

Eurocode 8: Design provisions for earthquake resistance of structures —

Part 1.4: General rules — Strengthening and repair of buildings

ICS 91.040:91.120.20

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Committees responsible for this Draft for Development

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

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

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National foreword

This Draft for Development has been prepared by Subcommittee B/525/8 and is the English language version of ENV 1998-1.4:1996 *Eurocode 8: Design provisions for earthquake resistance of structures — Part 1-4: General rules — Strengthening and repair of buildings* published by the European Committee for Standardization (CEN).

ENV 1998-1.4:1996 results from a programme of work sponsored by the European Commission to make available a common set of rules for the structural and geotechnical design of buildings and civil engineering works. The full range of codes covers the basis of design and actions, the design of structures in concrete, steel composite construction, timber, masonry and aluminium alloy, and also geotechnical and seismic design.

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

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

There is no existing British Standard equivalent to ENV 1998-1.4 and there is no requirement in the Building Regulations to consider seismic actions on buildings and civil engineering works in the UK.

ENV 1998-1.1 states that the provisions of Eurocode 8 may be simplified or not observed in regions of low seismicity. Within the UK, the application of Eurocode 8 should not be necessary, unless the client or user of the works assesses that the associated seismic risk is such that it needs to be addressed.

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

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

Summary of pages

This document comprises a front cover, an inside front cover, pages i and ii, the ENV title page, pages 2 to 71 and a back cover.

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

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Descriptors: Civil engineering, buildings, structures, earthquake resistant structures, earthquake resistant design, reinforcement, repair, computation

English version

**Eurocode 8 —
Design provisions for earthquake resistance of structures —
Part 1-4: General rules —
Strengthening and repair of buildings**

Eurocode 8 — Conception et dimensionnement
des structures pour la résistance aux séismes —
Partie 1-4: Règles générales — Renforcement et
réparation des bâtiments

Eurocode 8 — Auslegung von Bauwerken gegen
Erdbeben —
Teil 1-4: Grundlagen — Verstärkung und
Reparatur von Hochbauten

This European Prestandard (ENV) was approved by CEN on 1995-06-29 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into an European Standard (EN).

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

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

CEN

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

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

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Foreword

Objectives of the Eurocodes

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

Background of the Eurocode Programme

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

Eurocode Programme

- 7) Work is in hand on the following Structural Eurocodes, each generally consisting of a number of parts:

EN 1991: *Eurocode 1 Basis of design and actions on structures.*

EN 1992: *Eurocode 2 Design of concrete structures.*

EN 1993: *Eurocode 3 Design of steel structures*

EN 1994: *Eurocode 4 Design of composite steel and concrete structures.*

EN 1995: *Eurocode 5 Design of timber structures.*

EN 1996: *Eurocode 6 Design of masonry structures.*

EN 1997: *Eurocode 7 Geotechnical design.*

EN 1998: *Eurocode 8 Design provisions for earthquake resistance of structures.*

EN 1999: *Eurocode 9 Design of aluminium alloy structures.*

- 8) Separate sub-committees have been formed by CEN/TC250 for the various Eurocodes listed above.

- 9) This Prestandard is being published as a European Prestandard (ENV) with an initial life of three years.

- 10) This Prestandard is intended for experimental application and for the submission of comments.

- 11) After approximately two years CEN members will be invited to submit formal comments to be taken into account in determining future actions.

- 12) Meanwhile feedback and comments on this Prestandard should be sent to the Secretariat of CEN/TC250/SC8 at the following address:

IPQ c/o LNEC

Avenida do Brasil 101

P — 1799 LISBOA Codex

PORTUGAL

or to your national standards organisation.

National Application Documents (NAD's)

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

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

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

Matters specific to this Prestandard

- 16) The scope of Eurocode 8 is defined in 1.1.1 of ENV 1998 1-1:1994 and the scope of this Prestandard is defined in 1.1. Additional parts of Eurocode 8 which are planned are indicated in 1.1.3 of ENV 1998-1.1:1994.

17) 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.
- The occurrence of earthquakes may create the need for important repairs.

18) 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.

19) Interventions into the foundation, if justified, require special care. This Prestandard does not fully cover the repair and strengthening of foundations, for which the advice of specialists is usually required. It is intended during the ENV period of this Prestandard to prepare a new Annex devoted to this aspect.

20) In strengthening and repair interventions, as in the design of new structures, an optimisation is sought between the initial costs and the expected decrease of future costs. Since the marginal costs of increasing the structural resistance or ductility may be higher in strengthening and repair interventions, it may be acceptable under some conditions to use a reduced seismic action for redesign purposes [see 3.2 2)].

21) Modelling of damaged/repared or strengthened regions (discontinuous and multi-material) is more difficult and less reliable than in the case of the design of new structures. Consequently, this Prestandard, contains mainly basic information aiming at setting a rational framework in a field where empiricism may result in unsafe or uneconomical solutions.

22) In the Informative Annexes, under some well specified conditions (and for each of the main structural materials, separately), selected “deemed to satisfy rules” are also included. Procedures are also given for the more common intervention techniques.

23) In strengthening and repair situations, qualitative verifications for the identification and elimination of major structural defects may be very important and should not be discouraged by the more quantitative approach for which this Part of Eurocode 8 was developed. Although beyond the scope of this Prestandard, it is foreseeable that, in some particular situations, it will be possible to derive therefrom specific Recommendations for redesign in which qualitative aspects predominate.

24) Since repair and strengthening under non-seismic conditions is not yet covered by the relevant material-dependent Eurocodes, this Prestandard treats this topic in a broader way; besides, a structural intervention against seismic actions, has to deal simultaneously with all the other actions as well. However, structural interventions against specific pathological causes such as lack of durability of materials are not covered in this Prestandard.

25) This Prestandard addresses the structural aspects of repair and strengthening, which is only one component of a broader strategy for seismic risk mitigation which includes pre and/or post-earthquake steps to be taken by several responsible agencies, in Annex A the inter-relationships between those steps is presented.

