gamma(1 \pm k_v)K^{*down}_{PH} d^2$$

Note that in the Mononobe–Okabe approach the effect of the seismic action on the supporting soil can be thought of as a reduction in resistance, rather than as the action of an additional load. This reduction is introduced in the model assigning to the soil in front of the wall an **equivalent coefficient of passive earth pressure**, given by

$$K^{down}_{PE} = (1 - k_v)K^{*down}_{PH} = (1 - 0.0469) \times 2.46 = 2.34$$

instead of the coefficient $K^{down}_{PH}$ that would be applicable under static conditions. It should be noted that the coefficient $K^{down}_{PE}$ incorporates the effect of horizontal stress reduction due to the vertical acceleration.

The coefficient of active earth pressure has a modest influence in front of the wall, and can be conservatively assumed to be equal to the static value.

The same procedure was followed using the second choice of the design seismic action ($a_g = 0.1g$), for which a 'seismic' load intensity $p = 5.23$ kPa was obtained.

### *Results*

*First choice of design acceleration (seismic case 1, $a_g = 0.19g$)*

By using the same computational tool as in the static case, the bending moments in the pile wall are obtained under both static and seismic conditions. They are displayed in Fig. 10.17, labelled 'seismic 1'. The maximum values are:

- static case: $M_{max} = 221$ kN m/m
- seismic case 1: $M_{max} = 664$ kN m/m.

For a concrete with $f_{ck} = 35$ N/mm$^2$, the allowable moment of a single 1 m diameter pile used in the wall is 1080 kN m, which gives 720 kN m/m for the wall (given the pile spacing

