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Eurocode 8: Design of structures for earthquake resistance

Part 3: Strengthening and repair of buildings

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Foreword

This European Standard EN 1998-3, Eurocode 8: Design of structures for earthquake resistance. Part 3: Strengthening and repair of buildings, has been prepared on behalf of Technical Committee CEN/TC250 «Structural Eurocodes», the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1998-3 on **YYYY-MM-DD**.

No existing European Standard is superseded.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990	Eurocode :	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures

[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire ;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

a) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc.;

a) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters that are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, *i.e.*:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), *e.g.* snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

- decisions on the use of informative annexes, and
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products that refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1998-3

(1) Although repair and strengthening under non-seismic actions is not yet covered by the relevant material-dependent Eurocodes, this Part of Eurocode 8 was specifically developed because:

- For most of the old structures seismic design was not considered originally, whereas the ordinary actions were considered, at least by means of traditional construction rules
- Seismic hazard evaluations in accordance with the present knowledge may indicate the need of strengthening campaigns.

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[4] see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

- The occurrence of earthquakes may create the need for important repairs.

(2) Furthermore, since within the philosophy of Eurocode 8 the aseismic design of new structures is based on a certain acceptable degree of structural damage in the event of the design earthquake, criteria for redesign (of structures designed according to Eurocode 8 and subsequently damaged) constitute an integral part of the entire process for seismic structural safety.

(3) In strengthening and repair situations, qualitative verifications for the identification and elimination of major structural defects are very important and should not be discouraged by the quantitative analytical approach proper to this Part of Eurocode 8. Preparation of documents of more qualitative nature is left to the initiative of the National Authorities.

(4) This Standard addresses the structural aspects of repair and strengthening, which is only one component of a broader strategy for seismic risk mitigation that includes pre and/or post-earthquake steps to be taken by several responsible agencies.

(5) In low seismicity zones, this Standard may be adapted to local conditions by appropriate National Annexes.

National annex for EN 1998-3

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1998-3:200X should have a National annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1998-3:200X through clauses:

Reference	Item	National Annex
Fill in	Informative Annexes A and B.	NA

1 GENERAL

1.1 Scope

(1)P The scope of Eurocode 8 is defined in 1.1.1 of EN 1998-1 and the scope of this Standard is defined in 1.1. Additional parts of Eurocode 8 that are planned are indicated in 1.1.3 of EN 1998-1.

(2) Its scope is the following:

- To provide criteria for the evaluation of the seismic performance of existing individual structures.
- To describe the approach in selecting necessary corrective measures
- To set forth criteria for the design of the repair/strengthening measures (i.e. conception, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural elements).

(3) When designing a structural intervention to provide adequate resistance against seismic actions, structural verifications shall also be made with respect to non-seismic load combinations.

(4) Reflecting the basic requirements of EN 1998-1, this Standard covers the repair and strengthening of buildings and, where applicable, monuments, considering commonly used structural materials (concrete, steel, masonry and timber).

(5) Although the provisions of this Standard are applicable to all categories of buildings, the repair or strengthening of monuments and historical buildings often requires different types of provisions and approaches, which should take in proper consideration the nature of the monuments.

(6) Since existing structures:

(i) reflect the state of knowledge of the time of their construction,

(ii) possibly contain hidden gross errors,

(iii) may have been submitted to previous earthquakes or other accidental actions with unknown effects,

structural evaluation and possible structural intervention are typically subjected to a different degree of uncertainty (level of knowledge) than the design of new structures. Different sets of material and structural safety factors are therefore required, as well as different analysis procedures, depending on the completeness and reliability of the information available.

1.2 Assumptions

(1) Reference is made to 1.2 of EN 1998-1.

(2) The provisions of this Standard assume both that the data collection and tests shall be performed by experienced personnel and that the engineer responsible for the

assessment, possible redesign and execution of work has appropriate experience of the type of structures being strengthened or repaired.

(3) Inspection procedures, check-lists and other data-collection procedures should be documented and filed, and should be referred to in the design documents.

1.3 Distinction between Principles and Application Rules

(1) The rules in EN 1990 clause 1.4 apply.

1.4 Definitions

1.4.1 Terms common to all Eurocodes

(1) The definitions of EN 1990 clause 1.5 apply.

1.4.2 Further terms used in EN 1998

(1) The definitions of EN 1998-1 clause 1.4.2 apply.

1.5 S.I. Units

(1) Reference is made to 1.5 of EN 1998-1.

1.6 Symbols

(1) Reference is made to Section 1.6 and Section 4.1.2 of EN1998-1.

(2) Further symbols used in this Standard are defined in the text where they occur.

1.7 S.I. Units

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

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

- forces and loads: kN, kN/m, kN/m^2
- unit mass: kg/m^3, t/m^3
- mass: kg, t
- unit weight: kN/m^3
- stresses and strengths: N/mm^2 (= MN/m^2 or MPa), kN/m^2 (=kPa)
- moments (bending, etc): kNm
- acceleration: m/s^2, g (=9,81 m/s^2)

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P The fundamental requirements refer to the state of damage in the structure, herein defined through the three following Limit States (LS):

- LS of near collapse (NC). The structure is heavily damaged, with small residual strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would not survive another earthquake, even of moderate intensity.
- LS of significant damage (SD). The structure is significantly damaged, with some residual strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure is likely to be uneconomic to repair.
- LS of damage limitation (DL). The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show a diffused state of cracking that could however be economically repaired. No permanent drifts are present. The structure does not need any repair measures.

(2)P National Authorities define the appropriate levels of protection required against the exceedance of the above-described Limit States. They also decide whether all three LS must be checked, or two of them, or just one of them.

(3)P The appropriate levels of protection are achieved by selecting, for each of the LS, a return period for the design seismic action. The return periods are established by the National Authorities.

Note 1: The definition of the LS of collapse given in this Part 3 of EC8 is closer to the actual collapse of the building than the one given in Part 1 of EC8 and corresponds to the fullest exploitation of the deformation capacity of structural elements. The one given in Part 1 of EC8 denoted as 'no collapse' is roughly equivalent to the one that is here defined as 'LS of significant damage'.

Note 2: Suggested values for the return periods are:
– LS of NC: 2475 years, corresponding to a probability of exceedance of 2% in 50 years
– LS of SD: 475 years, corresponding to a probability of exceedance of 10% in 50 years
– LS of DL: 225 years, corresponding to a probability of exceedance of 20% in 50 years

(4)P National Authorities may identify particular categories of structures and issue National Annexes with simplified provisions of qualitative nature, deemed to provide a sufficient improvement of their seismic resistance.

2.2 Compliance criteria

2.2.1 General

(1)P Compliance with the above requirements is achieved by adoption of the analysis, verification and detailing procedures contained in this part, as appropriate for the different structural materials (concrete, steel, masonry).

(2)P For what concerns verification of the structural elements, a distinction is made between 'ductile' and 'brittle' ones. The former shall be verified by comparing demands and capacities in terms of deformations. The latter shall be verified by checking that demands do not exceed the corresponding capacity in terms of strengths.

2.2.2 LS of NC

(1)P Demands shall be based on the design seismic action relevant to this LS. For ductile elements demands shall be evaluated based on the result of the analysis, either linear or non-linear, while for brittle elements demands may need to be modified as indicated in 4.5.

(2)P Capacities shall be based on ultimate deformations for ductile elements and on ultimate strengths for brittle ones.

2.2.3 LS of SD

(1)P Demands shall be based on the design seismic action relevant to this LS. For ductile elements demands shall be evaluated based on the result of the analysis, either linear or non-linear, while for brittle elements demands may need to be modified as indicated in 4.5.

(2)P Capacities shall be based on damage-related deformations for ductile elements and on conservatively estimated strengths for brittle ones.

2.2.4 LS of DL

(1)P Demands shall be based on the design seismic action relevant to this LS. They shall be evaluated on the basis of the analysis method, either linear or non-linear.

(2)P Capacities shall be based on yield strengths for all structural elements, both ductile and brittle, and on mean interstory drift capacity for the infills.

3 INFORMATION FOR STRUCTURAL ASSESSMENT

3.1 General information and history

(1)P In assessing the earthquake resistance of existing structures, taking also into account the effects of actions in other design situations, the input data shall be collected from available records, relevant information, field investigations and, in most cases, from in-situ and/or laboratory measurements and tests.

(2)P Cross-examination of the results of each data-source shall be performed to minimise uncertainties.

3.2 Required input data

(1) In general, the information for structural evaluation should cover the following points from a) to i).

a) Identification of the structural system and of its compliance with the regularity criteria in 4.2.3 of EN 1998-1. The information should be collected either from on site investigation or from original design drawings, if available. In this latter case, information on possible structural changes since construction should also be collected.

b) Identification of the type of building foundations.

c) Identification of the subsoil conditions as categorised in 3.1 of EN 1998-1.

d) Information about the overall dimensions and cross-sectional properties of the building elements and the mechanical properties and condition of constituent materials.

e) Information about identifiable material defects and inadequate detailing.

f) Information on the seismic design criteria used for the initial design, including the value of the force reduction factor (q-factor), if applicable.

g) Description of the present and/or the planned use of the building (with identification of its importance category, as described in 4.2.5 of EN 1998-1).

h) Re-assessment of variable loads considering the use of the building.

i) Information about the type and extent of previous and present structural damages, if any, including earlier repair measures.

Depending on the amount and quality of the information collected on the points above, different types of analysis and different values of the partial safety factors shall be adopted, as indicated in 3.3.

3.3 Knowledge levels

(1) For the purpose of choosing the admissible type of analysis and the appropriate partial safety factor values, the following three knowledge levels are defined:

KL1 : Limited knowledge

KL2 : Normal knowledge

KL3 : Full knowledge

(2) The aspects entering in the definition of the above-listed knowledge levels are:

i) *geometry*: the geometrical properties of the structural system,

ii) *details*: the amount and detailing of reinforcement (for r.c., both longitudinal and transverse), connections (for steel, either welded or bolted),

iii) *materials*: the mechanical properties of the constituent materials.

(3) The knowledge level achieved determines the allowable method of analysis (see 4.4), as well as the values to be adopted for the characteristic values of the material properties, and for the partial safety factors (PSF). The procedures for obtaining the required data are given in 3.4.

(4) The relationship between knowledge levels and applicable methods of analysis and partial safety factors is illustrated in the Table below. The definitions of the terms 'visual', 'full', 'limited', 'extended' and 'comprehensive' in the Table are given in 3.4.

Table 3.1: Knowledge levels and corresponding methods of analysis (LS: Linear Static, LD: Linear Dynamic) and partial safety factors (PSF).

KNOWLEDGE LEVEL	GEOMETRY	DETAILS	MATERIALS	ANALYSIS	PSF
KL1	From original architectural drawings with sample **visual** survey ***or*** from **full** survey	Simulated design according to relevant practice ***and*** from **limited** *in-situ* inspection	Default values according to standards of the time of construction ***and*** from **limited** *in-situ* testing	LS-LD	increased
KL2	From original architectural drawings with sample **visual** survey ***or*** from **full** survey	From incomplete original executive construction drawings with **limited** *in-situ* inspection ***or*** from **extended** *in-situ* inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	code
KL3	From original architectural drawings with sample **visual** survey ***or*** from **full** survey	From original executive construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	decreased

3.3.1 KL1: Limited knowledge

(1) The knowledge level is referred to the following three items:

i) *geometry*: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction. The information collected regards elements dimensions, beams spans and columns heights and is sufficient to build a structural model for linear analysis.

ii) *details*: the structural details are not known from original construction drawings and should be assumed based on simulated design according to usual practice of the time of construction. Limited *in-situ* inspections in the most critical elements should be performed to check that the assumptions correspond to the actual situation. The information collected should be sufficient to perform local verifications.

iii) *materials*: no direct information on the mechanical properties of the construction materials is available, neither from original design specifications nor from original test reports. In this case, default values should be assumed according to standards of the time of construction, accompanied by limited *in-situ* testing in the most critical elements.

(2) Structural evaluation based on a state of limited knowledge shall be performed through linear analysis methods, either static or dynamic (see 4.4). The relevant partial safety factors for the material properties shall be appropriately increased (see 3.5).

3.3.2 KL2: Normal knowledge

(1) The knowledge level is referred to the following three items:

i) *geometry*: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction. The information collected regards elements dimensions, beams spans and columns heights and is sufficient, together with those regarding the details, to build a structural model for either linear or nonlinear analysis.

ii) *details*: the structural details are known either from extended *in-situ* inspection or from incomplete original executive construction drawings. In the latter case, limited *in-situ* inspections in the most critical elements should be performed to check that the available information correspond to the actual situation. The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

iii) *materials*: information on the mechanical properties of the construction materials is available either from extended *in-situ* testing or from original design specifications. In this latter case, limited *in-situ* testing should be performed. The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

(2) Structural evaluation based on a state of normal knowledge shall be performed through either linear or nonlinear analysis methods, either static or dynamic (see 4.4). The relevant partial safety factors for the material properties shall be taken equal to those given in EN 1998-1 (see 3.5).

3.3.3 KL3: Full knowledge

(1) The knowledge level is referred to the following three items:

i) *geometry*: the structure's geometry is known either from survey or from original architectural drawings. In this latter case, a sample visual survey should be performed in order to check that the actual situation of the structure corresponds to the information contained in the drawings and has not changed from the time of construction. The information collected regards elements dimensions, beams spans and columns heights and is sufficient, together with those regarding the details, to build a structural model for both linear and nonlinear analysis.

ii) *details*: the structural details are known either from comprehensive *in-situ* inspection or from original executive construction drawings. In the latter case, limited *in-situ* inspections in the most critical elements should be performed to check that the available information correspond to the actual situation. The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

iii) *materials*: information on the mechanical properties of the construction materials is available either from comprehensive *in-situ* testing or from original test reports. In this latter case, limited *in-situ* testing should be performed. The information collected should be sufficient for either performing local verifications or setting up a nonlinear structural model.

(2) Structural evaluation based on a state of full knowledge shall be performed through either linear or nonlinear analysis methods, either static or dynamic (see 4.4). The relevant partial safety factors for the material properties shall be appropriately decreased (see 3.5).

3.4 Identification of the Knowledge Level

3.4.1 Geometry

3.4.1.1 Original Architectural Drawings

(1) The original architectural drawings are those documents that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions.

3.4.1.2 Original Executive Construction Drawings

(1) The original executive drawings are those documents that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions. In addition, information about details (as specified in 3.3) is contained.

3.4.1.3 Visual Survey

(1) A visual survey is a procedure for checking correspondence between the actual geometry of the structure with the available original architectural drawings. Sample geometry measurements on selected elements should be carried out. Possible structural changes occurred during or after construction should be object of a survey as in 3.4.1.4.

3.4.1.4 Full Survey

(1) A full survey is a procedure resulting in the production of architectural drawings that describe the geometry of the structure, allowing for identification of structural components and their dimensions, as well as the structural system to resist both vertical and lateral actions.

3.4.2 Details

(1) Reliable non-destructive methods can be adopted in the inspections specified below.

3.4.2.1 Simulated Design

(1) A simulated design is a procedure resulting in the definition of the amount and layout of reinforcement, both longitudinal and transverse, in all members participating in the vertical and lateral resistance of the building. The design should be carried out based on regulatory documents and state of the practice used at the time of construction.

3.4.2.2 Limited In-situ Inspection

(1) A limited *in-situ* inspection is a procedure for checking correspondence between the actual details of the structure with either the available original executive construction drawings or the results of the simulated design in 3.4.2.1. This involves performing inspections as indicated in Table 3.2.