26) With reference to section 2 of ENV 1998-1.1:1994, only the non-collapse requirement is sought in redesigning existing structures; however, satisfaction of the damage limitation requirement should be sought whenever possible and economical, and whenever safety consequences may be expected (e.g. falling down of nonstructural elements, etc.).

27) In low seismicity zones, this Prestandard should be adapted to local conditions by appropriate Recommendations.

28) The provisions of this Prestandard assume that the engineer responsible for the design work has suitable professional qualifications, as well as appropriate experience of the type of structures being strengthened or repaired.

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1 General

1.1 Scope

1) P Reflecting the basic requirements of ENV 1998-1.1, ENV 1998-1.2 and ENV 1998-1.3, this Prestandard covers the repair and strengthening of buildings and, where applicable, monuments, considering commonly used structural materials (concrete, steel, masonry and timber).

2) P Its scope is the following:

- To provide criteria for the evaluation of the seismic performance of existing individual structures.¹⁾

- 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). Corrosion damaged buildings are not covered by this Prestandard.

NOTE This Prestandard may be taken as a basis for the structural evaluation, repair or strengthening of buildings in non-seismic area as well.

3) P Although the provisions of this Prestandard are applicable to all categories of buildings, the repair or strengthening of monuments and historical buildings normally needs additional provisions (and sometimes alternative formats). The structural solutions, methods and procedures, as well as the extent of their application have to be compatible with the value of the monuments. Informative Annex F deals specifically with additional considerations for monuments and historical buildings.

4) P 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 unknown previous earthquakes, structural evaluation and eventual structural intervention are bound to an exceptionally large amount of uncertainties, differing from Country to Country.

1.2 Distinction between principles and application rules

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

2) P The Principles comprise:

- general statements and definitions for which there is no alternative,

- requirements and analytical models for which no alternative is permitted unless specifically stated.

3) P The Application Rules are generally recognised rules which follow the Principles and satisfy their requirements.

4) The Principles are identified by the letter P, following the paragraph number the other items (without P) are Application Rules, e.g. as this paragraph.

5) It is permissible to use alternative design rules which differ from the Application Rules given in this Prestandard, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the safety and serviceability achieved for the structures designed according to the Application rules of this Prestandard.

1.3 Assumptions

1) Reference is made to 1.3 of ENV 1998-1.1:1994

1.4 Definitions

1) Reference is made to 1.4 of ENV 1998-1.1:1994

1.5 S.I. Units

1) Reference is made to 1.5 of ENV 1998-1.1:1994

1.6 Symbols

1) Reference is made to 1.6.1 1) of ENV 1998-1.1:1994

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

¹⁾ Methods specified in this Prestandard (and especially the vulnerability method in Annex D) may be directly, or after further simplification, used in assessing urban areas in general. For individual buildings in such an urban area requiring repair or strengthening, the relevant clauses of this Prestandard are applicable.

2 Information for structural assessment

2.1 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 public records, relevant information, field investigations and, in most cases, from in-situ and/or laboratory measurements and tests.
- 2) P The data collection and tests shall performed by qualified personnel.
- 3) P Cross-examination of the results of each data-source shall be performed to minimise uncertainties.
- 4) Inspection procedures, check-lists and other data-collection procedures, should follow the recommendations of pertinent professional organisations and should reflect the availability of local resources for inspection, investigation and repair/strengthening measures.

NOTE In the absence of more specific recommendations, the methodology summarised in Annex B may be followed.

2.2 Minimum input data

- 1) In all cases, the information for structural evaluation should cover the following. However, such comprehensive information could be very difficult to establish for traditional buildings, and may not be obtainable. In such cases, appropriate allowance must be made for the consequent uncertainty.
 - a) Identification of the structural system and its categorisation within the structural types considered in ENV 1998-1.1, and ENV 1998-1.2 and ENV 1998-1.3 (regularity and, if possible, ductility class included).
 - b) Information on structural changes since construction, likely altering the structural behaviour.
 - c) Identification of the subsoil conditions, as categorised in 3.2 of ENV 1998-1.1:1994.
 - d) Identification of the type of building foundations.
 - e) Identification of the exposure class regarding environmental influence.
 - f) Information about the overall dimensions and cross-sectional properties of the building elements and the quality and condition of constituent materials.

g) Description of the actual and/or the planned use of the building (with identification of its importance category, as described in 3.7 of ENV 1998-1.2:1994).

h) Evaluation of the potential live loads by considering the actual use of the various spaces.

i) Information on the quality of existing materials, expressed in quantitative terms where possible.

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

k) Information about identifiable gross errors in structural concept, material defects and detailing.

3 Evaluation

3.1 General

- 1) Evaluation consists of the verification of the seismic resistance of an existing damaged or undamaged building, taking into account both non-seismic and seismic actions, for the period of its intended lifetime.
- 2) Within the scope of this Prestandard, evaluation is made for individual buildings, but its use may be twofold:
 - a) For decisions related to the need for structural intervention in a building and for the design of its strengthening or repair measures.
 - b) For decisions related to large scale retrofitting of urban areas for mitigation of seismic risk; to this end, the evaluation-of selected representative buildings should be used.
- 3) P Depending on the importance of a building and the extent of its possible damages, as well as on the available information, time and resources, the evaluation shall be made (see 3.5) by means of one of the following methods:
 - a) General methods derived from ENV 1998-1.2:1994, using linear or nonlinear analysis (see 3.5.2)
 - b) Approximate nonlinear methods specifically adapted to masonry (see. 3.5.3).
- 4) In all cases, the method used should be calibrated considering information of the observed behaviour of the same type of building or similar buildings during the considered or previous earthquakes (see also Annex B 1c).

5) Analysis and redesign of existing structures may be based on appropriately modified actions and possibly modified safety-factors (in comparison with the design of new structures) in order to account for smaller remaining life times, smaller uncertainty with respect to dead loads, and for properties of existing materials.

6) In case of heavily damaged simple buildings, urgent and obvious repair may be implemented without being preceded by a detailed evaluation, following the conditions of 3.6.2 2).