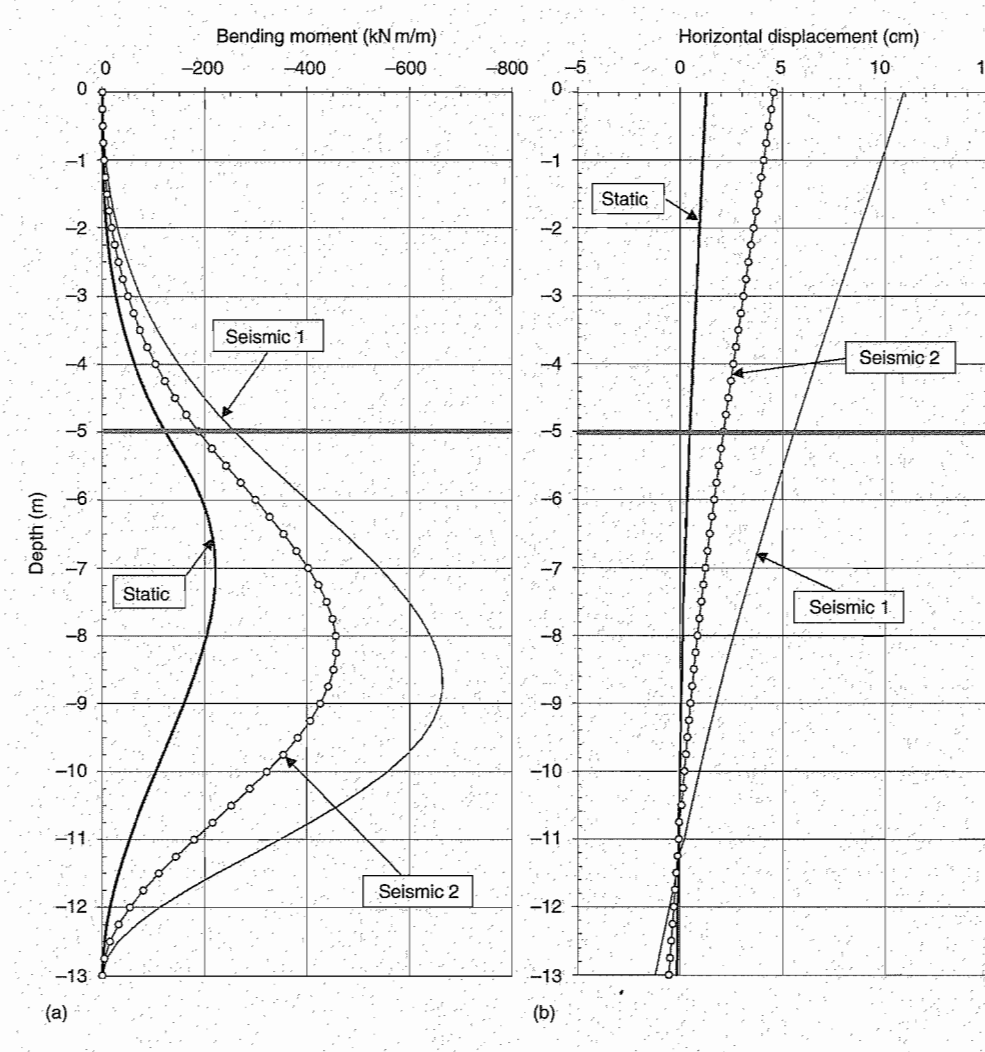



**Fig. 10.17.** (a) Bending moments and (b) horizontal displacement profiles in a pile wall under static and seismic conditions. 'Seismic 1' corresponds to the first choice of the design seismic action ($a_g = 0.19g$), while 'seismic 2' denotes the second choice ($a_g = 0.10g$)

of 1.5 m). The horizontal displacement diagrams are also illustrated in Fig. 10.17, as 'seismic 1'. The displacement values at the top of the wall are:

- static case: $U_{max} = 13$ mm
- seismic case: $U_{max,t} = 109$ mm (total value, including the static displacement).

The purely 'seismic' displacement of 96 mm is rather large, but can be considered acceptable for the serviceability requirements of the structure under consideration.

*Second choice of design acceleration (seismic case 1, $a_g = 0.10g$)*
The results for this case are:

- seismic case 2: maximum moment $M_{max} = 456$ kN m/m; maximum displacement $U_{max} = 46$ mm.

The moment reduction is 33% with respect to the previous choice, but the maximum displacement reduction is nearly 60%. In view of the indications of Example 10.8 below, the results obtained with the second choice of design acceleration are regarded as more realistic.

## Example 10.8: non-linear dynamic analysis of the flexible retaining structure of Example 10.7 subjected to earthquake excitation

### *Description of the problem and soil/structure input data*

A full dynamic analysis conducted on the same structure considered in Example 10.7 is detailed in this example. The dynamic response of the soil–structure system to earthquake excitation has been computed numerically with a non-linear finite-element code,[137] capable of simulating the structure and the surrounding soil during both the construction and the seismic excitation phases, in sequence, so as to ensure that realistic stress states are obtained from the analysis.

Nearly all the soil and structure physical and mechanical characteristics were left unchanged with respect to Example 10.7. The only differences are:

- the shear wave velocity $v_s$ in the soil is taken equal to 220 m/s, and corresponds to a Young modulus at small deformation of $E_{0s} = (1 + 2\nu)(\gamma/g)v_s^2 = 148$ MPa, in agreement with the assumption discussed in Example 10.7.
- The constitutive model of the soil was explicitly assumed to be elastic–perfectly plastic, obeying a Mohr–Coulomb (two-dimensional formulation) yield criterion.

Note that, with the (constant) $v_s$ value assumed, ground type C of EN 1998-1 applies.

### *Computational mesh and boundary conditions*

The domain of analysis illustrated in Fig. 10.18 was used, which contains 1257 nodes and 2338 triangular two-dimensional linear elements, with an average element length of about 3 m. Since an excitation with a relatively high-frequency content was chosen, a maximum frequency $f_{max} = 10$ Hz, i.e. the shortest wavelength $\lambda_{min} = v_s/f_{max} = 22$ m, was selected as a target for the wave propagation analysis. The length $\lambda_{min}$ was spatially sampled with about eight nodes, which is perfectly adequate for a homogeneous material.

The wall was modelled with elastic two-noded beam elements directly bonded to the mesh nodes, without interface elements between the structure and the soil.

The construction phase consisted of removing in two stages the soil in front of the wall, roughly modelling in this way the excavation process. During the initial, static, construction phase, zero horizontal and vertical displacements were imposed on the base of the model. In the subsequent dynamic analysis, all the boundary constraints were removed, with initial stress conditions following from the equilibrium set of nodal forces resulting from the static analysis.