3.4.2.3 Extended In-situ Inspection

(1) An extended *in-situ* inspection is a procedure used when the original executive construction drawings are not available. This involves performing inspections as indicated in Table 3.2.

3.4.2.4 Comprehensive In-situ Inspection

(1) A comprehensive *in-situ* inspection is a procedure used when the original executive construction drawings are not available and when a higher knowledge level is sought. This involves performing inspections as indicated in Table 3.2.

3.4.3 Materials

(1) Non-destructive test methods cannot be used in place of test methods on material samples extracted from the structure.

3.4.3.1 Limited In-situ Testing

(1) A limited *in-situ* testing is a procedure for complementing the information on material properties derived either from the standards of the time of construction, or from original design specifications, or from original test reports. This involves performing tests as indicated in Table 3.2. However, if values from tests are lower than default values according to standards of the time of construction, an extended *in-situ* testing is required.

3.4.3.2 Extended In-situ Testing

(1) An extended *in-situ* testing is a procedure for obtaining information when both original design specification and test reports are not available. This involves performing tests as indicated in Table 3.2.

3.4.3.3 Comprehensive In-situ Testing

(1) A comprehensive *in-situ* testing is a procedure for obtaining information when both original design specification and test reports are not available and when a higher knowledge level is sought. This involves performing tests as indicated in Table 3.2.

Table 3.2: Definition of levels of inspection and testing.

	Inspection (of Details)	Testing (of Materials)
	For each type of primary member (beam, column, wall):	
Limited	$\boxed{20\%}$ of the members are checked for details	$\boxed{1}$ material sample per floor
Extended	$\boxed{50\%}$ of the members are checked for details	$\boxed{2}$ material samples per floor
Comprehensive	$\boxed{80\%}$ of the members are checked for details	$\boxed{3}$ material samples per floor

3.5 Partial Safety Factors

(1) Based on the knowledge level achieved through the different levels of survey, inspection and testing, the following set of partial safety factors (PSF) as shown in Table 3.3 are used in the verifications.

Table 3.3: Partial safety factors (PSF) used for verifications, according to the different knowledge levels (KL).

	PSF	
KNOWLEDGE LEVEL	Material	Overstrength
KL1	$\boxed{1.20}\,\gamma_m$	$\boxed{1.20}\,\gamma_{Rd}, \gamma_{ov}$
KL2	γ_m as in EN1998-1	γ_{Rd}, γ_{ov} as in EN1998-1
KL3	$\boxed{0.80}\,\gamma_m$	$\boxed{0.80}\,\gamma_{Rd}, \gamma_{ov}$

4 ASSESSMENT

4.1 General

(1) Assessment is a quantitative procedure by which it is checked whether an existing undamaged or damaged building can resist the design seismic load combination as specified in this code.

(2)P Within the scope of this Standard, assessment is made for individual buildings, in order to decide about the need for structural intervention and about the strengthening or repair measures to be implemented.

(3)P The assessment procedure shall be carried out by means of the general analysis methods foreseen in EN 1998-1 (4.3), as modified in this standard to suit the specific problems encountered in the assessment.

(4) Whenever possible, the method used should incorporate information of the observed behaviour of the same type of building or similar buildings during previous earthquakes.

4.2 Seismic action and seismic load combination

(1)P The basic models for the definition of the seismic motion are those presented in 3.2.2 and 3.2.3 of EN 1998-1.

(2)P Reference is made in particular to the elastic response spectrum given in 3.2.2.2 of EN 1998-1, scaled with the values of the design ground acceleration established for the verification of the different Limit States. The alternative representations given in 3.2.3 of EN 1998-1 in terms of either artificial or recorded accelerograms are also applicable.

(3)P The design seismic action shall be combined with the other appropriate permanent and variable actions in accordance with the rule given in 3.2.4 of EN 1998-1.

4.3 Structural modelling

(1)P Based on the information collected as indicated in 3.2, a model of the structure shall be set up. The model shall be adequate for determining the action effects in all structural elements under the seismic load combination given in 4.2.

(2)P All provisions of EN 1998-1 regarding modelling (4.3.1) and accidental torsional effects (4.3.2) apply without modifications.

(3)P Some of the existing structural members can be designated as "secondary", in accordance to the definitions given in 4.2.2 of EN 1998-1, items (1)P, (2) and (3). The strength and the stiffness of these members against lateral actions shall be neglected, but they shall be checked to maintain their integrity and capacity of supporting gravity loads when subjected to the design displacements, with due allowance for 2nd order effects. Deemed to satisfy rules for the design and detailing of secondary seismic elements are contained in section 5 to 9 of EN 1998-1. The choice of the members to be considered as secondary can be varied after the results of a preliminary analysis, but in no case the selection of these elements shall be such as to change the classification of the structure from non regular to regular, according to the definitions given in 4.2.3 of EN 1998-1.

4.4 Methods of analysis

4.4.1 General

(1) The seismic action effects, to be combined with the effects of the other permanent and variable loads according to the seismic combination in 4.2, may be evaluated using one of the following methods:

- lateral force analysis,
- multi-modal response spectrum analysis,
- non-linear static analysis,
- non-linear time history dynamic analyses.

(2) In all cases, the seismic action to be used is the one corresponding to the elastic (*i.e.*, un-reduced by the behaviour factor q) response spectrum in 3.2.2.2 of EN 1998-1, or its equivalent alternative representations given in 3.2.3 of EN 1998-1, respectively, factored by the appropriate importance factor γ_I (see 4.2.5 of EN 1998-1).

(3) Non-linear analyses shall be properly substantiated with respect to the definitions of the seismic input, to the structural model adopted, to the criteria for the interpretation of the results of the analysis, and to the requirements to be met.

(4) The above-listed methods of analysis are applicable subject to the conditions specified in 4.4.2-4.4.5, with the exception of masonry structures for which appropriate procedures accounting for the peculiarities of this construction typology need to be used. Information on these procedures may be found in the relevant material-related Annex.

4.4.2 Lateral force analysis

(1)P The conditions for this method to be applicable are given in 4.3.3.2.1 of EN 1998-1, with the addition of the following:

- denoting by $\rho_i = D_i/C_i$ the ratio between the bending moment demand D_i obtained from the analysis under the seismic load combination, and the corresponding capacity C_i for the i-th primary element of the structure ($\rho_i \nless 1$), and by ρ_{max} and ρ_{min} the maximum and minimum values of ρ_i, respectively, over all primary elements of the structure, the ratio ρ_{max}/ρ_{min} does not exceed the value of $\boxed{2}$,
- ρ_{max} does not exceed the value of $\boxed{15}$ for beams and $\boxed{7}$ for columns,
- furthermore, the capacity C_i of the "brittle" components is larger than the corresponding demand D_i, this latter evaluated either from the strength of the adjoining ductile components, if their ρ_i is larger than 1, or from the analysis, if their ρ_i is lower than 1.

(2)P The method shall be applied as described in 4.3.3.2.2/3/4 of EN 1998-1, except that the response spectrum in expression (4.3) shall be the elastic spectrum $S_e(T_1)$ instead of the design spectrum $S_d(T_1)$.

4.4.3 Multi-modal response spectrum analysis

(1)P The conditions of applicability for this method are given in 4.3.3.3.1 of EN 1998-1 with the addition of the conditions specified in 4.4.2.

(2)P The method shall be applied as described in 4.3.3.3.2/3 of EN 1998-1, using the elastic response spectrum $S_e(T_1)$.

4.4.4 Nonlinear static analysis

(1)P Nonlinear static (pushover) analysis is a non-linear static analysis under constant gravity loads and monotonically increasing horizontal loads.

(2)P Buildings not complying with the criteria of 4.2.3.2 of EN 1998-1 for regularity in plan shall be analysed using a spatial model.

(3)P For buildings complying with the regularity criteria of 4.2.3.2 of EN 1998-1 the analysis may be performed using two planar models, one for each main direction.

4.4.4.1 Lateral loads

(1) At least two vertical distributions of lateral loads should be applied:

- a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration)
- a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis

(2) Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity should be considered.

4.4.4.2 Capacity curve

(1) The relation between base-shear force and the control displacement (the "capacity curve") should be determined by pushover analysis for control displacement ranging between zero and the value corresponding to 150% of the target displacement, defined 4.4.4.3.

(2) The control displacement may be taken at the center of mass at the roof of the building. The top of a penthouse should be considered as the roof.

4.4.4.3 Target displacement

(1)P Target displacement is defined as seismic demand in terms of the displacement of an equivalent single-degree-of-freedom system in the seismic design situation.

(2) Target displacement may be determined according to the procedure given in Annex B of EN 1998-1.

4.4.4.4 Procedure for estimation of the torsional effects

(1)P Pushover analysis may significantly underestimate deformations at the stiff-strong edges of a torsionally flexible structure, i.e. a structure with first mode predominately torsional. The same applied for the stiff/strong edge deformations in one direction of a structure with second mode predominately torsional. For such structures, displacements at the stiff/strong edge should be increased, compared to those in the corresponding torsionally balanced structure.

(2) The requirement above is deemed to be satisfied if the amplification factor to be applied is based on results of elastic multi-modal analysis of the spatial model.

(3) If two planar models are used for analysis of structures regular in plan, the torsional effects may be approximately estimated according to 4.4.3.2.4 and 4.4.3.3.3 of EN 1998-1.

4.4.5 Nonlinear time-history analysis

(1) The time dependent response of the structure can be obtained through direct numerical integration of its differential equations of motion, using the accelerograms, defined in 3.2.3.1 of EN 1998-1 to represent the ground motions.

(2) The mechanical model shall be able to describe the behaviour of the elements under post-elastic unloading and reloading cycles. The model should also reflect realistically the energy dissipation in the elements over the range of displacement amplitudes expected in the seismic design situation.

4.4.6 Combination of the components of the seismic action

(1)P The two horizontal components of the seismic action shall be combined according to 4.4.3.5.1 of EN 1998-1.

(2)P The vertical component of the seismic action shall be considered in the cases contemplated in 4.4.3.5.2 of EN 1998-1 and, when appropriate, combined with the horizontal components as indicated in the same clause.

4.4.7 Additional measures for masonry infilled structures

(1) Provisions of 4.4.6 of EN 1998-1 apply, whenever relevant

4.4.8 Combination coefficients for variable actions

(1) Provisions of 4.4.7 of EN 1998-1 apply

4.4.9 Importance categories and importance factors

(1) Provisions of 4.4.8 of EN 1998-1 apply.

4.5 Safety verifications

4.5.1 Linear methods of analysis (static or dynamic)

(1)P A distinction shall be made between "ductile" and "brittle" components or mechanisms. Information for classifying components/mechanisms as "ductile" or "brittle" may be found in the relevant material-related Annexes.

(2)P "Ductile" components/mechanisms shall be considered as implicitly verified, due to the fact that the maximum ratio between acting and resisting bending moment does not exceed the value of $\boxed{15}$ for beams and $\boxed{7}$ for columns, as required by 4.4.2 and 4.4.3.

(3)P "Brittle" components/mechanisms shall be verified with two alternative demands D: either the value obtained from the analysis, if the ductile components with capacity C, delivering load to them, satisfy $D/C \leq 1$, or the value obtained by means of equilibrium conditions, considering the strength of the ductile components delivering load to the brittle component under consideration. In the latter case, the strength of the ductile components shall be multiplied, according to the principle of capacity design, by the appropriate γ_{Rd} factor as in EN 1998-1 (section 5.5 for r.c., 6.4 to 6.8 for steel and for steel-concrete composite), modified in accordance with 3.3. The demand obtained as above shall be checked to be lower than the corresponding capacity of the brittle component, evaluated using the same procedures as in EN 1998-1, with appropriate γ_m factors as in EN 1998-1, modified in accordance with 3.3.

4.5.2 Nonlinear methods of analysis (static or dynamic)

(1)P The seismic action effects on both "ductile" and "brittle" components shall be those obtained from the analysis performed according to 3.5.4 or 3.5.5.

(2)P "Ductile" components shall be checked to possess a deformation capacity not less than the maximum calculated deformations.

(3) The capacities of "ductile" components may be determined according to the rules or to the default values provided in the relevant material-related Annexes.

(3)P "Brittle" components shall be checked to have strengths not less than the maximum calculated action effects. The strength of the components shall be evaluated using the same procedures as in EN 1998-1, with appropriate γ_m factors as in EN 1998-1, modified in accordance with 3.3.

5 DECISIONS FOR STRUCTURAL INTERVENTION

5.1 Criteria for a structural intervention

(1) On the basis of the conclusions of the assessment of the structure and/or the nature and extent of the damage, decisions should be taken, seeking to minimise the cost of intervention and to optimise social interests.

(2) This Standard describes the technical aspects of the relevant criteria.

5.1.1 Technical criteria

(1)P The selection of the type, technique, extent and urgency of the intervention shall be based on the structural information collected during the assessment of the building.

(2) The following aspects should be considered:

a) All identified local gross errors should be appropriately remedied.

b) In case of highly irregular buildings (both in terms of stiffness and overstrength distributions), their structural regularity should be improved as much as possible, both in elevation and in plan.

c) The required characteristics of regularity and resistance can be achieved by either direct strengthening of a (reduced) number of deficient components, or by the insertion of new lateral load-resisting elements.

d) The increase of local ductility should be sought where needed.

e) The increase in strength after the intervention should not reduce the necessary global available ductility.

f) Specifically for masonry structures: non-ductile lintels should be replaced, inadequate connections between floor and walls should be improved, horizontal thrusts against walls should be eliminated.

5.1.2 Type of intervention

(1) An intervention may be selected from the following indicative types; one or more types in combination may be selected. In all cases, the effect of structural modifications on the foundation shall be considered.

a) Local or overall modification of damaged or undamaged elements (repair, strengthening or full replacement), considering their stiffness, strength and/or ductility.

b) Addition of new structural elements (e.g. bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc).

c) Modification of the structural system (elimination of some structural joints; widening of joints; elimination of vulnerable elements; modification into more regular and/or more ductile arrangements)[5].

d) Addition of a new structural system to sustain the entire seismic action.

e) Possible transformation of existing non-structural elements into structural elements.

f) Introduction of passive protection devices through either dissipative bracing or base isolation.

g) Mass reduction.

h) Restriction or change of use of the building.

i) Partial or total demolition.

5.1.3 Non-structural elements

1(P) Decisions regarding repair or strengthening of non-structural elements shall also be taken whenever, in addition to functional requirements, the seismic behaviour of these elements may endanger the life of inhabitants or affect the value of goods stored in the building.

(2) In such cases, full or partial collapse of these elements should be avoided by means of:

a) Appropriate connections to structural elements (see 3.5 of EN1998-1).

b) Increasing the resistance of non-structural elements (see 3.5 of EN 1998-1).

c) Taking measures of anchorage to prevent possible falling out of parts of these elements.

(3) The possible consequences of these provisions on the behaviour of structural elements should be taken into account.

5.1.4 Justification of the selected intervention type

(1)P In all cases, the redesign documents shall include the justification of the type of intervention selected and the description of its expected structural function and consequences.

(2) This justification should be made available to the person or organisation responsible for the long-term maintenance of the structure.

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[5] This is for instance the case when vulnerable low shear-ratio columns or entire soft storeys are transformed into more ductile arrangements; similarly, when overstrength irregularities in elevation, or in-plan eccentricities are reduced by modifying the structural system.