7) Vulnerability evaluation methods for individual buildings may be used only under well specified conditions and for limited purposes. Annex D gives more detailed information on such methods, mainly applicable to populations of buildings.

3.2 Modified seismic action. Non-seismic actions

1) P The basic models for the definition of the seismic motion are those presented in 4.2 and 4.3 of ENV 1998-1.1:1994.

2) The effective peak ground acceleration a_g defined for general purposes in the various seismic zones (see 4.1 of ENV 1998-1.1:1994) may be reduced for redesign purposes, taking into account:²⁾

a) The admissibility to take the design remaining life of certain categories of buildings, not belonging to category I (3.7 of ENV 1998-1.2:1994), less than the design life for new buildings, provided that these buildings shall be demolished or fully strengthened at the end of the reduced life time period (using a_g as specified for the corresponding seismic zone). Appropriate records should be kept to ensure that these actions are taken at the end of the assumed design life.

b) A possibly higher value of the acceptable probability of exceedance (see 4.1 of ENV 1998-1.1:1994) in order to optimise broader social, economic and/or historical goals.

NOTE E.g. in the case that the anticipated total costs (of strengthening the entire building inventory of urban areas) sharply increase if a_g -values would be raised towards the code required level; or in the case that code required a_g -values for designing the repair or strengthening of a monument, would lead to completely unacceptable architectural alterations.

3) P The values of non-seismic actions entering the seismic combination (section 4.4, of ENV 1998-1.1:1994) shall not be reduced.

3.3 Modified material strengths and partial safety factors related to resistance

1) Design values based on the characteristic strengths f_k established in the relevant Eurocodes may be used under the following conditions:

- a) initial design and construction documents are available and reliable, and
- b) there are no indications of material strength inadequacy or alteration, and
- c) results of in-situ tests are available, confirming the previous assumptions.

2) P When one of these conditions is not met, adequate assessment of strengths shall be made, either by testing, or by reference to the materials' standard specifications in force at the time of construction. If testing is resorted to, the design values $f_d = f_k/\gamma_M$ to be used in the analyses shall be established based on characteristic values estimated as prescribed in the following paragraphs.

3) P The value of f_k shall be estimated on the basis of the 5 % fractile of the test results. Appropriately calibrated non destructive methods may be used in order to have a required number of tests.

4) In estimating f_k , the characteristic values of strengths should be evaluated from homogeneous populations, corresponding to critical regions of similar building elements (e.g. in case of reinforced concrete, all areas of beams may be considered together; separately the upper parts of columns or walls, etc.).

5) For the computation of f_k values, from the population of in-situ determined strength values, the effects of the additional variability inherent to the experimental method used, may be compensated for.

NOTE Taking into account the difference between laboratory test results and in-situ strength values, the values of γ_M used in design may, subject to additional considerations be lower than used typically in design new construction.

6) The variability inherent to a given experimental method may be estimated from data in the literature or by means of measurements carried out on a given testing area of high uniformity.

²⁾ Annex C describes further the possible basis and the rationales of such reduction.

7) If a sufficiently representative knowledge of the actual in-situ strengths is available, and the cross-section geometry is checked at a similar number of positions to the number of in-situ tests, the γ_M values used may be reduced with regard to those specified for the design of new structures.

8) Nominal values of material strengths, may be used as design strengths in case of non-engineered old structures built with particularly non-homogenous materials (e.g. old masonry). These values or the conditions for its assessment should be based on previous experience or determined by means of available in-situ strength measurements.

9) P Whenever the analytical procedures described in 3.5.2 2) are followed, higher γ_{Rd} factors³⁾ shall be used accounting for the inadequacy or uncertainties of the respective models used for computing the structural elements' resistance (e.g. see Informative Annex G).

3.4 Structural data and combination of actions

1) Section 3 of ENV 1998-1.2:1994 applies, but with the actual values replacing the nominal values.

3.4.1 Estimation of the actual structural characteristics of building elements before intervention

1) The effects of the seismic action to be considered in the analysis of existing structures, should be calculated taking also into account the possibly modified stiffness characteristics of the relevant structural elements. Similarly, any possible modification of the available ductility (because of previous earthquakes or environmental actions) should be taken into account when selecting the appropriate, possibly modified, behaviour factor. To this end, possible previous damage and structural interventions (if any) should be considered in order to get the best possible estimation of the natural vibration period of the structure. Occasionally, some parametric studies may quantify the consequences of existing uncertainties, but they are not required in routine situations.

2) On the basis of the documentation collected (see section 2), the following classification of the building should be made:

a) The Structural Type of the building should be identified, in particular whether it can be identified as a type satisfying ENV 1998-1.3, or, failing this, whether it satisfies Eurocodes 2 to 7 (for which nominal ductility may be assumed).

b) Regularity Level should be attributed to the building, according to ENV 1998-1.2 and ENV 1998-1.3.

c) For reinforced concrete buildings which satisfy ENV 1998-1.3, a ductility class should be assessed.

3) Subsequently, the following characteristics should be evaluated:

a) Structural characteristics of building and elements, i.e. fundamental period of vibration of the building, actual stiffness of elements and actual resistance (against axial and shear action-effects).

These structural characteristics should be calculated taking into account the actual dimensions and positions of members and quality of materials (see also 3.3).

Alternatively, ambient vibrations can be performed to determine the building's vibration periods.

For undamaged elements, the vibration periods calculations shall be made as prescribed in the relevant Eurocodes. The structural characteristics of damaged elements should be calculated on the basis of appropriate models, accounting for the actual discontinuities and material properties. The models of load transfer described in 5.3 are also valid for estimating the residual structural characteristics of the elements.

b) Alternatively, it is allowed to estimate the residual structural characteristics of damaged elements by using appropriate reductions of the values calculated under the assumption of non-damaged conditions, taking into account the type and the extent of damage. To this end experimental findings under comparable conditions may be used, as well as acceptable empirical rules.

4) P In all cases, an evaluation of the in-plane stiffness of floors (diaphragm stiffness) shall be made and appropriately taken into account in the structural analysis (see 3.5).