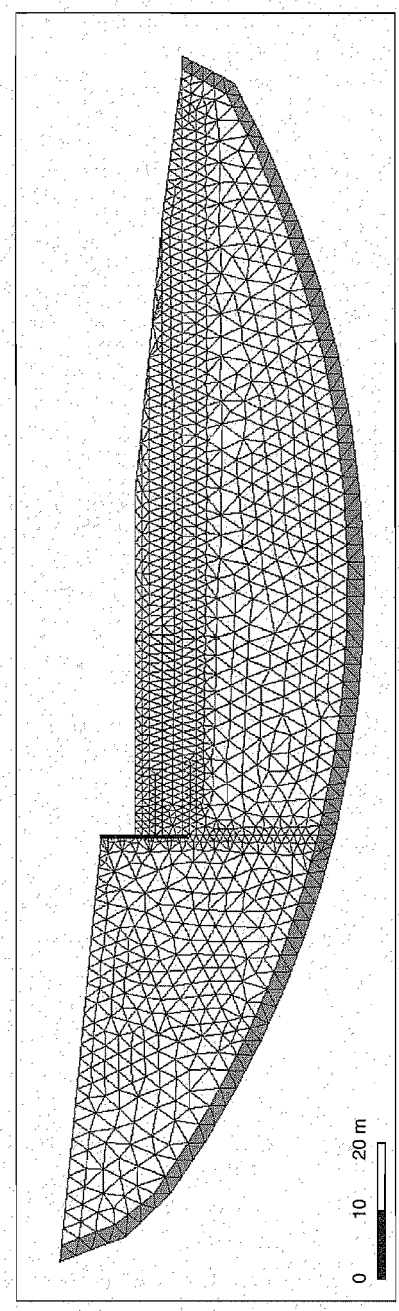



**Fig. 10.18.** The finite-element model used for both the static and dynamic analyses of the retaining structure of Fig. 10.15

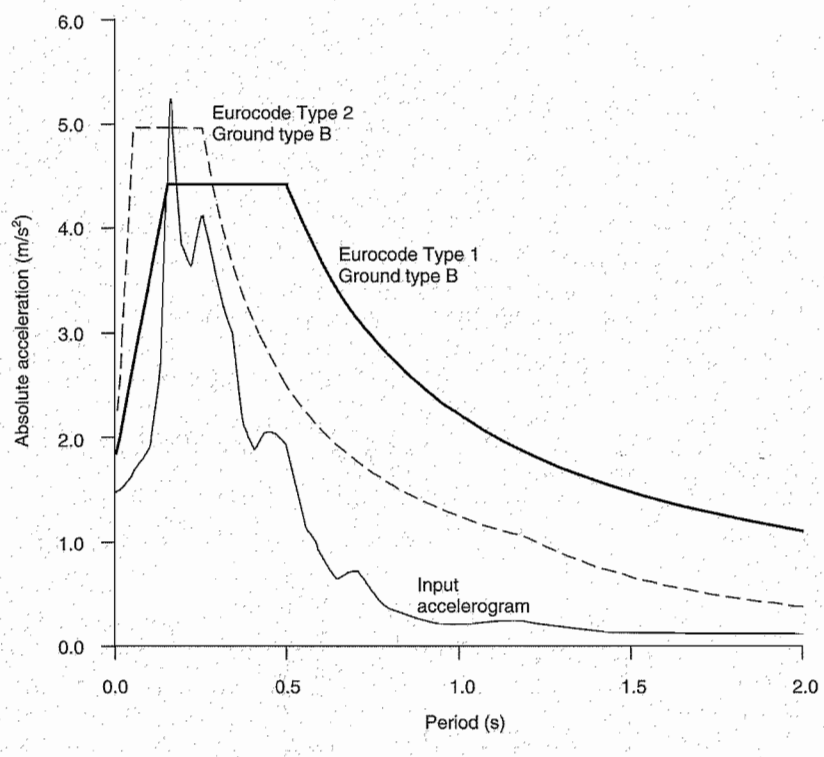



**Fig. 10.19.** A comparison of the 5% damped acceleration response spectra of the input accelerogram used in the dynamic analysis of the retaining structure of Fig. 10.15, with those of EN 1998-1

Seismic excitation was input into the model as a vertically incident plane wave of horizontal acceleration, by means of two-dimensional paraxial elements located along the lower boundary.[137] The paraxial element formulation, coupled with the curved shape of the lower boundary, ensured a nearly complete transparency of the boundary itself to waves scattered by both the topographical surface and the structure.