6 DESIGN OF STRUCTURAL INTERVENTION

6.1 Redesign Procedure

(1)P The redesign process shall cover the following steps:

a) Conceptual design

(i) Selection of techniques and/or materials, as well as of the type and configuration of the intervention.

(ii) Preliminary estimation of dimensions of additional structural parts

(iii) Preliminary estimation of the modified stiffness of the repaired/strengthened elements.

b) Analysis

(2) The methods of analysis of the structure as redesigned shall be those indicated in 4.4, as appropriate, considering the new characteristics of the building.

c) Verifications

(3) Safety verifications shall be carried out in accordance with 4.5.

(4) For new and old components, material safety factors γ_m shall be the same as in EN 1998-1, according to the level of knowledge specified in 3.3.

ANNEX A (Informative)

A REINFORCED CONCRETE STRUCTURES

A.1 Scope

(1) This section contains specific information for the assessment of reinforced concrete buildings in their present state, and for their upgrading, when necessary.

A.2 Identification of geometry, details and materials

A.2.1 General

(1) The following aspects should be carefully examined:

i. Physical condition of reinforced concrete elements and presence of any degradation, due to carbonation, steel corrosion, etc.

ii. Continuity of load paths between lateral resisting members.

A.2.2 Geometry

(1) The collected data should include the following items:

i. Identification of the lateral resisting systems in both directions.

ii. Orientation of one-way floor slabs.

iii. Depth and width of beams, columns and walls.

iv. Width of flanges in T-beams.

v. Possible eccentricities between beams and columns axes at joints.

A.2.3 Details

(1) The collected data should include the following items:

i. Amount of longitudinal steel in beams, columns and walls.

ii. Amount and proper detailing of confining steel in critical regions and in beam-column joints.

iii. Amount of steel reinforcement in floor slabs contributing to the negative resisting bending moment of T-beams.

iv. Seating lengths and support conditions of horizontal elements.

v. Depth of concrete cover.

vi. Lap-splices for longitudinal reinforcement.

A.2.4 Materials

(1) The collected data should include the following items:

i. Concrete strength.

ii. Steel yield strength, hardening ratio and ultimate strength.

A.3 Capacity Models for Assessment

(1) Classification of components/mechanisms:

i. "ductile": beam-columns under flexure with and without axial force, and walls,

ii. "brittle": shear mechanism of beam-columns and of joints.

A.3.1 Beam-columns under flexure with and without axial force and walls

(1) The deformation capacity of beam-columns is defined as the chord rotation θ, *i.e.*, the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span (L_V = M/V = moment/shear), *i.e.*, the point of contraflexure. The chord rotation is also equal to the member drift ratio, *i.e.*, the deflection at the end of the shear span divided by the length.

A.3.1.1 LS of near collapse (NC)

(1) The chord rotation at ultimate θ_u can be evaluated as:

$$\theta_u = a_{st}(1-0.38a_{cyc})(1+\frac{a_{sl}}{1.7})(1-0.37a_{wall})\cdot$$
$$\cdot(0.3^{\nu})\left[\frac{\max(0.01,\omega')}{\max(0.01,\omega)}f_c\right]^{0.2}(\frac{L_V}{h})^{0.425}\,25^{(\alpha\rho_{sx}\frac{f_{yw}}{f_c})}(1.45^{100\rho_d}) \qquad \text{(A.1)}$$

where a_{st} = equal to 0.016 for ductile hot-rolled steel and for Tempcore steel, or to 0.0105 for brittle cold-worked steel, a_{cyc} = equal to 0 for monotonic and to 1 for cyclic for at least one full cycle at peak deformation, a_{sl} = equal to 1 if there is slippage of the longitudinal bars from their anchorage beyond the member end, or to 0 if there is not, a_{wall} = equal to 1 for shear walls, or to 0 for beams or columns, ν = normalized axial force, ω and ω' = mechanical reinforcement ratio of the tension and compression, respectively, longitudinal reinforcement (in walls the entire vertical web reinforcement is included in the tension steel), f_c is the estimated value of the concrete compressive strength, $\rho_{sx} = A_{sx}/b_w s_h$ = ratio of transverse steel parallel to the direction x of loading

(s_h = stirrup spacing), ρ_d = steel ratio of diagonal reinforcement (if any), in each diagonal direction, α = confinement effectiveness factor, equal to:

$$\alpha = \left(1 - \frac{s_h}{2b_c}\right)\left(1 - \frac{s_h}{2h_c}\right)\left(1 - \frac{\sum b_i^2}{6h_c b_c}\right) \qquad \text{(A.2)}$$

(b_c and h_c = dimension of concrete core, b_i = distances of restrained longitudinal bars on the perimeter).

(2) An alternative equation may be used:

$$\theta_u = \theta_y + (\phi_u - \phi_y)L_{pl}\left(1 - \frac{0.5L_{pl}}{L_V}\right) \qquad \text{(A.3)}$$

where θ_y is the chord rotation at yield as defined in (A.5), ϕ_u is the ultimate curvature computed considering the compressive concrete strain at its ultimate value ε_{cu}, ϕ_y is the yield curvature computed considering the tensile steel strain at its yield value ε_{sy}, L_{pl} is the plastic hinge length given by:

$$L_{pl} = 0.08L_V + \frac{1}{60}\alpha_{sl} d_b f_y \qquad \text{(A.4)}$$

where α_{sl} is equal to 1 if there is slippage of the longitudinal bars from their anchorage beyond the member end, or to 0 if there is not, d_b is the diameter of longitudinal bars, f_y is the estimated value of the steel yield strength.

A.3.1.2 LS of severe damage (SD)

(1) The chord rotation relative to severe damage θ_{SD} can be assumed as 3/4 of the ultimate chord rotation θ_u given in (A.1) or (A.3).

A.3.1.3 LS of damage limitation (DL)

(1) The chord rotation at yielding θ_y can be evaluated as:

$$\theta_y = \phi_y \frac{L_V}{3} + 0.0025 + \alpha_{sl} \frac{0.25\,\varepsilon_{sy} d_b f_y}{(d - d')\sqrt{f_c}} \qquad \text{(A.5)}$$

where the first two terms account for flexural and shear contributions, respectively, and the third for anchorage slip of bars. In the above equation, d and d' are the depth to the tension and compression reinforcement, respectively, and f_y and f_c are the estimated values of the steel tensile and concrete compressive strength, respectively.

A.3.2 Beam-columns: shear

A.3.2.1 LS of near collapse (NC)

(1) The shear resistance is computed according to EN 1998-1, paragraph 5.5.3.1.2 for beams and paragraph 5.5.3.2.1 for columns.

A.3.2.2 LS of severe damage (SD) and of damage limitation (DL)

(1) The verification against the exceedance of these two LS is not required.

A.3.3 Beam-column joints

A.3.3.1 LS of near collapse (NC)

(1) The shear demand on the joints is evaluated according to EN 1998-1, paragraph 5.5.2.3.

(2) The shear capacity on the joints is evaluated according to EN 1998-1, paragraph 5.5.3.3.

A.3.3.2 LS of severe damage (SD) and of damage limitation (DL)

(1) The verification against the exceedance of these two LS is not required.

A.4 Capacity Models for Strengthening

A.4.1 Concrete jacketing

(1) Concrete jackets are applied to columns and walls for all or some of the following purposes: increasing the bearing capacity, increasing the flexural and/or shear strength, increasing the deformation capacity, improving the strength of deficient lap-splices.

(2) The thickness of the jackets should be such as to allow for placement of both longitudinal and transverse reinforcement with an adequate cover.

(3) If the jacket does not surround completely the old column, the reinforcement of the non-jacketed faces will have to be exposed, and the ties of the jackets may be welded to the old ties, or bent around the old vertical bars.

(4) When jackets aim at increasing flexural strength, longitudinal bars should be continued to the adjacent story through holes piercing the slab, while horizontal ties should be placed in the joint region through horizontal holes drilled in the beams. Ties can be omitted in the case of fully confined interior joints.

(5) When only shear strength and deformation capacity increases are of concern, jointly with a possible improvement of lap-splicing, then jackets will be terminated (both concreting and reinforcement) leaving a gap with the slab of the order of 10 mm.

A.4.1.1 Enhancement of strength and deformation capacities

(1) For the purpose of evaluating strength and deformation capacities of jacketed members, the following approximate simplifying assumptions may be made:

- the jacketed member behaves monolithically, with full composite action between old and new concrete;
- the fact that axial load is originally applied to the old column alone is disregarded, and the full axial load is assumed to act on the jacketed member;
- the concrete properties of the jacket are considered to apply over the full section of the member.

(2) The following relations may be assumed to hold between the values of V_R, M_y, θ_y, and θ_u, calculated under the assumption above and the values to be adopted in design:

$$V_R^* = 0.9 V_R \tag{A.6}$$

$$M_y^* = 0.9 M_y \tag{A.7}$$

$$\theta_y^* = 0.9 \theta_y \tag{A.8}$$

$$\theta_u^* = 1.0 \theta_u \tag{A.9}$$

A.4.2 Steel jacketing

(1) Steel jackets are applied to columns with the purpose of: increasing shear strength, improve the strength of deficient lap-splices, and increase ductility through confinement.

(2) Steel jackets around rectangular columns are usually made up of four corner angles to which either continuous steel plates, or thicker discrete horizontal steel straps, are welded. Corner angles may be epoxy-bonded to the concrete, or just made to adhere to it without gaps along the entire height. Straps may be pre-heated just prior to welding, in order to provide afterwards some positive confinement on the column.

A.4.2.1 Shear strength

(1) The contribution of the jacket to shear strength may be considered as additive to existing strength, provided the jacket remains well within the elastic range. This condition is necessary for the jacket to be able to control the width of internal cracks and to ensure the integrity of the concrete, thus allowing the original shear resisting mechanism to continue to operate.

(2) If only 50% of the steel yield strength of the jacket is used, the expression for the additional shear V_j carried by the jacket is:

$$V_j = 0.5 \frac{2 t_j b}{s} f_{yw} \frac{1}{\cos \alpha} \qquad \text{(A.10)}$$

where t_j, b, s are thickness, width and spacing of the steel straps, respectively, ($b/s = 1$, in case of continuous steel plates).

A.4.2.2 Confinement action

(1) The confining effect of a steel jacket may be evaluated in the same way as for hoops and ties, using for the geometric steel ratio in each transverse direction, the cross-sectional area of steel relative to a vertical section through the column.

(2) For the properties of confined concrete, the expression provided in 3.1.9 of ENV1992-1 may be used.

(3) Alternatively, the strength of confined concrete may be evaluated from:

$$f_{cc} = f_{cd}\left[1 + 3.7\left(\frac{0.5\,\alpha\,\rho_s\, f_{yw}}{f_{cd}}\right)^{0.87}\right] \qquad \text{(A.11)}$$

where ρ_s and f_{yw} are the geometric steel ratio and yield strength of the jacketing steel, respectively, and α is the so-called efficiency factor given by the ratio of the confined (shaded) concrete area to the total area in Figure 1.

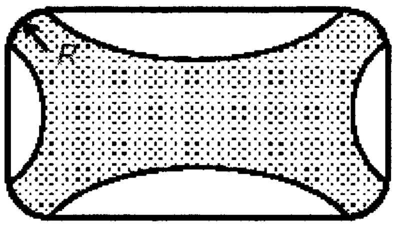

Figure 1. Effectively confined area.

(4) The ultimate deformation of concrete corresponding to (A.11) is given by:

$$\varepsilon_{cc} = 0.004 + 0.6\,\varepsilon_{su}\,\rho_s \frac{f_{yw}}{f_{cc}} \qquad \text{(A.12)}$$

A.4.2.3 Clamping of lap-splices

(1) Steel jackets can provide effective clamping in the regions of lap-splices, so as to achieve high cyclic deformation capacity. For this result to be obtained it is necessary that:

- the length of the jacket exceeds by no less than 50% the length of the splice region,
- the jacket is pressured against the faces of the column by at least two rows of bolts on each side normal to the direction of loading,

- when splicing occurs at the base of the column, the rows of bolts should be located one at the top of the splice region and another at 1/3 of that region, starting from the base.

A.4.3 FRP plating and wrapping

(1) Externally bonded FRP have been used extensively in retrofitting reinforced concrete structures, though mostly in non-seismic cases. The main uses of FRP in seismic strengthening of existing reinforced concrete elements are the following:

- Enhancement of the shear capacity of columns and walls, by applying externally bonded FRP with the fibers in the hoop direction;
- Enhancement of the available ductility at beam or column ends, through added confinement in the form of FRP jackets, with the fibers placed along the perimeter;
- Prevention of lap splicing, through increased lap confinement again with the fibers along the perimeter.

A.4.3.1 Shear strength

(1) Shear capacity of brittle components can be enhanced in beams, columns or shear walls through the application of FRP sheets. These can be applied either by fully wrapping the member, or by bonding them to the sides and the soffit of the beam (U-shaped sheet), or by bonding them to the sides only.

(2) The shear capacity is evaluated as the sum of three contributions, of concrete, of steel transverse reinforcement, and of FRP:

$$V_R = V_c + V_w + V_f \tag{A.13}$$

where V_c and V_w, the concrete and steel contributions, respectively, are evaluated according to EN 1992-1.

(3) For rectangular sections, the FRP contribution is evaluated as:

$$V_f = 0.9\ d \cdot b_w \cdot \rho_f \cdot E_f \cdot \varepsilon_{f,e} \cdot (1 + \cot\beta) \cdot \sin\beta \tag{A.14}$$

where d is the section depth, b_w is the minimum width of cross section over the effective depth, $\rho_f = 2t_f \sin\beta / b_w$ is the FRP reinforcement ratio, E_f is the FRP elastic modulus in the principal fiber orientation, β is the angle between principal fiber orientation and longitudinal axis of member, and $\varepsilon_{f,e} \leq 0.006$ is the effective strain, defined as:

- For fully wrapped (i.e., closed) or properly anchored (in the compression zone) CFRP (carbon fiber) jackets:

$$\varepsilon_{f,e} = 0.17 \cdot \left(\frac{f_c^{2/3}}{E_f \rho_f} \right)^{0.30} \varepsilon_{fu} \tag{A.15}$$

- For side or U-shaped (i.e., open) CFRP jackets:

$$\varepsilon_{f,e} = \min\left[0.65\cdot 10^{-3}\cdot\left(\frac{f_c^{2/3}}{E_f\,\rho_f}\right)^{0.56}\;;\;0.17\cdot\left(\frac{f_c^{2/3}}{E_f\,\rho_f}\right)^{0.30}\varepsilon_{fu}\right] \tag{A.16}$$

- For fully wrapped (i.e., closed) or properly anchored (in the compression zone) AFRP (aramid fiber) jackets:

$$\varepsilon_{f,e} = 0.048\cdot\left(\frac{f_c^{2/3}}{E_f\,\rho_f}\right)^{0.47}\varepsilon_{fu} \tag{A.17}$$

where f_c is the estimated value of the concrete compressive strength, and ε_{fu} is the ultimate strain of FRP. Note that f_c and E_f should be expressed in MPa and GPa, respectively.

(4) For circular sections, the FRP contribution is evaluated as:

$$V_f = 0.5\ A_c\cdot\rho_f\cdot E_f\cdot\varepsilon_{f,e} \tag{A.18}$$

where A_c is the column cross-section area, and $\varepsilon_{f,e} = 0.004$.

A.4.3.2 Confinement action

(1) The enhancement of deformation capacity is achieved through concrete confinement by means of FRP jackets. These are applied around the member to be strengthened in the potential plastic hinge region.