³⁾ In case of damaged structures, these factors account for the inadequacy of models (which are basically conceived for non damaged conditions). In case of undamaged structures, these factors account mainly for possible deficiency in ductility related detailing; thus, in selecting γ_{Rd} values, the provisions of the Code at the time of construction shall be considered.

3.4.2 Combination of the seismic action

- 1) P The combination of the seismic action with other actions shall be made according to the general combination rule presented in 4.4 of ENV 1998-1.1:1994.
- 2) P If subsoil or foundation deficiencies are expected under seismic conditions, possible seismically induced differential settlements shall also be taken into account in the analysis.

3.5 Analysis (prior to intervention)

3.5.1 General

- 1) Depending on the importance of the building and the available information, time and resources, the analysis may be made by the methods specified in 3.1 3).
- 2) In the analyses (3.5.2 and 3.5.3), a model uncertainty factor γ_{sd} may be introduced, covering the additional uncertainties related to the analysis of the pertinent structure; higher values should be used for higher damage levels, for which a sensitivity analysis has not been carried out.

3.5.2 General methods

- 1) P The analysis shall be based on the actual structural characteristics, as estimated in 3.4, and on the seismic action as derived in 3.2.
- 2) One of the following methods may be used in order to calculate the design action-effects $E_{new,d}$ under the actual conditions of the structure:
 - a) Standard method
The analysis is carried out according to ENV 1998-1.1 and ENV 1998-1.2, with the actual structural characteristics. To this end, an appropriate behaviour-factor should be selected, as in ENV 1998-1.3, based on the structural type, the regularity class and the ductility level assigned to the building. The design action-effects $E_{new,d}$ in each critical region of the building, will be calculated by means of an appropriate structural analysis, according to section 3 ENV 1998-1.2:1994, accounting for the degree of base fixity and for the integrity of joints and connections.

b) Time-domain dynamic non-linear analysis

In exceptional cases, a more sophisticated computational method may be used, especially for reinforced concrete and steel structures:

- i) The hysteretic behaviour of the critical regions of the structure has to be modelled appropriately. Reliable limit-values of post-yield deformations need to be defined so that the calculated post-elastic deformations may be checked.
Corresponding material and/or element constitutive laws, as well as critical strain values, may be taken from the literature and pertinent Parts of Eurocode 8.
- ii) Accelerograms compatible with the elastic response spectrum need to be used, in accordance with 4.2 of ENV 1998-1.1:1994.
- iii) In-time integrations for the structural response of the entire structure are carried out, with second order effects modelled adequately at floor levels.
- iv) In critical regions, checks need to be made, comparing deformation demands with design values of deformational capacity.
- v) Where possible, account should be taken of strain hardening, strain softening and cumulative inelastic strains or other cumulative damage effects.
- vi) The validity of the conclusions shall be checked by means of a sensitivity analysis. Hence the process should be repeated with appropriately modified constitutive laws (elastic and post elastic). The initial assumptions made may be rejected if results from subsequent analyses are considerably different; indicatively, if the calculated design action-effects are less than 60 % of those calculated by the standard methods, the results should be rejected, unless adequate experimental evidence is available.

3.5.3 Approximate static non-linear methods

1) Where appropriate, e.g. plain masonry buildings, the analysis may be carried out alternatively by using static non-linear methods, based on non-linear force-deflection relationships of all structural members resisting the lateral loads. A step-by-step procedure is followed, using decreased stiffness values under increasing lateral loads. A conservative elastoplastic behaviour of individual elements should be considered, provided that the relevant material properties and structural configuration permits. The kinematic post-yield follow-up of separated rigid masonry elements up to the necessary equilibrium of the global seismic force, may be a method belonging to this category.

2) In some cases, the use of approximate methods requires an extensive calibration against results obtained by the general methods (3.5.2).

b) The level of damage predicted for both structural and non-structural elements is kept within acceptable limits (see 2.2.2, ENV 1998-1.1:1994) as defined on a case by case basis.

4) If the approximate method (3.5.3) is used, an approximate overall evaluation of the building may be made by comparing the maximum lateral resistance (of the entire structure or of each storey) and the expected seismic forces corresponding to the maximum value of the elastic response spectrum.

3.6.2 Vulnerability evaluation methods

1) In case a global evaluation is needed rapidly, special assessment techniques may be used, by means of which the vulnerability of a building is evaluated. These methods should be used under well specified conditions regarding construction materials, techniques and workmanship and should include the decision-making procedures. The following conditions should be fulfilled by these methods:

- a) Be based on recognised experience
- b) Refer to well defined types and configurations of structures, construction techniques and materials
- c) Take into account the age of the building, so that in addition to possible durability effects, the philosophy of the Codes at the time of construction is accounted for, and
- d) The effects of the structural irregularities in configuration, stiffness and mass distribution, as well as, overstrengths distributions, are taken into account.

2) Additionally, for damaged buildings, practical evaluation should be based on an objective description of the damage levels and modes of failure.

3) Annex D provides more detailed information on this category of methods.

3.6 Verification (final evaluation)

3.6.1 Computational verification

1) P If standard methods (3.5.2 2) a) are used for the analysis, the evaluation shall be based on the verification of all cross-sections by means of the inequality:

$$\gamma_{Sd} E_{new,d} \leq \frac{1}{\gamma_{Rd}} R_{new,d} \quad (3.1)$$

where

- γ_{Sd} as defined in 3.5.1 2)
- $E_{new,d}$ as per 3.5.2 2), accounting however for the capacity design as required in the relevant sections of ENV 1998-1.3.
- γ_{Rd} as defined in 3.3 9)
- $R_{new,d}$ the design resistance values of cross-sections of the structural elements, see 3.4.1 3).

2) This verification is also meant to be carried out on foundation elements, under the conditions specified in ENV 1998-5.

3) P If the Time Domain method (3.5.2 2) b) is used, code requirements are deemed to be satisfied if all of the following conditions are met:

- a) Available post-yield deformations are higher than the corresponding demand values; to a limited extent, exceptions are allowed, provided that the stability of the structure is not jeopardised.