### *Earthquake excitation*

For excitation in the dynamic analysis a single acceleration history was selected, recorded near Tortona (in north-west Italy) in 2003, during a moderate earthquake of magnitude $M = 4.6$ with a peak ground acceleration close to $0.1g$. The record, obtained at a firm ground site (presumably type B), has been scaled in amplitude to $0.15g$, to match the $a_{gR}$ value assumed in Example 10.7 (first choice).

A comparison between the acceleration spectrum of the scaled Tortona accelerogram and the type 1 and 2 spectra of EN 1998-1 for ground type B is shown in Fig. 10.19. The accelerogram has been chosen to represent realistic ground motion in a low-to-moderate seismicity area of the north-west Apennines in Italy, rather than to closely match an elastic design spectrum.

The computational code performs a de-convolution of the excitation waveform prior to analysis, in order to calculate the correct amplitude and phase of the input motion at each of the lower boundary nodes. Thus, the peak amplitude of the motion incident at the base of the model is about $0.07g$ (as expected).

A time step $\Delta t = 0.001$ s was used to advance the solution in time, after checking that a smaller value did not lead to significant improvements.

***Results***

Given the limited severity of the excitation and the soil characteristics, the dynamic response of the soil–wall system was found to be predominantly elastic, with modest plastic deformations occurring in the vicinity of the wall. In Fig. 10.20 the bending moment and horizontal displacement diagrams for the wall are displayed. The (transient) dynamic displacements of the structure are represented by means of the envelope of maxima occurring throughout the seismic excitation. The moment diagram also represents an envelope of maxima. It is worth noting that:

- The **static** displacements are significantly smaller than those calculated in Example 10.7 (Fig. 10.17), due to the differences in modelling and in taking the excavation phase into account (the latter is simulated more accurately in the static case of Example 10.7).
- The dynamic bending moments given by the finite-element analysis are considerably smaller than the corresponding 'seismic' values obtained by the pseudo-static method, for both choices of the design seismic action: a maximum dynamic moment of about 230 kN m/m (Fig. 10.20) should be compared with pseudo-static values of