(2) The necessary amount of confinement pressure to be applied depends on the ratio $I_\chi = \mu_{\chi,tar}/\mu_{\chi,ava}$, between the target curvature ductility $\mu_{\chi,tar}$ and the available curvature ductility $\mu_{\chi,ava}$, and can be evaluated as:

$$f_l = 0.4\ I_\chi^2\,\frac{f_{cd}\cdot\varepsilon_{cu}^2}{\varepsilon_{ju}^{1.5}} \tag{A.19}$$

where f_{cd} is the concrete design strength, ε_{cu} is the concrete ultimate strain and ε_{ju} is the adopted FRP jacket ultimate strain, which is lower than the ultimate strain of FRP ε_{fu}.

(3) For the case of circular cross-sections wrapped with continuous sheets (not in strips), the confinement pressure is related to the amount of FRP sheet ρ_j through: $f_l = \frac{1}{2}\rho_j\,E_j\,\varepsilon_{ju}$, with E_j being the jacket elastic modulus. The thickness of the FRP jacket can be calculated as: $t_j = \rho_j d_j/4$, where d_j is the diameter of the jacket around the circular cross-section.

(4) For the case of rectangular cross-sections in which the corners have been rounded to allow wrapping the FRP around them, the confinement pressure is evaluated as: $f_l' = k_s f_l$, with $k_s = \frac{2R_c}{D}$ and $f_l = 2E_j \varepsilon_{ju} t_j / D$, where D is the larger section width.

(5) For the case of wrapping applied through strips with spacing s_f, the confinement pressure is evaluated as: $f_l' = k_g f_l$, with $k_g = (1 - s_f / 2d)^2$.

A.4.3.3 Clamping of lap-splices

(1) Slippage of lap-splices can be prevented by applying a lateral pressure σ_l through FRP jackets. For circular columns, having diameter *D*, the necessary thickness may be estimated as:

$$t_f = \frac{D(\sigma_l - \sigma_{sw})}{2E_j \cdot 0.001} \tag{A.20}$$

where σ_{sw} is the hoop stress in the stirrups at a strain of 0.001, or the active pressure from the grouting between the FRP and the column, if provided, while σ_{sw} represents the clamping stress over the lap-splice length L_s, as given by:

$$\sigma_l = \frac{A_s f_{yd}}{\left[\frac{p}{2n} + 2(d_b + c)\right] L_s} \tag{A.21}$$

where A_s and f_{yd} are the area and the design yield strength of longitudinal steel reinforcement, respectively, *p* is the perimeter line in the column cross-section along the inside of longitudinal steel, *n* is the number of spliced bars along *p*, and *c* is the concrete cover thickness.

(2) For rectangular columns, the expressions above may be used by replacing *D* by b_w, the section width, and by reducing the effectiveness of FRP jacketing by means of the coefficient in the previous paragraph, item 4).

ANNEX B (Informative)

B STEEL AND COMPOSITE STRUCTURES

B.1 Scope

This section contains information for the assessment of steel and composite framed buildings in their present state and for their upgrading, when necessary.

Seismic retrofitting may be either local or global.

B.2 Identification of geometry, details and materials

(1) The behaviour of structural components is dictated by such properties as area, width-to-thickness and slenderness ratios, lateral buckling resistance and connection details. Thus, to perform the seismic retrofitting of steel and composite buildings the evaluation of the following properties is of paramount importance:

i. Original cross-sectional shape and physical dimensions.

ii. Size and thickness of additional connected materials, including cover plates, bracing and stiffeners.

iii. Existing cross sectional area, section moduli, moment of inertia, and torsional properties at critical sections.

iv. As built configuration of intermediate, splice and end connections.

v. Current physical conditions of base metal and connector materials including the presence of distortions.

vi. Current physical condition of primary and secondary components including the presence of any degradation.

vii. Condition of assessment of existing buildings site condition.

Areas of reduced stress, such as flange tips at beam-column ends and external plate edges, should be selected for inspection as far as possible.

To evaluate materials properties, samples should be removed from web plates of hot rolled profiles for components designed as dissipative.

Flange plate specimens should be used to characterise the material properties of non-dissipative members and/or connections.

Gamma radiography, ultrasonic testing through the architectural fabric or boroscopic review through drilled access holes are viable testing methods when accessibility is limited or for composite components.

Soundness of base and filler materials should be proved on the basis of chemical and metallurgical data.

Charpy V-Notch toughness tests should prove that heat affected zones, if any, and surrounding material have adequate resistance for brittle fracture.

Destructive and/or non-destructive testing (liquid penetrant, magnetic particle, acoustic emission), and ultrasonic or tomography methods can be used.

B.3 Requirements on geometry, details and materials

B.3.1 Geometry

(1) Steel sections should satisfy width-to-thickness slenderness limitations based on coupled buckling of flange and web plates.

(2) The following interaction relationship should be satisfied by wide flange sections:

$$\frac{\left(\frac{c}{t_f}\right)^2}{\left(\frac{K_1}{\sqrt{f_y}}\right)^2}+\frac{\left(\frac{d_e}{t_w}\right)^2}{\left(\frac{K_2}{\sqrt{f_y}}\right)^2}\leq 1 \qquad \text{(B.1)}$$

in which c and t_f are the flange outstand and thickness, t_w is the web thickness and f_y is the yield strength which should be conservatively assumed equal to the web yield strength. The equivalent section height (d_e) should be computed as follows:

$$d_e=\frac{1}{2}\left[1+\frac{A}{A_w}\cdot s\right]\cdot d \qquad \text{(B.2.1)}$$

where A_w and A are the web and total area of profiled sections; d is the section depth.

The normalised critical stress ($s = f_{cr}/f_y$) should be computed via the following formula based on regression analysis:

$$\frac{1}{s}=0.689+0.651\frac{1}{\alpha_f}+0.0553\frac{1}{\alpha_w}\pm 0.0303 \qquad \text{(B.2.2)}$$

with flange (α_f) and web (α_w) slenderness parameters expressed as:

$$\alpha_f=\frac{E}{f_{yf}}\cdot\left(\frac{t_f}{c}\right)^2 \qquad \text{(B.2.3)}$$

$$\alpha_w=\frac{E}{f_{yw}}\cdot\left(\frac{t_w}{d_e}\right)^2 \qquad \text{(B.2.4)}$$

with f_{yf} and f_{yw} the yield strength of flanges and webs, respectively. Conservatively, it should be assumed:

$$f_{yf} = f_{yw} = f_y \tag{B.2.5}$$

The coefficients K_1 and K_2 are provided in Table B.1 for beams and columns as a function of the LSs. However, the maximum axial loads in the columns should not be greater than 30% of the squash load.

Table B.1. - K_1 and K_2 for beams and columns.

	Beams			Columns		
	DL	SD	NC	DL	SD	NC
K_1	200	181	139	208	192	150
K_2	1289	1170	893	869	710	557

To achieve adequate ductility in dissipative zones, the distance (d_{PNA}) of the plastic neutral axis (PNA) from the maximum concrete compression fibre in composite cross sections should be checked at each LS. The d_{PNA} should not exceed the following limitation:

$$d_{PNA} \leq \frac{d_b + d_c}{1 + \alpha \cdot \left(\frac{f_y}{E}\right)} \tag{B.3}$$

with d_b the depth of the beam and d_c the concrete cover thickness, if any. The coefficient α is a function of the LS; values should not exceed those in Table B.2.

Table B.2. - Values of α in eqn. (B.3).

DL	SD	NC
1000	2500	2850

(3) The transverse links enhance the rotation capacities of beam-columns even with slender flanges and webs. Such transverse bars should be welded between the flanges (Figure B.1).

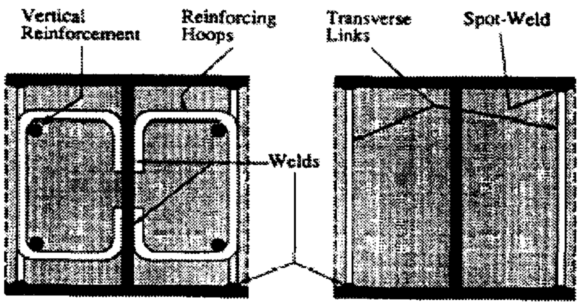

Figure B.1. - Stirrups and flange buckling inhibitors for encased members.

The transverse links should be spaced as transverse stirrups used for encased members.

B.3.2 Detailing

(1) Complete penetration groove welds should be used when joining steel components, particularly at beam-to-column, bracing and base plate connections.

Cover thicknesses should not be less than 50 mm for composite members.

B.3.3 Materials

B.3.3.1 Structural Steel

(1) Steel grades as in EN 1998-1 should be used for new parts or to replace existing structural components.

Specified minimum yield strength should be used to design dissipative components that are expected to yield under the design earthquake.

The effects of composite action should be used to evaluate the strength and stiffness of the structural components at each LS.

The through-thickness resistance in column flanges should be based upon the reduced strength as follows:

$$f_u = 0.90 \cdot f_y \tag{B.4}$$

Base material with thickness greater than 40mm should possess toughness not less than 27J measured at 20°C. Charpy V-Notch (CVN) tests are adequate to perform toughness tests.

Weld metal CVN toughness should be not less than 27J measured at -30°C.

In wide flange sections coupons should be cut from intersection zones between flange and web. This is, in fact, an area (k-area) of potentially reduced notch toughness because of the slow cooling process during fabrication.

B.3.3.2 Reinforcement Steel

(1) Reinforcement steel for new and existing RC parts should satisfy the requirements in 5.5 of EN 1998-1. Ductility class H should be used for dissipative and non-dissipative zones.

B.3.3.3 Concrete

(1) Concrete classes should range between C20/25 and C40/50 either for dissipative and non-dissipative zones.

B.4 Member Retrofitting

B.4.1 General

(1) Beams should develop full plastic moments without flange or web local buckling at SD. However, local buckling should be limited at NC.

(2) Axial and flexural yielding and buckling should be avoided in beam-columns at LSs of DL and SD.

(3) Diagonal braces should sustain plastic deformations and dissipate energy through successive cycles of yielding and buckling. However, the amount of buckling should be limited particularly at DL.

(4) Steel plates should be welded to flange and/or webs to reduce the slenderness ratios.

(5) Encasement of structural steel beams should be performed in compliance with requirements in B.4.2.2.

(6) The moment capacity $M_{pb,Rd}$ of a beam at the location of the plastic hinge should be computed as:

$$M_{pb,Rd} = \gamma_{ov} \cdot Z_e \cdot f_{yb} \tag{B.5}$$

where γ_{ov} is the overstrength partial safety factor, Z_e is the effective plastic section modulus of the section at the plastic hinge location.

(7) The moment demand $M_{cf,Sd}$ in the critical section at a column flange is evaluated as follows:

$$M_{cf,Sd} = M_{pb,Rd} + V_{pb,Rd} \cdot e \tag{B.6}$$

where $M_{pb,Rd}$ and $V_{pb,Rd}$ are respectively the beam plastic moment and the shear at the plastic hinge; e is the distance between the plastic hinge and the column face and d_c the column depth.

(8) The moment demand $M_{cc,Sd}$ in the critical section at a column centreline is as follows:

$$M_{cc,Sd} = M_{pb,Rd} + V_{pb,Rd} \cdot \left(e + \frac{d_c}{2} \right) \tag{B.7}$$

where d_c is the column depth.

B.4.2 Beams

B.4.2.1 Stability Deficiencies

(1) Width-to-thickness ratios should satisfy the limitation provided in eq. (B.1).

(2) Beam with span-to-depth ratios lying between 7 and 10 should be preferred to enhance energy absorption. Therefore, intermediate supports should be used to shorten long spans.

(3) Unsupported lengths (L/r_y) should be computed accurately. To achieve adequate rotational ductility they should satisfy at DL the following:

$$\frac{L}{r_y} \leq 70 \cdot \sqrt{\frac{235}{f_y}} \tag{B.8}$$

(4) Lateral support should be provided only for bottom flange if the slab composite action is reliable. Alternatively, the composite action should be augmented by fulfilling the requirements in B.4.2.4.

B.4.2.2 Resistance Deficiencies

(1) Added plates should be added only to bottom flange if slab composite action is reliable. Alternatively, the composite action should be augmented by fulfilling the requirements in B.4.2.4.

Flexural Capacity

(2) Structural steel beams should be encased in RC concrete in compliance with EN 1998-1. Adequate longitudinal reinforcement bars as in EN 1992-2 for ductility class H should also be used to perform satisfactory at SD and NC. However, reinforcement should not be lower than ductility class M.

Shear Capacity

(3) Steel plates should be added parallel to the beam web for H-section or parallel to the wall thickness for hollow sections.

(4) Additional transverse reinforcement should be provided for the RC encasement in compliance with EN 1998-1 and prEN1998-3 and rules for ductility class H in EN 1992-2 for the LS of SD and NC. Ductility class M can be used for DL.

(5) Structural steel beams should be encased in RC concrete in compliance with requirements in EN 1998-1.

B.4.2.3 Repair of Buckled and Fractured Flanges

(1) Buckled and/or fractured flanges should be either strengthened or replaced with new plates.

(2) Buckled bottom and/or top flanges should be repaired through full height web stiffeners on both sides of the beam webs, heat straightening or cutting of the buckled flange and replacement with similar plate.

(3) Web stiffeners should be located at the edge and centre of the buckled flange, respectively; the stiffener thickness should be equal to the beam web to achieve satisfactory performance at SD and NC.

(4) New plates should be either welded in the same location as the original flange, i.e., welding the plate directly to the beam web, or welded onto the existing flange. In both cases the added plates should be oriented with the rolling direction in the proper direction.

(5) Special shoring of the flange plates should be provided during the intervention of cutting and replacement.

(6) Local buckling of beam flanges and webs should be prevented through either transverse stiffeners or by encasing the beam in RC (composite action).

(7) Transverse stiffeners should possess adequate strength and stiffness to prevent local buckling. Therefore, they should be compliant with the design rules for new buildings as in EN 1993-3 and EN 1994-4.

(8) The encasement of steel beams in RC should be preferred to plates welded onto the flanges in the case of thick plates.

(9) Encasement of steel beams should be either partial or full.

(10) The transverse links as in Figure B.1 should be used to enhance the local ductility. Their spacing should conform to EN 1998-1.

B.4.2.4 Composite Action

(1) The presence of composite slabs should be accounted for to evaluate beam strength and stiffness at all LSs.

(2) The averaged inertia (Ic) of composite beams at DL should be assumed as follows:

$$I_c = 0.4 \cdot I^+ + 0.6 \cdot I^- \qquad \text{(B.9)}$$

where I^+ and I^- are respectively the moment of inertia of the composite section under positive and negative moments.

(3) The capacity of composite beams should account for the degree of shear connection between the steel member and the slab.

(4) Shear connectors between steel beams and composite slabs should not be used within dissipative zones. They should be removed if present in existing buildings.

(5) For ductile connectors as in EN 1994-4, partial shear connection may be adopted with a minimum connection degree of 0.80 to achieve satisfactory performance at all LSs. Full shear connection is required for non ductile connectors.

(6) The design resistance of connectors in dissipative zones should be obtained from the design resistance provided for non seismic design applying at the LS of DL a reduction factor equal to 0.75.

(7) Studs should be attached to flanges arc-spot welds but without full penetration of the flange. Either shot or screwed attachments should be avoided.

(8) It should be checked that the maximum tensile strains due to the presence of composite slabs do not provoke flange tearing.