4 Decisions for structural intervention

4.1 Post earthquake immediate measures

1) After a severe earthquake, certain immediate measures should be taken as soon as possible, in order to ensure the safety of the population and to minimise further losses. The nature and the extent of these measures should depend on the damage-level observed, and the possibility of imminent aftershock. Annex A contains an indicative list of such measures.

4.2 Criteria for a structural intervention

1) On the basis of the conclusions of the evaluation 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.

NOTE To this effect, several non-engineering requirements may influence the decisions, e.g. availability of funds, cultural and social importance as well as priorities related to a particular case, possibilities for relocating building's functions, (temporary or permanent), etc.

2) This Prestandard describes the technical aspects of the relevant criteria.

4.2.1 General criteria

1) P For the definition of the intervention measures, the following shall be considered:

- a) Costs, both initial and future (i.e. accounting for maintenance costs and possible future damage), versus the importance of the structure under consideration.
- b) Available workmanship (it is of fundamental importance that the measures should be feasible with the available workmanship and equipment).
- c) Availability of appropriate quality control.
- d) Occupancy (impact on the use of the building, both during and after the works).
- e) Aesthetics (the intervention policy may vary from a totally invisible solution to an intentionally identifiable new/additional structural scheme).
- f) Preservation of the architectural identity of historical buildings and consideration of the level of reversibility of intervention.
- g) Duration of the works.

4.2.2 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 inspection of the building.

2) The following guidelines 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.

c) If the low damageability requirement of ENV 1998-1.2 regarding the non-structural elements is not sufficiently fulfilled, appropriate intervention measures should be taken (e.g. stiffening, separation of vulnerable non-structural elements from load-bearing elements, etc). However, in case of disproportionately high costs or adverse structural implications, the criteria of ENV 1998-1.2 regarding limitations of damage, may be modified.

d) All resistance requirements of the relevant Eurocode and of ENV 1998-1.1 and ENV 1998-1.2 should be fulfilled after the intervention, taking however into account the special provisions of this Prestandard.

e) The minimum possible modification of the local stiffness should be sought, unless required otherwise by criteria a) and b) above.

f) Where possible, the increase of local ductility should be sought in critical areas. Care should be taken that, as far as possible, local repair and/or strengthening does not reduce the available ductility of critical areas.

g) The durability of both the original and new elements, as well as the possibility of accelerated deterioration when in contact with each other, should be taken into account.

4.2.3 Type of intervention

1) Observing the criteria of 4.2.1 and 4.2.2, an intervention may be selected from the following indicative types; one or more types in combination may be selected. In most cases, the effect of structural modifications on the foundation has to be considered:

- a) No intervention at all. The possibly acceptable reduced remaining life of the structure [see 3.2 2)] may be relevant in this respect.
- b) Restriction or change of use of the building.
- c) Local or overall modification of damaged or undamaged elements (repair or strengthening), considering their stiffness, strength and/or ductility.
- d) Possible transformation (or incorporation) of existing non-structural elements into structural elements (e.g. adding elastic and damping connectors between brittle infills and frame, when the resistance of these infills allows it).
- e) Modification of the structural system (elimination of some structural joints; elimination of vulnerable elements; modification into more regular and/or more ductile arrangements).

NOTE 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.

- f) Modification of the structural system aimed at a beneficial change of the natural period of the structure, may also be envisaged.
- g) Mass reduction.
- h) Addition of new structural elements (e.g. bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc).
- i) Full replacement of inadequate or heavily damaged elements.
- j) Redistribution of action-effects (e.g. by means of re-levelling of supports or by adding external prestressing).
- k) Addition of a new structural system to take the seismic action.
- l) Addition of local friction, global damping devices or active control at appropriate locations of the building.
- m) Base isolation.
- n) Partial or total demolition.
- o) Monolithic relocation of the entire structure.

4.2.4 Ductility considerations

- 1) P The value of the q factor for redesign, in addition to the criteria set forth in ENV 1998-1.3, shall be based on the final local ductilities after intervention.
- 2) It has to be taken into account that an increase in resistance of structural elements may frequently be accompanied by a reduction in ductility, unless specific measures are taken.⁴⁾

4.2.5 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 jeopardise 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 ENV 1998-1.2:1994)
 - b) increasing the resistance of non-structural elements (see 3.5 of ENV 1998-1.2:1994)
 - c) taking measures of retainment 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.

4.2.6 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.

4.3 Priority of interventions

- 1) The timing of intervention in a single building, depends on parameters such as:
 - a) available margins of safety
 - b) available funds
 - c) availability of appropriate technical means
 - d) importance of the structure under consideration
 - 2) Accounting for the local damage data a “priority index”, indicating the suggested priority as described in c) here below may be established, taking into account mainly safety reasons.
 - a) The existing structure should be verified in accordance with the criteria set forth in ENV 1998-1.1, ENV 1998-1.2, ENV 1998-1.3 and in this Prestandard, taking into account the possibility of modifying the seismic action as described in 3.2
- For normal structurally redundant buildings, local seismic resistance indexes L_{Ri} for each critical region may be computed, considering the various internal forces due to the relevant action-effects, namely.

$$L_{Ri} = \frac{R_{di} - E_{di} \text{ (N.S.A.)}}{E_{Ei}} \quad (4.1)$$

where

- R_d Design resistance according to the relevant chapter of Eurocode 8. Modified γ_m values, as discussed in 5.2 may be used.
- $E_d \text{ (N.S.A.)}$ Sum of the action effects due to all “non seismic actions” (N.S.A.) entering into the seismic combination as prescribed in ENV 1998-1.1.

⁴⁾ E.g. in reinforced concrete members, an increase in tensile reinforcement should possibly be accompanied by sufficient amount of compressive steel and confinement.