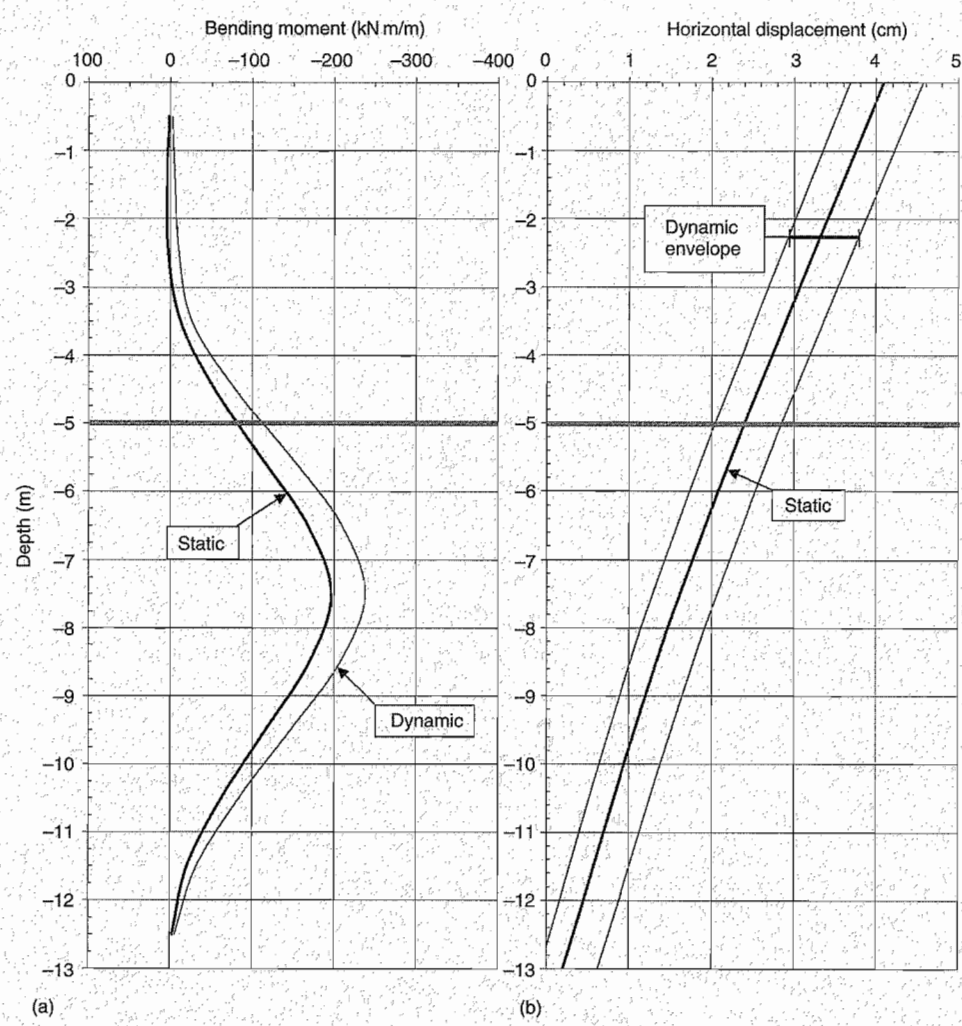



**Fig. 10.20.** (a) Bending moments and (b) horizontal displacement profiles in a pile wall obtained from the finite-element analysis under static and dynamic conditions

