Chapter 6
Seismic Assessment and Retrofitting of Existing Concrete Buildings

This chapter is the only one in the book devoted exclusively to existing concrete buildings. It builds on:

- Chapter 1, for the seismic performance requirements that may be apply to existing buildings and to their upgrading,
- Chapter 2, for the demonstration of the inherent vulnerability of substandard existing buildings,
- Chapter 3 for the quantification of the cyclic force and deformation capacity of concrete members, including the effect of poor detailing or retrofitting, and on
- Chapter 4 for the estimation of the seismic response via (mainly nonlinear) analysis.

The first part of the chapter is mainly devoted to seismic assessment of the as-is building. The rest to its upgrading through appropriate and cost-effective retrofitting. One section, namely Section 6.5, is specific to Part 3 of Eurocode 8. Retrofitting strategies and techniques commonly used today and in the foreseeable future for a concrete building as a whole and for its members, respectively, are described and their scope, pros and cons highlighted. Procedures, rules and expressions are given for practical member retrofit design. Finally, practical retrofitting is illustrated through two real applications.

6.1 Introduction

In many parts of the world, including Europe, design of new buildings for earthquake resistance is a relatively recent development. In those regions, resistance of buildings to lateral forces resulted in the past only from wind considerations. Provisions for seismic design and detailing of members and structures resembling those found in modern seismic codes did not appear before the mid-1970s in US standards, or the mid-1980s in European national codes. So, in the light of our current knowledge, the building inventory of many seismic regions worldwide is by and large substandard and seismically deficient.

M.N. Fardis, *Seismic Design, Assessment and Retrofitting of Concrete Buildings*, Geotechnical, Geological, and Earthquake Engineering 8, DOI 10.1007/978-1-4020-9842-0_6,

Although today and for the years to come the major earthquake threat to human life and property comes from existing substandard buildings, the emphasis of earthquake engineering research, practice and code-writing has been, and still is, on new construction. Policy makers hope that the problem of existing buildings will be solved gradually by attrition (sometimes accelerated by urban renewal and redevelopment). This may be a socio-economically optimal solution for those regions where the rate of occurrence of moderate to strong earthquakes is much lower than the attrition rate of old buildings. Although seismic resistance adds very little to the construction cost of a new building, the cost of seismic upgrading an existing one, including disruption of use, relocation of tenants, removal and replacement of non-structural parts, etc., is normally a large fraction of the building replacement cost and may be prohibitive for private owners or difficult for the local economy to bear.

Regardless of the economics, seismic retrofitting of buildings is effective in mitigating the seismic risk posed by a substandard building stock. The owner, private or public, may take the initiative individually, often on the occasion of a change in use, architectural remodelling or repair of damage due to an earthquake. Besides, in the context of a broader strategy for seismic risk mitigation, national or local authorities may launch "active" or "passive" seismic assessment and retrofitting programmes:

- In "active" programmes owners of certain categories of buildings may be required to complete the seismic assessment and – depending on its outcome – the retrofitting by certain deadlines. The buildings to be targeted may depend on the seismicity and ground conditions, on the importance, occupancy and perceived vulnerability of the building – as influenced by the type of material and structural system, the number of storeys, the time of construction with respect to certain benchmark dates of code enforcement, etc.
- In "passive" programmes seismic assessment – possibly leading to retrofitting – is triggered by events or activities related to the use of the building, such as a change that increases occupancy or importance class, repair of damage after an earthquake, remodelling of a minimum percentage of the floor area or above a certain budget, etc.

The seismic performance requirements to be met by the building as is or after retrofitting – if necessary – may be less stringent in compulsory, "active" programmes than in "passive" ones. In a "passive" programme triggered by remodelling, performance requirements may gradate with the extent and cost of remodelling works.

The need to retrofit or not a specific building and the scope and targets of the retrofitting (in terms of weaknesses and deficiencies to be corrected) normally come out of a detailed seismic assessment (or evaluation) of the building. A technically sound seismic assessment is a challenge. However, even when the need to retrofit is obvious, a detailed seismic assessment is worth carrying out: once a structural model of the building as-is is set up and analysed, it can be used at little extra cost as the basis for studying various retrofitting options and for detailed retrofit design. Besides, a detailed assessment in principle provides an objective picture of the seismic vulnerability and resistance, independent of any preconceptions of the engineer doing it. So, recent years have seen a dramatic decline in the use and perceived value of empirical rapid screening methods, used in the past to identify whether seismic retrofitting is indeed necessary.

It is nowadays recognised that rapid screening procedures are not reliable and cannot replace detailed seismic assessment of individual buildings. Rapid screening may be considered as a useful guide in "active" seismic assessment and retrofitting programmes for seismic risk mitigation in a certain region or urban area, helping to identify priority buildings or categories thereof. The motivation of rapid screening derives from the fairly good performance of most substandard buildings in earthquakes as strong as in Mexico 1985, Luzon Philippines 1990, Erzincan 1992, Kobe 1995 or Kocaeli 1999. The objective then is to screen out the buildings that can withstand a strong earthquake by virtue of overstrength alone, rather than ductility. The more costly detailed assessment may be reserved for the most vulnerable part of the building stock. The final verdict for the individual buildings identified as most vulnerable has to await the outcome of their detailed seismic assessment.

For the reasons put forward in the previous paragraph, rapid screening procedures are not dealt with in this book beyond broadly defining their features in the rest of Section 6.1.