E_E	Effect of the design seismic action, possibly reduced as described in 3.2 2)
i	i th critical region and type of internal force

Considering the various internal forces, for each critical region, the minimum value of L_{Ri} should be selected.

b) The global seismic resistance of the existing structure is assessed by means of a global seismic resistance index G_R for each storey defined by the following expression:

$$G_R = \frac{\sum_i w_i \cdot (L_{Ri})^\lambda}{\sum_i w_i \cdot (1/L_{Ri})^\lambda} \quad (4.2)$$

where:

L_{Ri}	Local seismic resistance indices (expression 4.1) for each critical region i ; L_{Ri} values need not be put lower than zero or larger of 1,3.
w_i	weighing factor, accounting for the significance of the local region and their effect in case of failure on the global behaviour, expressed by the product $w = w_1 \times w_2$ in which: $w_1 = 1,0$ for brittle failures (due to shear, high axial load or axial load in absence of confinement) $w_1 = 0,5$ for ductile failures $w_2 = 1,5$ for foundations $w_2 = 1,0$ for columns and walls $w_2 = 0,5$ for beams
λ	an exponent intended to reduce the significance of local deficiencies on the overall resistance; its value may be taken between 1,0 (for structures with low redundancy) and 0,5 (for normal redundant structures)

Summation is extended over all structural elements i of the most critical storey.

c) A priority index I_p serving to assess the urgency of intervention within each seismic zone, is defined as:

$$I_p = (1 - G_R) \times I_0 \quad (4.3)$$

where

G_R	Global seismic resistance index as defined in 4.3 2) b)
I_0	Reference priority index, to be taken with the following values: $I_0 = 2,5$ for buildings with frequent large occupancies $I_0 = 1,5$ for schools, hospitals and buildings with rare large occupancies $I_0 = 1,0$ for apartment buildings and individual houses

5 Redesign of repair and/or strengthening

5.1 Redesign procedures

1) P After the selection of an intervention scheme (section 4), an appropriate verification of the design criteria (see 2.2 of ENV 1998-1.1:1994) shall be carried out using ultimate limit state procedures:

- At the connections of additional parts to existing structural parts
- At the critical cross-sections of the entire structure and the foundations.

2) The redesign process should cover the following steps:

- Conceptual design, i.e.
 - Selection of techniques and/or materials, as well as of the type and configuration of the intervention.
 - Preliminary estimation of dimensions of additional structural parts
 - Preliminary estimation of the modified stiffness of the repaired/strengthened elements.
 - Preliminary estimation of the appropriate ductility class and behaviour factor on the basis of the provisions of ENV 1998-1.3.
- Analysis, (see also 5.5) i.e.
 - Identification and quantification of the non-seismic actions.
 - Selection of the seismic action according to 3.2
 - Determination of the action-effects, taking into account the modified stiffnesses and using a method of analysis in accordance with 3.5.2.

- c) Verification, i.e.
 - i) Definition of the structural behaviour model of repaired/strengthened elements or connections.
 - ii) Selection of γ_m factors (see 5.2)
 - iii) Calculation of capacity design values of action-effects wherever needed (3.6.1)
 - iv) Calculation of design resistance values (see 3.6.1)
 - v) Verification of the safety inequalities regarding seismic and non-seismic actions
 - vi) Confirmation of the overall behaviour factor initially selected, as a function of the ductility level achieved; to this end, qualitatively the relevant rules included in ENV 1998- 1-3 should be used.
- 3) Under well defined conditions, simplified redesign procedures may be followed for simple buildings to be repaired or strengthened against future seismic events. These procedures should be established in special documents ("Recommendations"), where the definition of "simple" buildings is presented and the simplified rules are deemed to satisfy the requirements of this Prestandard.

5.2 Revised γ_m factors

- 1) P Properties of existing materials shall be divided by γ_m factors as described in 3.3.
- 2) P The strengths of additional materials connected to the existing structural elements in repair or strengthening procedures, shall be divided by increased γ_m factors (γ'_m) in recognition of possible reduced resistance in reconstruction operations (e.g. by welding) and due to limitations of in-situ quality control effectiveness.

5.3 Basic data for force transfer

- 1) P The structural characteristics of damaged as well as of repaired and/or strengthened building elements (in particular their critical regions) shall be conservatively estimated on the basis of the data of section 2, and 3.3, taking into account the force transfer mechanisms along the interfaces between existing-to-existing or between existing-to-additional components, as described in 5.3.1 to 5.3.9.
- 2) In every case, the interaction (and corresponding possible resistance reduction) between these mechanisms shall be taken into account.

5.3.1 Compression against pre cracked interfaces

- 1) Reloading after tension cracking may cause compressive forces before full recovery of previous extensional deformations; inelastic distortions along the crack, and crack debris inside the crack prevent crack to close completely.
- 2) It is allowed to account for this phenomenon by means of an appropriate model. However, such a model shall conservatively account for cyclic reloading, which tends to minimise such an early compressive resistance of "open" cracks.
- 3) A conservative simplification consists in disregarding compressive stresses before full closure of previous extensional cracks.

5.3.2 Adhesion between non-metallic materials

- 1) "Shear adhesion versus slip" relationships between existing and new materials may be accounted for by means of appropriate models, taking into account the effects of curing and the characteristics of possible bonding agents.
- 2) Slip may be disregarded if shear adhesion stresses are below conservative estimates of the long term shear adhesion-resistance.

5.3.3 Friction between non-metallic materials

- 1) Friction resistance may be accounted for as a function of the relative displacement (slip) along a discontinuity or the interface. A constitutive law should be used for this purpose, valid for the relevant normal stress (σ) acting across the interface.
- 2) In case the slip needed to mobilise the maximum friction resistance (τ_u) is relatively low, a friction coefficient $\mu = \tau_u/\sigma$ may be used. However, for low σ values, the strong dependence of μ on σ values should be appropriately taken into account.
- 3) P Whenever a cyclic reversal of the relative displacements is expected, the corresponding degradation of the friction-resistance shall be considered.

5.3.4 Load transfer through resin layers

- 1) The design tensile resistance of a joint (i.e. the contact interface) between a resin layer and a given material may be taken equal to the tensile resistance of the weakest of the two divided by γ_m (at least equal to [1,5]), or the tensile resistance of the interface (divided by [2,0]), whatever is lower.
- 2) P The influence of the thickness of the resin layer, as well as the conditions of surface preparation shall be taken into account appropriately in evaluating the tensile resistance.