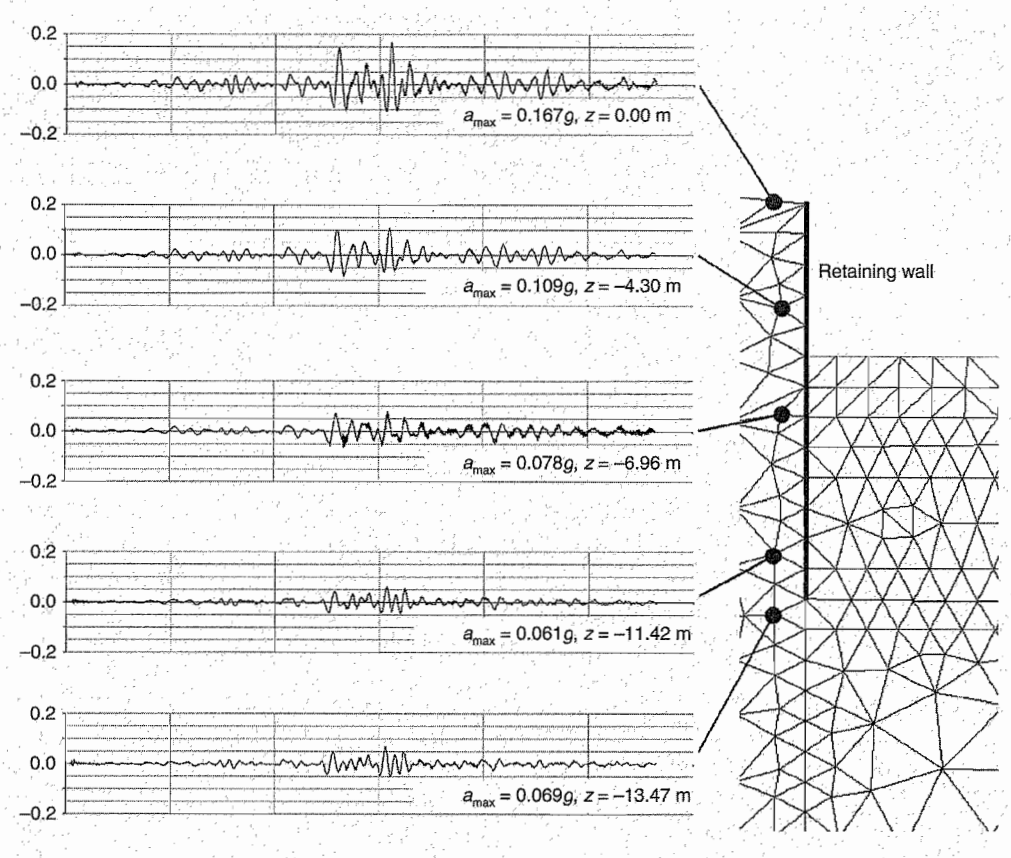



**Fig. 10.21.** Horizontal acceleration histories at the back of the pile wall, evaluated at different depths

664 kN m/m (Fig. 10.17, seismic 1) and 456 kN m/m (Fig. 10.17, seismic 2), both occurring in the embedded portion of the wall at a distance of about 7.5 m from the top.

- The dynamic displacement profile for the wall is in remarkably good agreement with the pseudo-static profile corresponding to the second choice of the design seismic action (Fig. 10.18, 'seismic 2', left part).
- While in the finite-element analysis the maximum dynamic moments exceed the static ones only by about 14% and the dynamic displacements are on average quite close to the static ones, much larger differences are given by the conventional analysis, as already discussed. In particular, Fig. 10.17 shows that the seismic-to-static displacement ratio at the top of the wall is about 10 for 'seismic 1' and over 3 for 'seismic 2'.

Thus, for the particular example considered, characterized by an essentially elastic response of the soil–wall system to the dynamic excitation, the simplified pseudo-static method yields conservative results with respect to the refined finite-element analysis, even for a horizontal seismic coefficient as small as 0.05.

Since the wall has a total height exceeding 10 m, it was of interest to check, in connection with *clause 7.3.3.2(6)* and *equation (E.2)* of EN 1998-5, the distribution of the peak horizontal soil acceleration along the depth of the profile, close to the wall. This is illustrated in Fig. 10.21, where it can be seen that:

- a variation of 260%, i.e. between 0.061*g* and 0.167*g*, occurs in the peak values
- most of the amplification takes place behind the exposed portion of the wall
- the average peak horizontal acceleration over the wall height (very close to 0.10*g*) provides the justification for the second choice of the design seismic action adopted for the pseudo-static analysis in Example 10.7.

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This series of Designers Guides to the Eurocodes provides comprehensive guidance in the form of design aids, indications for the most convenient design procedures and worked examples. The books also include background information to aid the designer in understanding the reasoning behind and the objectives of the codes. All of the individual guides work in conjunction with the *Designers' Guide to EN1990 Eurocode: Basis of Structural Design*.

All aspects of seismic design are covered in *Designers' Guide to EN 1998-1 and EN 1998-5 Eurocode 8: Design of structures for earthquake resistance. General rules, seismic actions, design rules for buildings, foundations and retaining structures*, instead of being distributed across the Eurocodes on actions (EN 1991), design with specific materials (EN 1992–1996) or geotechnical design (EN 1997). This was for the convenience of countries with very low seismicity, as they may not need to apply Eurocode 8 at all.

This present Designers' Guide covers EN1998-1 (*General Rules, seismic actions and rules for buildings*) and EN1998-5 (*Foundations, retaining structures, geotechnical aspects*), both published by CEN at the end of 2004. These two parts of Eurocode 8 will be included, as service items, in all Eurocode packages (each package referring to a specific type of civil engineering structure and construction material) along with *EN1990: Basis of Structural Design, EN1997: Geotechnical Design and the relevant parts of EN1991-1: Actions on Structures*.

The development of Eurocode 8 as the single seismic design code in Europe has been the focus of the European earthquake engineering community in the past twenty years. Its emergence as the European Standard is important not only for public safety in seismic southern regions of Europe, but also for competitiveness of the entire European consultancy and engineering services sector in seismic regions in Europe and beyond.

This guide is essential reading for:

- civil and structural engineers
- code-drafting committees
- clients
- structural-design students
- public authorities

in fact, everyone who will be affected by the Eurocodes.


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