(9) Encased beams should have stiffeners and stirrups in compliance with EN 1998-1 and prEN 1998-3.

B.4.2.5 Weakening of Beams

(1) Local ductility of steel beams is improved by weakening of the cross section at desired locations, i.e. shifting the dissipative zones from the column flange.

(2) Reduced beam sections (RBSs) or dog-bones behave like a fuse thus protecting beam-to-column connections against early fracture. Thus, the minimum rotations that should be provided at each LS are given in Table B.3.

Table B.3. - Rotations of RBSs (in *radians*).

DL	SD	NC
0.010	0.025	0.040

(3) To achieve the rotations given in Table B.3 the design of RBS beams should be carried out through the procedure outlined hereafter:

i. Compute the length and position of the flange reduction by defining a and b (Figure B.2) as follows:

$$a = 0.60 \cdot b_f \tag{B.10.1}$$

$$b = 0.75 \cdot d_b \tag{B.10.2}$$

where b_f and d_b are flange width and beam depth, respectively.

ii. Compute the distance of the plastic hinge formation (s) from the beam edge given by:

$$s = a + \frac{b}{2} \tag{B.11}$$

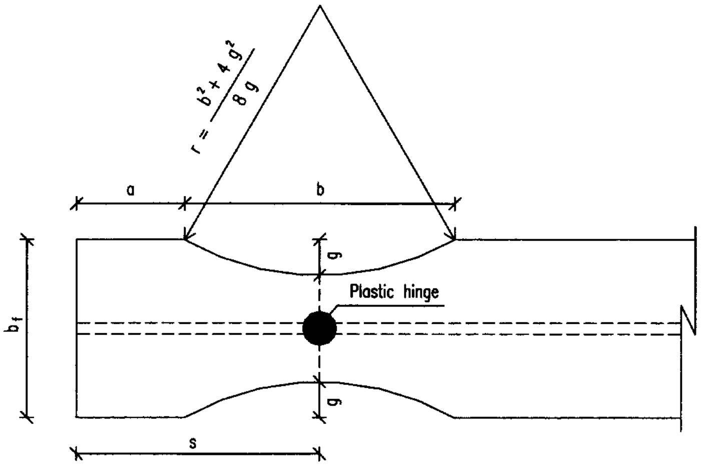

Figure B.2. - Geometry of radius cut for RBS.

iii. Compute the depth of the flange cut (g); it should be not greater than $0.25 \cdot b_f$. However, as first trial assume:

$$g = 0.20 \cdot b_f \tag{B.12}$$

iv. Compute the plastic module (Z_{RBS}) and hence the plastic module ($M_{pl,Rd,RBS}$) of the RBS:

$$M_{pl,Rd,RBS} = Z_{RBS} \cdot f_y \tag{B.13.1}$$

The plastic module (Z_{RBS}) of the RBS is

$$Z_{RBS} = Z_b - 2 \cdot g \cdot t_f \cdot (d_b - t_f) \tag{B.13.2}$$

where Z_b is the plastic module of the beam.

v. Compute the plastic shear ($V_{pl,\,Rd}$) in the section of plastic hinge formation via the free body equilibrium of the beam part (L') between hinges (Figure B.3):

$$V_{pl,Rd} = \frac{2 \cdot M_{pl,Rd,RBS}}{L'} + \frac{w \cdot L'}{2} \tag{B.14}$$

where w are the uniform beam gravity loads. Additional point loads along the beam span, if any, should be however accounted for.

vi. Compute the beam expected plastic moment ($M_{pl,Rd,be}$) as follows:

$$M_{pl,Rd,be} = \left(\frac{f_{ue} + f_{ye}}{2 \cdot f_{ye}} \right) \cdot Z_b \cdot f_{ye} \tag{B.15}$$

with f_{ue} and f_{ye} are respectively the tensile and yield expected strengths.

vii. Check that the bending moment $M_{cf,Sd}$ is less than $M_{pl,Rd,be}$; otherwise increase the cut-depth c and repeat steps (iv) to (vi). The length g should be chosen such that the maximum moment at the column flange is about 85% to 100% of the beam expected plastic moment.

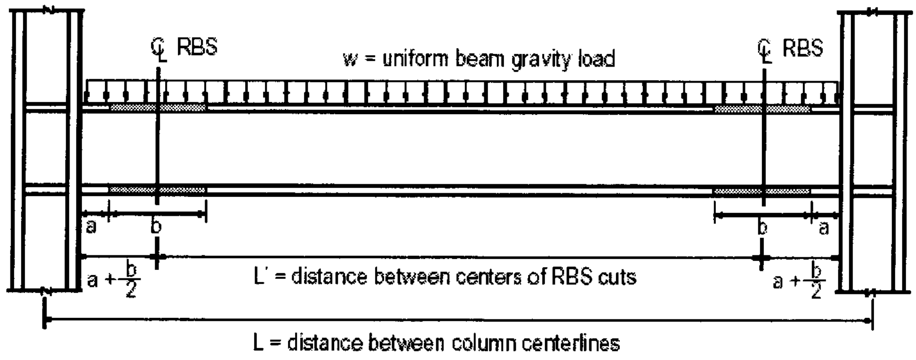

Figure B.3. -Typical sub-frame assembly with RBS.

viii. Check width-to-thickness ratios to prevent local buckling by satisfying the following:

$$\frac{\left(\dfrac{c}{t_f}\right)^2}{\left(\dfrac{E}{1.6 \cdot f_{yf}}\right) \cdot \left(\dfrac{1}{s} - 0.6003\right)} + \frac{1.25 \cdot \left(\dfrac{d_e}{t_w}\right)^2}{\left(\dfrac{E}{0.1535 \cdot f_{yf}}\right) \cdot \left(\dfrac{1}{s} - 0.6003\right)} \leq 1 \tag{B.16}$$

with s given in B.3.1.2. The flange width should be measure at the ends of the centre of 2/3 of the reduced section of the beam unless gravity loads are large enough to shift the hinge point significantly from the centre point of the reduced section.

ix. Compute the radius (r) of cuts in both top and bottom flanges over the length b of the beam:

$$r = \frac{b^2 + 4 \cdot g^2}{8 \cdot g} \tag{B.17}$$

x. Check that the fabrication process ensures the adequate surface roughness, i.e. 13 μm; for the finished cuts and grind marks are not present.

B.4.3 Columns

B.4.3.1 Stability Deficiencies

(1) Shallower column sections should be preferred to deeper ones.

(2) Hollow sections should be used to prevent lateral torsional buckling.

(3) Width-to-thickness ratios should satisfy the limitation provided in eqn. (B.1).

(4) Local buckling checks for hollow sections should employ a reduction of 20% for the wall slenderness with regard to the limits in EN 1993-3 to achieve satisfactory performance at DL and SD.

(5) Steel plates should be welded to flange and/or webs to reduce the slenderness ratios in compliance with eqn. (B.1).

(6) Wall slenderness of hollow section should be reduced by welding external steel plates.

(7) Encasement of structural steel columns should be performed in compliance with requirements in compliance with EN 1994-4, EN 1998-1 and B.4.3.53.

(8) Unsupported lengths (L/r_y) should be computed accurately. To achieve rotational ductility of 2.0 at DL, 4.0 at SD and 10 at NC the L/r_y should comply with the following limitation:

$$\frac{L}{r_y} \leq 50 \cdot \sqrt{\frac{235}{f_y}} \qquad \text{(B.18)}$$

(9) Adequate lateral support of column flanges should be provided at beam-to-column connections if the following condition is not satisfied:

$$\frac{\sum \overline{M^c_{i,Rd}}}{M_{cc,Sd}} \leq 2.0 \qquad \text{(B.19.1)}$$

in which $M_{cc,Sd}$ is bending moment in the critical section at column centreline and $\sum \overline{M^c_{i,Rd}}$ is the sum of the moment capacities of columns above and below the joint at the intersection of the beam and column centrelines. These moments should be evaluated by using the nominal flexural strength f_{yc} and accounting for a reduction for the axial load in the column as follows:

$$\sum \overline{M^c_{i,Rd}} = \sum \left[Z_c \cdot \left(f_{yc} - \frac{N_c}{A_c} \right) \right]_i \qquad \text{(B.19.2)}$$

where Z_c is the plastic modulus of the column section. The plastic modulus should account for haunches, if any. N_c and A_c are respectively the axial load and the area of the column section.

(10) Lateral support should be provided for both flanges. Stiffeners should have minimum strength at DL equal to:

$$0.06 \cdot f_{ye} \cdot b_f \cdot t_f \qquad \text{(B.20)}$$

with b_f and t_f the flange width and thickness, respectively.

B.4.3.2 Resistance Deficiencies

Axial Capacity

(1) Steel plates should be welded parallel to the flanges and/or webs for H-sections and parallel to the wall thickness for hollow sections.

(2) Structural steel columns should be encased in RC in compliance with requirements in EN 1998-1, B.4.3.53 and rules in EN 1992-2.

(3) The level of axial load should be reduced to 1/3 of the squash load at DL and 1/2 at SD and NC.

Flexural Capacity

(4) Steel plates should be welded parallel to both flanges and/or webs for H-sections and parallel to the wall thickness for hollow sections.

(5) Structural steel columns should be encased in RC in compliance with requirements in EN 1998-1, B.4.3.53 and rules in EN 1992-2.

Shear Capacity

(6) Steel plates should be added parallel to the column web for H-section or parallel to the wall thickness for hollow sections.

(7) Additional transverse reinforcement should be provided for the RC encasement in compliance with EN 1998-1 and prEN 1998-3.

(8) Structural steel columns should be encased in RC concrete in compliance with EN 1998-1.

B.4.3.3 Repair of Buckled and Fractured Flanges and Splices Fractures

(1) Buckled and/or fractured flanges and splice fractures should be either strengthened or replaced with new plates.

(2) Buckled and fractured flanges should be repaired through removal and replacement of the buckled plate flange with similar plate or flame straightening.

(3) Splice fractures should be repaired adding external plates on the column flanges via complete penetration groove welds. Thus the damaged part should be removed and replaced with sound material. The thickness of added plates should be equal to the existing ones and the replacement material should be aligned with the rolling direction matching that of the column.

(4) Small holes should be drilled at the edge of the crack to prevent its propagation.

(5) Magnetic particle or liquid dye penetrant tests should be used to ascertain that within a circular neighbour of the cracks, with radius of about 150mm, there are no defects and/or discontinuities.

B.4.3.4 Requirements for Column Splices

(1) Groove welds should be used in welded column splices.

(2) Net tension forces should be avoided in the welds of column splices.

(3) Splices should be located in the middle third of the column clear height. They should be designed to develop nominal strength not less than expected shear strength of the smaller connected member and 50% of the expected flexural strength of the smaller connected section. Thus, each flange of welded column splices should satisfy at DL the following:

$$A_{pl} \cdot f_{y,pl} \geq 0.50 \cdot R \cdot f_{y,fl} \cdot A_{fl} \tag{B.21}$$

where A_{pl} and $f_{y,pl}$ are the area and the nominal yield strength of each flange. The second member of eqn.(B.21) represents the expected yield strength of the column material. A_{fl} is the flange area of the smaller column connected.

B.4.3.5 Column Panel Zone

(1) Column panel zone should remain elastic at DL.

(2) The panel resistance ($V_{pz,Rd}$) should be computed at DL as follows:

$$V_{pz,Rd} = d_c \cdot t_p \frac{f_{y,wc}}{\sqrt{3}} \tag{B.22.1}$$

where d_c is the column depth and t_p the panel thickness. Note that if no doubler plates are present:

$$t_p = t_{wc} \tag{B.22.2}$$

with t_{wc} the column web thickness. However, to include the doubler plate thickness, plug welds between column web and added plate should be used.

(3) The post-yield resistance of the panel ($V_{pz,ult}$) should be computed for both SD and NC as follows:

$$V_{pz,ult} = d_c \cdot t_p \frac{f_{y,wc}}{\sqrt{3}} \left[1 + \frac{3 \cdot b_{cf} \cdot t_{fc}^2}{d_b \cdot d_c \cdot t_p} \right] \tag{B.23}$$

where b_{cf} and t_{cf} are the width and the thickness of the column flange, respectively.

(4) The thickness (t_p) of the column panel should comply with the following empirical equation to prevent premature local buckling under large inelastic shear deformations:

$$t_p \leq \frac{d_z + w_z}{90} \tag{B.24}$$

where d_z and w_z are respectively the panel-zone depth between continuity plates and panel-zone width between column flanges. The thickness of the column web includes the doubler plate, if any; plug welds between web and added plate should be used.

(5) Doubler plates may be used to stiffen and strength the column web. Such plates should be welded to column flanges. The welds should be either complete penetration groove or fillet.

(6) Continuity plates should be welded onto the column web at the same distance of beam flanges in beam-to-column connections. Welds should be full penetration.

(7) The thickness of continuity plates should be equal to that of beam flanges and should be placed symmetrically on both sides of the column web. This detail ensures adequate performance at all LSs.

B.4.3.6 Composite Action

(1) RC encasement should be used to enhance the stiffness, strength and ductility of steel columns. Details should conform to the design of new composite steel and concrete buildings as in EN 1998-1.

(2) Encased columns should have stiffeners and stirrups in compliance with EN 1998-1 and prEN 1998-3.

(3) To achieve effective composite action shear stresses should be transferred between the structural steel and reinforced concrete hence shear connectors should be placed along the column.

(4) To prevent shear bond failure the steel flange ratio (b_f/B) at DL should be less than the critical steel flange ratio defined as follows:

$$\left(\frac{b_f}{B}\right)_{cr} = 1 - 0.35 \cdot \left[0.17 \cdot \left(1 + 0.073 \cdot \frac{N_d}{A_g} \right) \cdot \sqrt{f_c} + 0.20 \cdot \rho_w \cdot f_{ywh} \right] \tag{B.25}$$

in which N_d is the design axial, A_g the gross area of the section, f_c is the concrete compressive strength and ρ_w and f_{ywh} are respectively the ratio and yield strength of transverse reinforcement. B is the width of the composite section, while b_f is the flange width.

B.4.4 Bracings

B.4.4.1 Stability Deficiencies

(1) Width-to-thickness ratios should satisfy the limitation provided in eqn. (B.1).

(2) Local buckling checks for hollow sections should employ a reduction of 20% for the wall slenderness with regard to the limits in EN 1993-3 to achieve satisfactory performance at DL and SD.

(3) Steel plates should be welded to flange and/or webs to reduce the slenderness ratios.

(4) Encasement of structural steel beams should be performed in compliance with requirements in B.4.3.53.

(5) The brace non-dimensionalised slenderness ($\overline{\lambda}$) should comply with the following limitation:

$$1.5 \leq \overline{\lambda} \leq 1.8 \tag{B.26}$$

(6) Lateral stiffness of diagonal braces should be improved by increasing the stiffness of the end connections.

(7) Lateral support for both flanges of structural steel bracings should be computed on the basis of the expected flange yield strength. Stiffeners should have minimum strength equal to:

$$0.06 \cdot f_{ye} \cdot b_f \cdot t_f \tag{B.27}$$

with b_f and t_f the flange width and thickness, respectively.

(8) V-Type, Inverted V-Type and K-Type bracings should be avoided.

(9) Toe-to-toe configurations of double angles should be preferred because they generally minimize bending strains and local buckling.

(10) Close spacing of stitches is effective to improve the post-buckling response of braces, particularly for double-angle and double-channel braces. Thus, if stitch plates are already in place it is recommended to weld new plates and/or strengthen existing stitch connections.