3) The local shear force response along such a joint is a function of the local slip and the normal stress acting on the considered area.

5.3.5 Clamping effect of steel across interfaces

1) P If large relative displacements along the interface are not expected or cannot be tolerated, the friction resistance shall be evaluated considering the compatibility of displacements on both faces or the interface.

2) P The friction mobilised across a sheared interface transversely reinforced with well anchored steel bars, may, in case of expected large relative displacements along the interface, be evaluated as follows:

$$\tau_R = \mu \cdot (\rho f_{sy} + \sigma_0) < \tau_{u,m} \quad (5.1)$$

where

μ	denotes the friction coefficient available under normal stress $\sigma_{tot} = \rho f_{sy} + \sigma_0$
ρ	steel ratio across the interface
f_{sy}	yield strength of steel
σ_0	external normal stress across the interface
$\tau_{u,m}$	the shear resistance of the material itself

In applying this model, appropriate design-values shall be used for μ , f_{sy} and $\tau_{u,m}$.

5.3.6 Dowel action

1) P The design value of the maximum shear force which may be transferred by a bar crossing an interface (dowel action) shall be calculated appropriately, taking into account the strength and deformability of both the dowel and the connected material, and the spacing between dowels and their distance to the edges of the cross-section.

5.3.7 Anchoring of new reinforcement

1) P Anchorage over the length of critical regions (i.e. potential plastic hinges) shall be avoided.

2) P Anchorage lengths of steel bars in concrete or in masonry shall follow the criteria of Eurocode 2 and Eurocode 6, respectively, provided that the shear-bond resistance of the accommodating layer is sufficient. Due to cyclic effects and related uncertainties, in high seismicity areas, it is not allowed to rely entirely on bond for the anchorage of new reinforcement. Appropriate mechanical means (such as end plates, dowels, etc.) are required to ensure an anchorage force resistance of F_{am} as:

$$F_{am} > \max \left(F_a - \frac{2}{3} F_b, F_a / 2 \right) \quad (5.2)$$

where

F_a	required total anchorage force
F_b	anchorage force resisted by bond resistance

3) P Anchorage of additional steel bars by welding onto existing bars (directly or via additional welded spacers) or on appropriately anchored fixing elements may be considered as rigid; weldability of existing and added steel elements shall be established. It shall be verified that the bond ensured by the existing bar is sufficient to resist the total force acting on both the existing and added bars.

4) The force-slip relationship for the pullout of embedded bars or fixing elements may be predicted by means of appropriate models or empirical formulae, accounting for the bar's diameter, the concrete cover, the maximum local bond strength, the embedment length and the yield strength of the steel.

5) If full anchorage of new reinforcement is not feasible, its strength should be appropriately reduced, and the consequences of this defect on the local ductility of the structural element should be taken into account.

5.3.8 Welding of steel elements

1) In designing steel-to-steel connections through welding, not only the weld resistance but also the stiffness of the connection should be considered:

- Direct welding of additional longitudinal bars or shapes onto existing ones, ensures a complete force transfer with almost zero displacement.
- In case the force transfer is achieved via intermediate elements, the stiffness of such elements may reduce the overall stiffness of the strengthened element(s) and should be considered in the overall force-transfer distributions.
- Flexural behaviour of eccentrically welded structural steel elements or bars should be evaluated, accounting for contacts or restraining effects to existing or added elements. Simplifications should be based on adequate experience or experimental evidence.
- Welding should be avoided on highly stressed steel bars or elements.

5.3.9 Connections of timber-to-timber elements

1) P In evaluating the ultimate resistance of these connections (nailing, doweling, bolting, wedging) the force-transfer between existing and added timber elements shall be estimated conservatively, taking into account displacements (in the connections) due to the local deformability of wood. Durability conditions shall also be accounted for. Simplifications shall be based on experience or experimental evidence.

5.4 Local and global ductility

5.4.1 General

1) P The repair/strengthening intervention should maximise the energy dissipation capacity of the structure, within the cost constraints of the project.

2) P Local brittleness or violations of capacity design provisions due to the intervention, when unavoidable, shall be taken into account by an appropriate reduction of the behaviour factor used in the re-analysis of the structure; this is not needed, if action effects due to the design seismic combination, taking $q = 1$, do not exceed the resistance of the particular member.

3) The following aspects should be considered:

a) Local ductility:

Repaired or strengthened regions should be designed to reflect the selected ductility class, unless these regions are clearly outside the critical regions where inelastic behaviour is expected.

b) Structural regularity:

Abrupt modifications of local strength and stiffness of building elements should be avoided, unless such a modification contributes to an improvement of the regularity of the strengthened/repared structure.

The overstrength and the interstorey drift of consecutive floors should be kept as constant as possible over the height of the building. To this end, the strengthening of a building element should extend beyond the level strictly needed for resistance purposes, thus avoiding creating a soft-storey effect.

The seismic resistance indices [as defined in 4.3 2) a) of critical regions after the intervention should be kept as constant as possible, both in plan and in elevation.

c) Distribution of areas of potential inelastic behaviour:

The areas of expected inelasticity should not be concentrated in only one storey; whenever possible, appropriate measures should be taken so that inelastic areas are well distributed over the entire structure.

5.4.2 Modified behaviour factors

1) P The energy-dissipating capacity of the repaired/strengthened structure shall be considered when defining the behaviour factor to be used in the re-analysis. The evaluation of this factor shall be based on the criteria of ENV 1998-1.1 and ENV 1998-1.3 and the degree in which the requirements of 5.4.1 affect the global ductility.

2) Available experimental evidence may also be used to this end.

5.5 Analysis (determination of action-effects) after the intervention

1) P The method of analysis shall fulfil the condition of 3.5.2.