B.4.4.2 Resistance Deficiencies

(1) Bracings members should possess adequate axial, flexural and shear capacity.

Axial Capacity

(2) Steel plates should be welded parallel to the flanges and/or webs for H-sections and parallel to the wall thickness for hollow sections.

(3) The unsupported length should be reduced via transverse stiffeners designed in compliance with B.4.4.1.

(4) Structural steel bracings should be encased in RC in compliance with requirements in EN 1998-1, prEN 1998-3 and design rules in EN 1992-2.

(5) Reduce the level of axial load to 80% of the squash load at DL.

Flexural Capacity

(6) Steel plates should be welded parallel to both flanges for H-sections and parallel to the wall thickness for hollow sections.

(7) Structural steel bracings should be encased in RC in compliance with requirements in EN 1998-1, prEN 1998-3 and design rules in EN 1992-2.

Shear Capacity

(8) Steel plates should be added parallel to the bracing web for H-section or parallel to the wall thickness for hollow sections.

(9) Additional transverse reinforcement should be provided for the RC encasement in compliance with EN 1998-1 and prEN 1998-3.

(10) Structural steel bracings should be encased in RC concrete in compliance with requirements in EN 1998-1.

B.4.4.3 Composite Action

(1) The encasement of steel bracings in RC increases their stiffness, strength and ductility. Partial or full RC encasements of steel braces should be used for H-sections.

(2) Encased bracings should have stiffeners and stirrups in compliance with EN 1998-1 and prEN 1998-3. The transverse confinement of RC should be spread uniformly along the brace and should comply with details for class ductility M as in EN 1992-2.

(3) Bracings with hollow sections should be filled with concrete with grade complying B.3.3.30.

(4) Composite bracings in tension should be designed on the basis of the structural steel section alone.

(5) The capacity design checks for bracings in either CBFs or EBFs should be base on expected rather than nominal resistances and should account adequately the overstrength of the brace due to the material and/or composite action, if any.

B.4.4.4 Unbounded Bracings

(1) Braces may be stiffened either in RC walls or concrete-filled tube (unbounded braces).

(2) The brace should be coated with debonding material in order to reduce the bond stress between the steel component and the RC panels or the infilling concrete.

(3) Low yield strength steels should be used for the steel bracing, while steel-fibre reinforced concrete may be used as unbonding material.

(4) The design of braces stiffened in RC walls at DL should comply the following:

$$\left(1-\frac{1}{n_E^B}\right)\cdot m_y^B > 1.30\cdot\frac{a}{l} \tag{B.28.1}$$

in which a and l are the initial imperfection and the length of the steel brace, respectively.

(5) The non-dimensional strength (m_y^B) and stiffness (n_E^B) of the RC panel parameters are given by:

$$m_y^B = \frac{M_y^B}{N_y \cdot l} \tag{B.28.2}$$

$$n_E^B = \frac{N_E^B}{N_y} \tag{B.28.3}$$

where:

$$M_y^B = \frac{5\cdot B_S \cdot T_C^2 \cdot f_{ct}}{6} \tag{B.28.4}$$

and

$$N_E^B = \frac{5\cdot \pi^2 \cdot B_S \cdot E_B \cdot T_C^3}{12\cdot l^2} \tag{B.28.5}$$

where E_B is the elastic modulus of the RC panel, B_B the width of the steel flat-bar brace, T_C is the thickness of the panel and f_{ct} the tensile strength of the concrete. N_y is the yield strength of the steel brace.

(6) Edge reinforcement of the RC panel should be adequately anchored to prevent the punching shear.

(7) Infilled concrete tube with debonding material should be adequate to prevent buckling of steel bracing.

B.5 Connections

(1) Design rules to retrofit beam-to-column connections, bracing and link connections are provided hereafter.

(2) The provided retrofitting strategies can be applied to steel and composite frames.

(3) Transverse steel plates (face bearing plates) should be used to stiffen the flanges of the beams framing into either RC or encased columns.

B.5.1 Beam-to-Column Connections

(1) The retrofitting schemes shift the beam plastic hinge away from the face column.

(2) Two rehabilitation measures may be adopted: either weakening of the beam section at a certain distance from the column flange (connections with RBS beam) or strengthening of the beam section at the column face (haunch and cover plate connections).

(3) The simplest way to upgrade welded beam-to-column connections consists of replacing the filler metal at the joined parts (improved unreinforced connections).

(4) The panel zone of the column should be checked to remain elastic and avoid premature local buckling. Strength and stability checks should conform B.4.3.5.

(5) The column-beam moment ratio (CBMR) should be computed at DL as follows if not specified otherwise:

$$CBMR = \frac{\sum M_{e,Rd,c}}{\sum M_{j,Rd,be}} \geq 1.50 \qquad \text{(B.29.1)}$$

with:

$$\sum M_{i,Rd,c} = \sum \left[Z_c \cdot \left(f_{yc} - \frac{N_c}{A_c} \right) \right]_i \qquad \text{(B.30.2)}$$

where Z_c is the plastic modulus of the column section, evaluated on the basis of actual geometrical properties, if available, rather than from standard tables. The plastic modulus should account for haunches, if any. N_c and A_c are respectively the axial load and the area of the column section. f_{yc} is the nominal yield strength of the columns.

(6) $\sum M_{j,Rd,be}$ is the sum of expected flexural strengths at plastic hinge locations to the column centreline. It should be computed as follows:

$$\sum M_{j,Rd,be} = \sum \left\lfloor Z_b \cdot f_{yb,e} + M_{cc,Sd} \right\rfloor_j \qquad \text{(B.30.3)}$$

in which Z_b is the plastic modulus of the beam section at the potential plastic hinge location; it should be computed on the basis of the actual geometry. $f_{yb,e}$ is the expected yield strength of the beams. The quantity $M_{cc,Sd}$ accounts for the additional moment at the column centreline due to the eccentricity of shear at the plastic hinge within the beam as in B.4.

(7) Properties of upgraded connections along with requirements for beams and columns are provided in Table B. 4.

Table B. 4. - Properties of upgraded connections.

	IWUFCs	WBHCs	WTBHCs	WCPFCs	RBSCs
Hinge location (from column centerline)	$(d_c/2)+(d_b/2)$	$(d_c/2)+l_h$	$(d_c/2)+l_h$	$(d_c/2)+l_{cp}$	$(d_c/2)+(b/2)+a$
Beam depth (mm)	≤ 1000	≤ 1000	≤ 1000	≤ 1000	≤ 1000
Beam span-to-depth ratio	≥ 7	≥ 7	≥ 7	≥ 7	≥ 7
Beam flange thickness (mm)	≤ 25	≤ 25	≤ 25	≤ 25	≤ 44
Column depth (mm)	any	≤ 570	≤ 570	≤ 570	≤ 570
Rotation @ DL (radians)	0.013	0.018	0.018	0.018	0.020
Rotation @ SD (radians)	0.030	0.038	0.038	0.060	0.030
Rotation @ NC (radians)	0.050	0.054	0.052	0.060	0.045

Keys:

IWUFCs = Improved welded unreinforced flange connections.

WBHCs = Welded bottom haunch connections.

WTBHCs = Welded top and bottom haunch connections.

WCPFCs = Welded cover plate flange connections.

RBSCs = Reduced beam section connections.

DL = LS of damage limitation.

SD = LS of severe damage.

NC = LS of near collapse.

d_c = Column depth.

d_b = Beam depth.

l_h = Haunch length.

l_{cp} = Cover plate length.

a = Distance of the radius cut from the beam edge.

b = Length of the radius-cut.

B.5.1.1 Improved Unreinforced Connections

(1) The existing filler material should be gouged out and replaced it with sound one.

(2) Backing bars should be removed after welding because they may cause initiation of cracks.

(3) Continuity plates at the top and bottom of the panel zone should be used to strengthen and stiffen the column panel. The thickness should be not less than the beam flanges.

(4) Column web plates may be also required; the thickness should be not less than the column web.

(5) Continuity plates and web stiffeners should be welded to column flanges and web via complete joint penetration welds. These welds should be stopped short of the k-area to prevent the likelihood of brittle fracture.

B.5.1.2 Connections with RBS Beams

(1) Plastic hinges are forced to occur within the reduced sections, thus reducing the likelihood of fracture occurring at the beam flange welds and surrounding heat affected zones (HAZs).

(2) Welded webs should be used to joint the beam to the column flange. Alternatively, shear tabs should be welded to the column flange face and beam web. The tab length should be equal to the distance between the weld access holes with an offset of 5 mm; a minimum thickness of 10 mm is required. They should be either cut square or tapered edges (tapering corner about 15°) and placed on both sides of the beam web.

(3) The welds should be groove welds or fillet for the column face and fillet welds for the beam web. Bolting of the shear tab to the beam web may be used if more convenient economically.

(4) A minimum gap of 30 mm should be used between the column flange and the face of the composite slab.

(5) Shear studs should not be placed within the RBS zones.

(6) The design procedure for RBS connections is outlined below:

i. Use RBS beams designed in compliance with the procedure in B.4.2.5. However it is advised to compute the expected beam probable plastic moment ($M_{pl,Rd,be}$) as follows:

$$M_{pl,Rd,be} = \left(\frac{f_{ye} + f_{ue}}{2 \cdot f_{ye}}\right) \cdot Z_{RBS} \cdot f_{ye} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot b}\right) \qquad \text{(B.31.1)}$$

in which L is the distance between column centrelines, d_c is the column depth and b is the length of RBS.

ii. Hence, the beam expected shear ($V_{pl,Rd,be}$) is given by:

$$V_{pl,Rd,be} = \frac{2 \cdot M_{pl,Rd,be}}{L'} + \frac{w \cdot L'}{2} \tag{B.31.2}$$

in which w is the uniform load along the beam span (L') between plastic hinges:

$$L' = L - d_c - 2 \cdot b \tag{B.31.3}$$

Additional point vertical loads, if any, should be included in eqn.(B.31.2).

iii. Check the web connection, e.g. welded shear tab, by using the expected shear $V_{pl,Rd,be}$ as given in eqn.(B.31.2).

iv. Check the strong column-weak beam requirement via the CBMRs, defined as:

$$CBMR = \frac{\sum Z_c \left(f_{yc} - f_a\right)}{\sum Z_b \cdot f_{ye,b} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot b}\right) \cdot \left(\frac{f_{ue,b} + f_{ye,b}}{2 \cdot f_{ye,b}}\right)} \geq 1.20 \tag{B.32}$$

with Z_b and Z_c the plastic moduli of the beams and columns, respectively. The yield stresses are minimum values for the columns (f_{yc}) and expected for beams ($f_{ye,b}$); $f_{ue,b}$ is the expected tensile strength. f_a is the design stress in the columns.

v. Compute the thickness of the continuity plates to stiffen the column web at top and bottom beam flange. Such thickness should be equal to that of the beam flange.

vi. Check the strength and stiffness of the panel zone. It should be assumed that the panel remains elastic thus:

$$d_c \cdot t_{wc} \cdot \frac{f_{y,wc}}{\sqrt{3}} \geq \frac{\sum Z_b \cdot f_{ye,b} \cdot \left(\frac{f_{ue,b} + f_{ye,b}}{2 \cdot f_{ye,b}}\right)}{d_b} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot b}\right) \cdot \left(\frac{H - d_b}{H}\right) \tag{B.33}$$

where d_c and t_{wc} are the depth and the thickness of the column web, $f_{y,wc}$ is the minimum specified yield strength and H is the frame story height. The column web thickness t_{wc} should include the doubler plates, if any.

vii. Compute and detail the welds between joined parts.

B.5.1.3 Haunched Connections

(1) Beam-to-column connections may be strengthened by placing haunches either at bottom or at top and bottom of the beam flanges, thus the dissipative zone is forced at the end of the haunch. However, the former details are more convenient because bottom flanges are generally far more accessible than top ones and the composite slab does not have to be removed.

(2) Triangular T-shaped haunches are the most effective among the different types of haunch details. Haunches should have slope equal to 2:1 (2 horizontal and 1

vertical). Their depth should be ¼ of the beam depth for bottom haunches. Haunches should be 1/3 of the beam height for connections with top and bottom haunches.

(3) Continuity plates should be used to strengthen the column panel and should be placed at top and bottom beam flanges.

(4) Steel plates should be used at the haunch edges to stiffen the column web and beam web, respectively.

(5) The vertical stiffeners for the beam web should be full depth and welded on both sides of the web. The thickness should be proportioned to withstand the vertical component of the force at that location. However, they should be not less thick than beam flanges. It is required to perform local checks for flange bending, web yielding and web crippling in compliance with EN 1993 design formulae.

(6) Haunches should be welded via complete joint penetration welds to both column and beam flanges.

(7) Bolted shear tabs may be left in place if existing. Alternatively, shear tabs may be used if required for either structural or erection purposes.

(8) The step-by-step design procedure for haunched connections is summarized below.

i. Select preliminary haunch dimensions on the basis of slenderness limitation for the haunch web. The following relationship may be used as first trial for the haunch length (a) and its slope (θ):

$$a = 0.55 \cdot d_b \tag{B.34.1}$$

$$\theta = 30^\circ \tag{B.34.2}$$

where d_b is the beam depth. However, the haunch depth b should be compatible with architectural restraints, e.g. ceilings and non structural elements. The haunch depth is given by $b = a \cdot \tan\theta$.

ii. Compute the beam probable plastic moment ($M_{pl,Rd,be}$) at the haunch tip

$$M_{pl,Rd,be} = \left(\frac{f_{ye} + f_{ue}}{2 \cdot f_{ye}} \right) \cdot Z_b \cdot f_{ye} \tag{B.35}$$

with Z_b the plastic modulus of the beam; f_{ue} and f_{ye} are respectively the expected ultimate and yield strengths.

iii. Compute the beam probable plastic shear ($V_{pl,Rd,be}$) from force equilibrium of the beam span (L') between plastic hinges:

$$V_{pl,Rd,be} = \frac{2 \cdot M_{pl,Rd,be}}{L'} + \frac{w \cdot L'}{2} \tag{B.36}$$

in which w is the uniform load between L`; additional point vertical loads, if any, should be included in eqn.(B.36).

iv. Check the strong column-weak beam requirement via the CBMRs, defined as:

$$CBMR = \frac{\sum Z_c \cdot (f_{yc} - f_a)}{\sum M_c} \geq 1.20 \qquad \text{(B.37.1)}$$

in which Z_c is the plastic section modulus of the columns, f_{yc} is the minimum specified yield strength; f_a is the axial stress in the columns due to the design loads. M_c is the sum of column moments at the top and bottom ends of the enlarged panel zone resulting from the development of the beam moment M_{pld} within each beam of the connection. It is given as follows:

$$\sum M_c = \left[2M_{pl,Rd,be} + V_{pl,Rd,be} \cdot (L - L')\right] \cdot \left(\frac{H_c - \overline{d_b}}{H_c}\right) \qquad \text{(B.37.2)}$$

where L is the distance between the column centrelines, $\overline{d_b}$ is the depth of the beam including the haunch and H_c is the story height of the frame.

v. Compute the actual value of the non-dimensionalised parameter β given by:

$$\beta = \frac{b}{a} \cdot \left(\frac{3 \cdot L' \cdot d + 3 \cdot a \cdot d + 3 \cdot b \cdot L' + 4 \cdot a \cdot b}{3 \cdot d^2 + 6 \cdot b \cdot d + 4 \cdot b^2 + \frac{12 \cdot I_b}{A_b} + \frac{12 \cdot I_b}{A_{hf} \cos^3 \theta}} \right) \qquad \text{(B.38)}$$

where A_{hf} is the area of the haunch flange.

vi. Compute the value of the non-dimensionalised parameter β_{min} given by:

$$\beta_{\min} = \frac{\frac{(M_{pl,Rd,be} + V_{pl,Rd,be} \cdot a)}{S_x} - 0.80 \cdot f_{uw}}{\frac{V_{pl,Rd,be} \cdot a}{S_x} + \frac{V_{pl,Rd,be}}{I_b \cdot \tan\theta} \cdot \left(\frac{d^2}{4} - \frac{I_b}{A_b}\right)} \qquad \text{(B.39)}$$

where f_{uw} is the tensile strength of the welds, S_x is the beam elastic (major) modulus, d is the beam depth. A_b and I_b are respectively the area and moment of inertia of the beam.

vii. Compare the non-dimensionalised β-values, as calculated above.

If $\beta \geq \beta_{\min}$ the haunch dimensions are adequate and further local checks as should be performed. By contrast, $\beta < \beta_{\min}$ requires an increase of the haunch flange stiffness. Stiffer flanges may be obtained by either increasing the area A_{hf} or modifying the haunch geometry.

viii. Perform strength and stability checks for the haunch flange:

$$\text{(strength)} \quad A_{hf} \geq \frac{\beta \cdot V_{pl,Rd,be}}{f_{ye,hf} \cdot \sin\theta} \tag{B.40.1}$$

$$\text{(stability)} \quad \frac{b_{hf}}{t_{hf}} \leq 10 \cdot \sqrt{\frac{235}{f_y}} \tag{B.40.2}$$

where $f_{ye,hf}$ is the expected yield strength of the haunch flange; b_{hf} and t_{hw} are respectively, the flange outstanding and flange thickness of the haunch.

ix. Perform strength and stability checks for the haunch web:

$$\text{(strength)} \quad \tau_{hw} = \frac{a \cdot V_{pl,Rd,be}}{2 \cdot (1+\upsilon) \cdot I_b} \left[\frac{L'}{2} - \frac{\beta}{\tan\theta} \left(\frac{d}{2} \right) + \frac{(1-\beta) \cdot a}{3} \right] \leq \frac{f_{ye,hw}}{\sqrt{3}} \tag{B.41.1}$$

$$\text{(stability)} \quad \frac{2 \cdot a \cdot \sin\theta}{t_{hw}} \leq 33 \cdot \sqrt{\frac{235}{f_y}} \tag{B.41.2}$$

where $f_{ye,hw}$ is the expected yield strength of the haunch web, t_{hw} is the web thickness; $\nu$is the Poisson's ratio of steel.

x. Check the shear capacity of the beam web. The shear in the beam web is given by:

$$V_{pl,Rd,bw} = (1-\beta) \cdot V_{pl,Rd,be} \tag{B.42}$$

Web yielding and web crippling should also be checked on the basis of the shear in eqn.(B.42) at DL.

Design continuity plates and beam web stiffeners. Their dimensions should be adequate to withstand the concentrated force $\beta \cdot V_{pl,Rd,be}/\tan\theta$. Furthermore, web stiffeners should possess sufficient strength to resist the concentrated load $\beta \cdot V_{pl,Rd,be}$ along with the beam web.

Width-to-thickness ratios for continuity plates and web stiffeners should be limited to 15 to prevent local buckling.

xi. Perform weld detailing by using complete joint penetration welds to connect each stiffener to the beam flange. Two-sided 8 mm fillet welds are adequate to connect the stiffeners to the beam web.

B.5.1.4 Cover Plate Connections

(1) Cover plate connections reinforce the connection, reduce the stress at the beam flange welds and force the yielding in the beam at the end of the cover plates.

(2) Reinforcing plates may be used either at bottom or top and bottom beam flanges.

(3) Reinforcing plates should have rectangular shapes and should be fabricated with rolling directions parallel to the beam.

(4) Cover plate connections are more economic than haunch connections as in B.5.1.30.

(5) Connections with welded beam webs and relatively thin and short cover plates should be preferred to bolted web and heavy and long plates.

(6) Long plates should not be used for beams with short spans and high moment gradient.

(7) The step-by-step design procedure for cover plate connections is summarized below.

i. Select preliminary cover plate dimensions on the basis of the beam size:

$$b_{cp} = b_{bf} \tag{B.43.1}$$

$$t_{cp} = 1.20 \cdot t_{bf} \tag{B.43.2}$$

$$l_{cp} = \frac{d_b}{2} \tag{B.43.3}$$

where b_{cp} is the width, t_{cp} the thickness and l_{cp} the length of the cover plate.

ii. Compute the beam probable plastic moment ($M_{pl,Rd,be}$) at the end of the cover plates as in eqn. (B.8).

iii. Compute the beam probable plastic shear ($V_{pl,Rd,be}$) from force equilibrium of the beam span (L') between plastic hinges:

$$V_{pl,Rd,be} = \frac{2 \cdot M_{pl,Rd,be}}{L'} + \frac{w \cdot L'}{2} \tag{B.44.1}$$

in which w is the uniform load between L'; additional point vertical loads, if any, should be included in eqn.(B.44.1). The distance L` between the plastic hinges in the beam is as follows:

$$L` = L - d_c - 2 \cdot l_{cp} \tag{B.44.2}$$

iv. Compute the moment at the column flange ($M_{cf,Sd}$):

$$M_{cf,Sd} = M_{pl,Rd,be} + V_{pl,Rd,be} \cdot l_{cp} \tag{B.45}$$

v. Check that the area of cover plates (A_{cp}) satisfies the following requirement:

$$\left[Z_b + A_{cp} \cdot (d_b + t_{cp})\right] \cdot f_y \geq M_{cf,Sd} \tag{B.46}$$

vi. Check the strong column-weak beam requirement via the CBMRs, defined as:

$$CBMR = \frac{\sum Z_c (f_{yc} - f_a)}{\sum Z_b \cdot f_{ye,b} \cdot \left(\frac{L - d_c}{L - d_c - 2 \cdot L_{cp}} \right) \cdot \left(\frac{f_{ue,b} + f_{ye,b}}{2 \cdot f_{ye,b}} \right)} \geq 1.20 \qquad \text{(B.47)}$$

with Z_b and Z_c the plastic moduli of the beams and columns, respectively. The yield stresses are minimum values for the columns (f_{yc}) and expected values for beams ($f_{ye,b}$).

vii. Compute the thickness of the continuity plates to stiffen the column web at top and bottom beam flange. Such thickness should be equal to that of the beam flange.

viii. Check the strength and stiffness of the panel zone. It should be assumed that the panel remains elastic thus:

$$d_c \cdot t_{wc} \cdot \frac{f_{y,wc}}{\sqrt{3}} \geq \frac{\sum M_f}{d_b} \cdot \left(\frac{L}{L - d_c} \right) \cdot \left(\frac{H - d_b}{H} \right) \qquad \text{(B.48)}$$

where d_c and t_{wc} are the depth and the thickness of the column web, $f_{y,wc}$ is the minimum specified yield strength and H is the frame story height. The column web thickness t_{wc} should include the doubler plates, if any.

ix. Compute and detail the welds between joined parts, i.e. beam to cover plates, column to cover plates and beam to column. It is required minimum CVN toughness equal to 27J at -30°C. Moreover, weld overlays should employ the same electrodes or at least with similar mechanical properties.

B.5.1.5 Semi-rigid Composite Connections

(1) Semi-rigid composite connections may be used to achieve large plastic deformations without fracturing.

(2) Full interaction shear studs should be welded onto the beam top flange while reinforcing bars should be placed around the column, thus activating the composite action at all LSs.

(3) The design of composite connections may be carried out by assuming that the shear is assigned to the web angles and the bending to the slab reinforcement and beam bottom flange.

(4) The step-by-step design procedure for semi-rigid connections is summarized below.

i. Consider the plastic moment of the bare steel beam ($M_{pl,Rd,be}$) and assume that the connection ($M_{c,Sd}$) will transfer 75% of $M_{pl,Rd,be}$:

$$M_{c,Sd} = 0.75 \cdot M_{pl,Rd,be} \qquad \text{(B.49)}$$

ii. Compute the slab reinforcement (A_{rb}) necessary to carry the connection moment ($M_{c,Sd}$):

$$A_{rb} = \frac{M_{c,Sd}}{(d_b + d_1 + d_2) \cdot f_{y,rb}} \tag{B.50}$$

where d_b is the beam depth, d_1 is the right height (metal deck) and d_2 should be assumed equal to 10 mm. $f_{y,rb}$ is the yield strength of the reinforcement bars.

iii. Select the seat angle whose leg area (A_{sl}) is capable to transmit a tensile force equal to 1.33 times that due to the slab reinforcement.

iv. Select the bolts between the beam and the angle. They should be designed for a shear force equal to 1.25 times the shear in the slab.

v. Select the double web angles which may withstand the design shear.

B.5.2 Bracing Connections

(1) Brace connection design should be based upon axial loads and account for the effect of the brace member cyclic post-buckling behaviour.

(2) The strength of the bracing connection should be adequate to prevent brittle fractures of the connection and failure by out-plane gusset buckling at DL and SD.

(3) Fixed end connections should be preferred to those that are pinned.

(4) Net section fractures and block shear rupture at the end of the brace should be avoided by using expected rather than nominal strengths in the capacity design checks at DL and SD.

(5) Bracing connections should resist axial forces equal to $f_{ye,d} \cdot A_{gd}$, with $A_{g,d}$ the gross area of the brace and $f_{yd,e}$ the expected yield strength at DL.

(6) The flexural strength of the connection should be not less than $1.10 \cdot M_{pl,Rd,de}$ with $M_{pl,Rd,de}$ the expected moment of the brace about the critical buckling axis.

(7) To improve out-of-plane stability of the bracing connection the continuity between beams and columns should not be interrupted.

(8) Composite bracing connections should withstand the full tensile/compressive strength capacity of the brace.

(9) The contribution of the concrete for composite braces should be considered when the capacity design of the connection is carried out.

B.5.3 Link Connections

(1) The link connections should be capacity designed considering the expected capacity of the links. Expected values should include the overstrength due to the presence of the composite slab, if any.

(2) The link expected shear strength should be amplified by a factor equal to 1.25 to account for the overstrength.

(3) The intersection of the brace and the beam centrelines located outside the link should be avoided.

(4) Connections between the diagonal brace and the beam should have centrelines intersecting either within the length of the link or at its end.

(5) No parts of the connection should be extended over the link length.

(6) Connections at the end of the link should be designed to develop the expected strength of the brace, including overstrength due to composite action, if any.

(7) Braces should be either fully restrained or pin-connected to the link.

(8) Pinned connections should employ gusset plates with stiffeners at the free edges.

(9) The beam should resist the entire link moment.

(10) For link-to-column connections at column flange face bearing plates should be used between the beam flange plates.

(11) Links connected to the column should be short.

(12) The retrofitting of beam-to-column connections may vary the link length. Therefore, it should be checked after the repairing strategy is adopted.

(13) Welded connections of the link to the column weak-axis should be avoided.

(14) Bolted partially restrained connections may be used provided that the links yield in shear (short link).

(15) The diagonal brace-to-beam connection at the end of the link should be fully restrained if the brace resists to the link end moment.

B.6 System Retrofitting

(1) Effective global upgrading strategies should be able to increase the capacity of the structure and/or decrease the demand imposed by earthquake loads.

(2) Concentrations of high inelastic demands within irregular zones should be avoided.

(3) New and existing structural systems should satisfy the following requirements:

i. Regularity of mass, stiffness and strength distribution, to avoid detrimental torsional effects and/or soft-story mechanisms.

ii. Reduced masses and sufficient stiffness, to avoid highly flexible structures which may give rise to extensive non-structural damage and significant P-Δ effects.

iii. Continuity and redundancy between members, so as to ensure a clear and uniform load path for horizontal loads and prevent brittle failures.

(4) Global interventions should include one or more of the following strategies:

i. Global structural stiffening.

ii. Removal or lessening of existing irregularities and discontinuities.

iii. Global structural strengthening.

iv. Mass reduction.

v. Seismic isolation.

vi. Supplemental damping.

(5) Global structural stiffening and strengthening should be achieved by either using braced frames or shear walls.

(6) The augmentation of the composite action between steel beams and concrete slabs through shear studs, the encasement or beams and columns in RC, and/or the upgrading of the pinned connections in gravity-load systems should also be used to increase the global stiffness at all LSs.

(7) Mass reductions may be achieved through one of the following measures:

i. Replacement of heavy cladding systems with lighter systems.

ii. Removal of unused equipment and storage loads.

iii. Replacement of masonry partition walls with lighter systems.

iv. Removal of one or more stories.

(8) Base isolation should be used for structures with fundamental period not greater than 1.0 sec. Such period should be computed through eingenvalue analysis.

(9) Base isolation and supplemental damping should be designed in compliance with EN 1998-1 for new buildings.

B.6.1 Braced Frames

(1) Braced bays should be introduced in moment resisting frames (MRFs) to increase the lateral stiffness at DL and SD.

(2) Eccentric braced frames (EBFs) and knee-brace frames (KBFs) should be preferred to concentric braced frames (CBFs).

(3) Bolted connections should be preferred to welds for bracings in CBFs, EBFs and KBFs.

(4) Bracings in CBFs should have in compression at least 50% of the tensile capacity at DL and SD.

(5) V-bracings and inverted V-bracings should be avoided. In existing buildings adequate stiffeners should be placed at the bracing-beam connections. The beam capacity at mid-span should be checked for the design load combinations and the additional effects due to the point load (F_b) given as follows:

$$F_b = 1.30 \cdot f_{ye} \cdot \left(\frac{f_u + f_y}{2 \cdot f_y} \right) \cdot A_b \tag{B.51}$$

where f_{ye} is the expected yield strength of the brace as specified, f_u and f_y are the nominal ultimate and yield strengths of the brace the A_b is the area of the brace.

(6) Alternatively, the unbalanced force in the beams should be eliminated through ad hoc bracing configurations, e.g. macro-bracings or V-bracings with a zipper column.

(7) The use of either aluminium or stainless steel for dissipative zones in CBFs is allowed but should be validated by testing. Similarly, for EBFs and KBFs.

(8) KBFs are framed systems in which the diagonal braces are connected to a dissipative zone (knee element), which is a secondary member, instead of the beam-to-column connection.

(9) Hollow sections should be used for knee elements.

(10) Knee element should be short.

(11) The maximum length (lmax) of the two parts of the knee element should satisfy the following:

$$l_{max} \leq \frac{2 \cdot M_p`}{V_p`} \tag{B.52.1}$$

where the moments M_p' and V_p' are as follows:

$$M_p` = t_f \cdot b \cdot (d - t_f) \cdot f_y \tag{B.52.2}$$

$$V_p` = t_w \cdot (d - t_f) \cdot \frac{f_y}{\sqrt{3}} \tag{B.52.3}$$

where f_y is the yield strength, d, t_f, b and t_w are the depth, flange, thickness width and web thickness of the knee member.

(12) The floor distortions γ_s) for KBFs are given as follows:

$$\gamma_s = \frac{\delta_c + \delta_k}{b} + \frac{2 \cdot \delta_k}{L - 2 \cdot b} \tag{B.53.1}$$

where δ_c and is the vertical displacements of the floor at the column, L the length of the beam.δ_k is the vertical displacement of the floor at the knee beam connection and b is the length from the column to the knee-beam connection.