2) In this respect the following should be considered:

a) Repair and/or strengthening is applied to an already stressed structure; this fact has to be duly considered in the analysis. Possible unfavourable redistribution of action-effects due to heavy damage should be considered, both in terms of available resistance or ductility; these effects may be neglected in case of low damage levels.

b) Possible force effects imposed by temporary shoring, bracing, tying, etc., should be checked in order to avoid overstress or residual deformations (which may produce unforeseen redistribution of final action-effects).

c) The effects of actions likely to appear after completion of the structural intervention should be determined, taking into account the change in local stiffnesses due to the intervention.

d) In the re-analysis, the consequences of local stiffness changes on the global stiffness of the building elements may be estimated by means of simplified methods (e.g. an estimated local reduction of stiffness, etc., see 5.6).

e) Considering horizontal loading, the in-plane stiffness and strength of the diaphragms should be checked whenever the intervention creates very stiff vertical elements.

f) The distribution of new (gravity or live) loads between pre-existing and additional structural elements, as well as the redistribution of existing gravity loads due to possible creep, should be appropriately estimated.

5.6 Post intervention stiffnesses and resistances

5.6.1 General

1) P The structural characteristics (resistance, deflections, crack widths) of repaired or strengthened building elements, used in verification of the Ultimate Limit State and of the Serviceability Limit State, shall be evaluated taking into account load-transfer mechanisms, as described in 5.3, as well as the:

- a) residual resistances of the existing materials (see 3.4.1)
- b) additional resistances due to the connection of the new materials to the existing ones (see 5.3).
- c) possible positive influence of the intervention on the residual structural characteristics of the building element in addition to the main role of intervention.

2) P Appropriate factors for model-uncertainties shall be used [see 5.7 3)].

3) Depending on the reliability of the available data and the importance of the structure, two approaches for the evaluation of resistances may be followed, namely:

- a) analytical estimation of resistances, based on physical models
- b) simplified estimation of resistances, based on practical rules.

5.6.2 Analytical estimation of stiffness and resistance

1) P Analytical models shall be based on constitutive laws describing the force/deformation characteristics of all connected materials. For each force transfer mechanism, the value of the force mobilised is calculated for the global deformational behaviour of the critical region. The sum of all mobilised forces under compatible deformations may be accepted, under the following conditions:

- a) Conservative constitutive laws are considered, accounting for the response degradation due to cyclic post-yield deformations. The softening branch of these laws shall be considered to an extent consistent with the overall ductility demand assumed in the analysis.

b) When appropriate structural measures have been taken (e.g. adequate confinement in the case of masonry or reinforced concrete elements) the constituent materials may be represented by their monotonic constitutive laws. However, this is not the case with connections between materials where, because of cyclic degradation, considerably conservative strength and deformational values need to be considered.

c) Possible interactions between individual force-transfer mechanisms (e.g. pullout bond and dowel action) are also to be considered.

2) Simplifications based on evidence from parametric studies or experiments may also be introduced in these calculations.

5.6.3 Simplified estimation of stiffness and resistance

1) For simple buildings [see 5.1 4)] and under limited conditions, it is allowed to use simplified behaviour models.

2) When estimating residual characteristics after damage, global “correction factors” r_K for stiffnesses and r_R for resistances may be applied, depending on the damage level assessed. Thus, the residual resistance R_{res} of a damaged structural element may be estimated as

$$R_{res} = r_R \cdot R_0 \quad (5.3)$$

where R_0 denotes its original resistance before damage. Similarly, for residual stiffness K_{res} of a damaged element

$$K_{res} = r_K \cdot K_0 \quad (5.4)$$

where K_0 denotes its original stiffness before damage.

3) When estimating added resistances due to additional material (repair or strengthening), partial correction factors may be applied accounting for the different degree of mobilisation of forces in each contributing transfer mechanism.

NOTE E.g. along a concrete-to-concrete or timber-to-timber interface crossed by anchored steel connectors, the overall shear resistance may be expressed as:

$$V_R = \delta_1 V_d + \delta_2 V_f$$

where

V_R	overall shear resistance
V_d	maximum dowell resistance
V_f	maximum frictional resistance under maximum clamping
δ_1, δ_2	partial correction factors. The values of δ factors depend on the final compatibility of deformations.

4) When estimating the resistance R and stiffness K of a repaired/strengthened region, the respective structural characteristics R_{mon} , K_{mon} of a supposedly “monolithic” region may be first calculated, disregarding existing discontinuities or interfaces. Subsequently, appropriate “model correction factors”

k_r	for resistances
k_k	for stiffnesses

may be used, in order to account (in an empirical way) for the effects of discontinuities or interfaces disregarded in the previous step. Since interfaces between existing and additional materials produce lower initial stiffness and resistance values than for a supposed “monolithic” region,⁵⁾

$$k_k \leq 1 \text{ and } k_r \leq 1 \quad (5.5)$$

Thus, the final characteristics of the repaired/strengthened region may be estimated as:

$$R_{res} = k_r \cdot R_{mon} \quad (5.6)$$

$$\text{and } K_{res} = k_k \cdot K_{mon} \quad (5.7)$$

5) The values of aforementioned factors r , δ and k to be used in evaluation or in redesign should be conservatively chosen, taking into account available technical literature and local experience.

6) When carrying out these simplified estimates, account should be taken of the performance of similar structures in previous, earthquakes.

5.7 Verification

1) P The safety verification shall be carried out in accordance with section 4 of ENV 1998-1.2:1994.

2) P Moreover, the safety verification of the relevant connections between existing and added materials shall be carried out.

3) P The design resistance of the critical regions to be used in the verification are computed as given in 5.6, taking into account the uncertainties of the relevant models. To this end appropriate uncertainty factors shall be used, their values depending on the nature and sensitivity of materials and intervention techniques, as well as on the quality assurance schemes applied.

4) In the specific case of buildings evaluated under the conditions specified in 3.1 6) and 3.1 7), verification is deemed to be satisfied if the practical structural measures for repair or strengthening observe the corresponding provisions of recommendations based in local experience (“qualitative approach”).

⁵⁾ Normally $k_k \leq k_r$