(13) For EBFs the distortion γ_s are as follows:

$$\gamma_s = \frac{2 \cdot (\delta_c + \delta_s)}{L - e_s} \tag{B.53.2}$$

with δ_c the vertical displacements of the floor at the brace ends and e_s is the length of the beam link.

B.6.2 Semi-Rigid Frames

(1) Semi-rigid or pinned steel frames may be upgraded as semi-rigid composite frames.

(2) The fundamental period of semi-rigid frames should be computed as follows:

$$T = 0.085 \cdot H^{\left(0.85 - m/120\right)} \qquad 5 < m < 18 \qquad \text{(semi-rigid)} \tag{B.54.1}$$

$$T = 0.085 \cdot H^{3/4} \qquad m \geq 18 \qquad \text{(rigid)} \tag{B.54.2}$$

where H is the frame height in metres and the parameter m is as follows:

$$m = \frac{\left(K_\varphi\right)_{con}}{\left(EI/L\right)_b} \tag{B.55}$$

in which $K\phi$ is the connection rotation stiffness, I and L are respectively the moment of inertia and the beam span. E is Young's modulus of the beam.

(3) A modified distribution of horizontal forces ($F_{x,i}$) should be used in the equivalent static analysis and for nonlinear analysis to detect the onset of all LSs:

$$F_{x,i} = \frac{W_{x,i} \cdot h_{x,i}^{\delta}}{\sum W_{x,i} \cdot h_{x,i}^{\delta}} \cdot F_t \tag{B.56.1}$$

with the power exponent δ given by:

$$\delta = \begin{cases} 1.0 & \mathrm{T} \leq 0.50\,\mathrm{s} \\ 0.50 \cdot T + 0.75 & 0.50 < \mathrm{T} < 2.50\,\mathrm{s} \\ 2.0 & \mathrm{T} > 2.50\,\mathrm{s} \end{cases} \tag{B.56.2}$$

(4) The design of semi-rigid frames should be based on slip capacity design and adequate effective lengths for the column stability checks.

(5) Beams should have yield overstrength equal to 40% at DL and SD, thus it is not required to impose column-to-beam overstrength factors.

B.6.3 Frames with Steel and Composite Walls

(1) Steel panel with thinner plates should be used to achieve ductile behaviour and to prevent column instability.

(2) Steel panels should employ low yield steel and should be shop welded-field bolted.

(3) Frames retrofitted with either stiffened or unstiffened steel plate shear walls should be designed with behaviour factors equal to 6.5.

(4) Steel plates may be either unstiffened; their width-to-thickness ratio (h/t_w) should satisfy:

$$\frac{h}{t_w} \le 100 \cdot \sqrt{\frac{235}{f_y}} \tag{B.57}$$

(5) However, non-compact shear walls should be used (see Table B. 5).

Table B. 5- Width-to-thickness ratios for steel shear walls.

	Panel Slenderness*	
Steel grade	Compact	Slender
S235	≤ 73	≥ 92
S275	≤ 68	≤ 85
S355	≤ 60	≤ 75

(6) Steel panels should be designed to withstand the entire storey shear.

(7) The expected yielding shear $V_{ye,w}$ of non compact steel panels should be evaluated as follows:

$$V_{ye,w} = \frac{f_{ye,w} + f_{ue,w}}{2 \cdot f_{ye,w}} \cdot \frac{A_w \cdot f_{ye,w}}{\sqrt{3}} \cdot 0.17 \cdot (1 - C_v) \tag{B.58.1}$$

where the coefficient C_v is given by:

$$C_v = \frac{2.46 \cdot \sqrt{E/f_{ye,w}}}{h / t_w} \tag{B.58.2}$$

with $f_{ye,w}$ and $f_{ue,w}$ the yield and ultimate expected strengths, respectively; A_w is the panel shear area.

(8) In panels with openings the continuity of tension field action should be ascertained.

(9) The windows and/or doors should be placed in the areas between the mid-height of the columns and the mid-span of the beams.

(10) Adequate beam and column member stiffness to provide effective tension field distribution should be equal to the flexibility coefficient (φ):

$$\varphi = 0.7 \cdot h \cdot \left(\frac{w}{2 \cdot L \cdot I_c} \right)^{0.25} \tag{B.59}$$

where w is the wall thickness, L the width of the panel, h the height of the panel, and I_c the moment of inertia of the column.

(11) Low panel aspect ratio (l/h), say $l/h \leq 3$, are essential to ensure development of yielding of the plate hence providing enhanced energy dissipation capacity of the retrofitted frame.

(12) Steel walls may be encased in RC (composite steel plate shear walls). The concrete stiffening may be placed on one or both sides of the plate. Infilled steel wall panels are also allowed.

(13) The design of either encased or infilled panel should take no account of the composite action. Thus, the shear strength of the system may be assumed equal to that of the steel panel.

(14) The thickness of the concrete cover and the shear studs should be designed to allow the yielding of the plate and to prevent elastic local and global buckling.

(15) A minimum thickness of 100mm (concrete on both sides) or 200mm (concrete on single side only) should be used for encasement.

(16) Reinforcement bars should be placed in vertical and horizontal directions; the spacing should not be less than 400mm.

(17) The connections between the plate and boundary members may be either welded or with bolted; complete penetration welds and high strength bolts are advised.

B.6.4 Frames with RC Walls

(1) RC walls should be used to improve the stiffness, strength and ductility of existing steel frames.

(2) RC walls should be designed to withstand the entire shear forces. The design rules should conform to EN 1998-1.

(3) Adequate steel reinforcement and connectors should be placed at the connection of the wall with the steel members.

(4) Crossties should be placed in the wall for a length equal to the section width (embedment details).

ANNEX C (INFORMATIVE)

C MASONRY STRUCTURES

C.1 Scope

(1) This annex contains recommendations for the assessment and the design of strengthening measures in masonry building structures in seismic regions.

(2) The recommendations of this section are applicable to concrete or brick masonry lateral force resisting elements within a building system in un-reinforced, confined and reinforced masonry.

C.2 Identification of geometry, details and materials

C.2.1 General

(2) The following aspects should be carefully examined:

iii. Physical condition of masonry elements and presence of any degradation;

iv. Configuration of masonry elements and their connections, as well as the continuity of load paths between lateral resisting members;

v. Properties of in-place materials of masonry elements and connections;

vi. The presence and attachment of veneers, the presence of nonstructural components, the distance between partition walls;

vii. Information on adjacent buildings potentially interacting with the building under consideration.

C.2.2 Geometry

(1) The collected data should include the following items:

i. Size and location of all shear walls, including height, length and thickness;

ii. Dimensions of masonry units;

iii. Location and size of wall openings (doors, windows);

iv. Distribution of gravity loads on bearing walls.

C.2.3 Details

(1) The collected data should include the following items:

i. Classification of the walls as un-reinforced, confined, or reinforced;

ii. Presence and quality of mortar;

iii. For reinforced masonry walls, amount of horizontal and vertical reinforcement;

iv. For multi-leaf masonry (rubble core masonry walls), identification of the number of leaves, respective distances, and location of ties, when existing;

v. For grouted masonry, evaluation of the type, quality and location of grout placements;

vi. Determination of the type and condition of the mortar and mortar joints; Examination of the resistance, erosion and hardness of the mortar; Identification of defects such as cracks, internal voids, weak components and deterioration of mortar;

vii. Identification of the type and condition of connections between orthogonal walls;

viii. Identification of the type and condition of connections between walls and floors or roofs.

ix. Identification and location of horizontal cracks in bed joints, vertical cracks in head joints and masonry units, and diagonal cracks near openings;

x. Examination of deviations in verticality of walls and separation of exterior leaves or other elements as parapets and chimneys;

xi. Identification of local condition of connections between walls and floors or roofs.

C.2.4 Materials

(1) Non-destructive testing is permitted to quantify and confirm the uniformity of construction quality and the presence and degree of deterioration. The following types of tests may be used:

i. Ultrasonic or mechanical pulse velocity to detect variations in the density and modulus of masonry materials and to detect the presence of cracks and discontinuities.

ii. Impact echo test to confirm whether reinforced walls are grouted.

iii. Radiography to confirm location of reinforcing steel.

(2) Supplementary tests may be performed to enhance the level of confidence in masonry material properties, or to assess masonry condition. Possible tests are:

i. Schmidt rebound hammer test to evaluate surface hardness of exterior masonry walls.

ii. Hydraulic flat jack test to measure the *in-situ* vertical compressive stress resisted by masonry. This test provides information such as the gravity load distribution, flexural stresses in out-of-plane walls, and stresses in masonry veneer walls compressed by surrounding concrete frame.

iii. Diagonal compression test to estimate shear strength and shear modulus of masonry.

iv. Large-scale destructive tests on particular regions or elements, to increase the confidence level on overall structural properties or to provide particular information such as out-of-plane strength, behaviour of connections and openings, in-plane strength and deformation capacity.

C.3 Methods of analysis

(1) In setting up the model for the analysis, the stiffness of the walls should be evaluated considering both flexural and shear flexibility, using cracked stiffness. In the absence of more accurate evaluations, both contributions to stiffness may be taken as one-half of their respective uncracked values.

(2) Masonry spandrels may be introduced in the model as coupling beams between two wall elements.

C.3.1 Linear methods: Static and Multi-modal

(1) These methods should be applicable under the following conditions:

i. regular arrangement of lateral load resisting walls in both directions,

ii. continuity of the walls along their height,

iii. the floors should possess enough in-plane stiffness and be safely connected to the perimeter walls in order to assume rigid distribution of the inertia forces among the vertical elements,

iv. floors on both sides of a common wall should be at the same height,

v. the ratio between the lateral stiffnesses of the stiffer wall and the weakest one, evaluated accounting for the presence of openings, should not exceed 2.5,

vi. spandrel elements included in the model should either be made of blocks adequately interlocked to those of the adjacent walls, or endowed with connecting ties.

C.3.2 Nonlinear methods: Static and Time-history

(1) These methods should be applicable when one or more of the above conditions are not met.

(2) The static method consists in the application of a set of horizontal forces of increasing intensity until attainment of the peak resistance of the structure. This is reached with a stiffness decrease due to progressive damage and failure of the participating lateral load resisting elements. The load-deformation curve is continued

after the peak, until a 20% reduction of the peak load is attained. The corresponding displacement is considered as the displacement capacity.

(3) The ultimate limit state verification of the structure consists in checking that the displacement capacity, evaluated as indicated above, is larger than the corresponding displacement induced by the elastic design seismic action.

C.4 Capacity models for assessment

C.4.1 Elements under normal force and bending

C.4.1.1 LS of near collapse (NC)

(1) The verification of the ultimate shear capacity corresponding to flexural collapse under an axial load P acting on the wall, should be made comparing the shear demand on the masonry wall with the capacity given as:

$$V_f = \frac{DP}{2H_0}(1-1.15\,\nu_d)$$

where D is the wall depth, $\nu_d = P/(D\,t\,f_d)$ is the normalized axial load (with $f_d = f_{mk}/\gamma_m$ being the masonry design strength, where f_{mk} is the characteristic compressive strength and γ_m is the partial safety factor for masonry), t is the wall thickness, and H_0 is the distance between the section to be verified and the contraflexure point.

(2) The ultimate capacity in terms of drift should be assumed equal to 0.008.

C.4.1.2 LS of severe damage (SD) and of damage limitation (DL)

(1) The verification against the exceedance of these two LS is not required.

C.4.2 Elements under shear force

C.4.2.1 LS of near collapse (NC)

(1) The verification of the ultimate shear capacity corresponding to shear collapse under an axial load P acting on the wall, should be made comparing the shear demand on the masonry wall with the capacity given as:

$$V_f = f_{vd}\,D't$$

where $f_{vd} = f_{vk}/\gamma_m$ is the shear design strength accounting for the presence of vertical load, with $f_{vk} = f_{vk0} + 0.4\frac{P}{D't} \le 0.065\,f_{mk}$, being f_{vk0} the characteristic shear strength in the absence of vertical load, and D' is the depth of the compressed area of the wall.

(2) In case the verification of the wall is governed by shear, the ultimate capacity in terms of drift should be assumed equal to 0.004.

C.4.2.2 LS of severe damage (SD) and of damage limitation (DL)

(1) The verification against the exceedance of these two LS is not required.

C.5 Structural interventions

C.5.1 Repair and strengthening techniques

C.5.1.1 Repair of cracks

(1) Cracks may be sealed with mortar if the crack width is small (e.g., less than 10 mm), and the thickness of the wall is relatively small.

(2) If the width of the cracks is small but the thickness of the masonry is considerable, cement grout injections should be used; where possible, the grout should be shrinkage-free. Epoxy grouting may be used for fine cracks.

(3) If the crack are relatively wide (e.g., more than 10 mm), the damaged area should be reconstructed using elongated (stitching) bricks of stones. Otherwise, dove-tailed clamps or metal plates should be used to tie together the two faces of the crack, and the voids should be filled with cement mortar.

(4) Where bed-joints are reasonably level, the resistance of a wall against vertical cracking can be considerably improved by embedding small diameter stranded wire rope in the bed-joints.

(5) For the repair of large diagonal cracks, vertical concrete ribs may be cast into irregular chases made in the masonry wall, normally on both sides; ribs should be reinforced with closed stirrups and longitudinal bars, while stranded wire rope as in (4) should run across the concrete ribs.

C.5.1.2 Repair and strengthening of wall intersections

(1) To improve connection between intersecting walls use should be made of cross-bonded bricks or stones. The connection may be made more effective in different ways:

i. construction of a reinforced concrete belt,

ii. addition of steel plates in the bed-joints,

iii. insertion of inclined steel bars in drilled holes and grouting thereafter.

C.5.1.3 Strengthening and stiffening of horizontal diaphragms

(1) Timber floors may be strengthened and stiffened against in-plane distortion by:

i. nailing an additional orthogonal or oblique layer of timber boards onto the existing ones,

ii. casting a thin layer of concrete reinforced with welded wire mesh. The concrete layer should have a shear connection with the timber floor, and should be anchored to the walls,

iii. placing a doubly diagonal mesh of flat steel ties anchored to the beams and to the perimeter walls.

(2) Roof trusses should be braced and anchored to the supporting walls.

C.5.1.4 Tie beams

(1) If existing tie beams between walls and floors are damaged, they should be appropriately repaired or rebuilt. If they are missing in the original structure, they should be added.

C.5.1.5 Strengthening of buildings by means of steel ties

(1) The addition of steel ties (along or transversely to the walls, external or within holes drilled in the walls) is an efficient means of connecting walls and improving the overall behavior of a masonry building.

(2) Pretensioned ties may be used to improve the resistance of the walls against tensile forces.

C.5.1.6 Strengthening of rubble core masonry walls (multi-leaf walls)

(1) The rubble core may be strengthened by cement grouting, if the penetration of the grout is satisfactory. However, if the adhesion of the grout to the rubble is likely to be poor, grouting should be complemented by insertion of steel bars across the core conveniently anchored to the walls.

C.5.1.7 Strengthening of walls by means of reinforced concrete jackets or steel profiles

(1) The concrete should be applied by the shotcrete method and the jackets should be reinforced by welded wire mesh or steel bars.

(2) The jackets may be on both sides of the wall or they may be applied on one part only. If two layers are placed, they should be connected with transverse ties. Simple jackets should be connected to the masonry by chases.

(3) Steel profiles may be used in a similar way, provided they are appropriately connected to both faces of the wall or on one part only.