The archetypal rapid screening procedure is that in ATC (1988). As this procedure is tailored to the building stock of the US, only its basic idea can be transferred elsewhere. Any adaptation to the specific features of the local building stock should be continuously calibrated on the basis of damage data from earthquakes that have hit the area where the procedure is being applied. In general, in a ATC (1988)-type of rapid screening, a single Basic Structural Score (*BSS*) is first assigned to a (concrete) building, depending on the combination of seicmicity zone, structural system (frame, wall or dual), design code(s) applied and special features (e.g., partial or full infilling, squat columns, open storey, etc.) that may or may not have been sufficiently addressed by the design code applicable at the time of construction. A Structural Score S is calculated then, as the sum of *BSS* and of a series of Performance Modification Factors (*PMF*s), accounting, e.g., for apparent construction quality and deterioration (e.g., reinforcement corrosion), irregularity of the building in plan and elevation, number of storeys, topography and ground conditions, location in the building block and possibility of mid-floor pounding with adjacent buildings, etc. The seismic vulnerability of the building at the specific site is evaluated on the basis of its final S-value.

To bridge the gap between the empirical, narrow-scope rapid screening in ATC (1988) and modern-day detailed seismic assessment, a three-tier assessment approach has been introduced in the US (ASCE 2003). The 3rd tier is a full-fledged detailed seismic assessment of the type in ASCE (2007). The 1st one comprises several checklists requiring trivial calculations and some detailed information on the as-built structure. Under certain conditions of seismicity, number of storeys and target performance level, a positive outcome of the 1st tier may be taken as definite and make recourse to the 2nd tier (which is a simplified detailed seismic assessment) redundant. The earlier Japanese three-tier approach (JBDPA 1977) is of a similar character.

6.2 Seismic Vulnerability of Existing Concrete Buildings

6.2.1 System and Layout Aspects and Deficiencies

Normally existing substandard buildings have been designed for very low lateral force resistance, if any. So, they are expected to develop significant inelastic action, even under a moderate earthquake. To sustain it, they should have considerable ductility. As emphasised in Chapter 1, this requires ductility at both the local and the global level. As pointed out in the next section, in existing substandard buildings potential plastic hinge regions are not detailed for ductility. Besides, members are not capacity-designed against pre-emptive brittle failure in shear. More important, though, existing substandard buildings seldom have the strong and stiff vertical spine necessary for spreading inelastic deformation demands throughout their height and avoiding a storey mechanism. If they have one in the form of strong walls, it is more by coincidence than by design. Even more important, as a rule their overall structural layout is seismically deficient.

The compelling consideration in conceptual structural design of old buildings was not earthquake resistance but gravity loads and conformity to the architectural layout. Complete two-way frames are the exception rather than the rule, as beams are often indirectly supported on other beams instead of columns (see Figs. 2.13(c), 2.25 and 4.14 for examples), or are mainly in a single horizontal direction to support one-way slabs (as, e.g., in Figs. 2.21 and 2.24). Overall, old buildings have few, if any, of the attributes listed in Section 2.1.2 and analysed in the rest of Section 2.1 as favourable for earthquake resistance. Torsional imbalances of lateral resistance and stiffness are common (see Fig. 2.1), as are vertical irregularities of strength and stiffness (Fig. 2.4) or geometry (Fig. 2.3). Floor diaphragms are sometimes not continuous or strong enough (see Figs. 2.21 and 2.25) or well connected to the lateral load resisting elements (see Figs. 2.6 and 2.7) to tie everything together into an integral system. Buildings with only the ground storey open (for commercial use or parking) are not protected by strong walls from soft-storey collapse (Fig. 2.10) or may develop plastic hinges in strong columns connected with weak beams owing to the effect in Fig. 2.9(b) and (c) (see Fig. 3.27(a) and (b) for examples). Captive columns are quite common for architectural reasons (Fig. 2.12), etc.

As pointed out in Section 2.1.13.5 in connection with Fig. 2.13(a), the contribution of staircases to the storey lateral strength and stiffness sometimes helps flexible buildings. Most of the times, however, depending on its connection to the elements of the lateral load resisting system and/or its location in plan, the staircase does more harm than good (see Fig. 2.13(b) and (c) and case at the bottom left corner of Fig. 3.35).

The systematic deficiencies of old substandard concrete buildings do not necessarily mean that they will do poorly in the event of an earthquake. Quite a few of them have gone unscathed through strong earthquakes thanks to unintended, albeit significant, lateral strength provided by design and construction practices of the past. Examples include closely spaced columns, uniform infilling with strong, good quality masonry having few large openings, or even large concrete walls, be it underreinforced.

6.2.2 Common Deficiencies and Failure Modes of Concrete Members

Owing to their poor structural layout, the lack of a strength hierarchy engineered to control the inelastic response mechanism, deficient or discontinuous load paths, etc., existing substandard buildings may experience certain concentration of seismic deformation demands to few of their elements in the event of a strong earthquake. Unfortunately these elements may be ill-prepared to withstand the increased demands, as they lack detailing for ductility and are not protected from pre-emptive brittle failures.

Non-ductile failures of members or connections due to poor detailing are aplenty in reconnaissance reports. Regarding flexural plastic hinges, Fig. 3.27(b), (c) and (f) depict failed column end regions with little confining reinforcement. In these figures ties not anchored by a 135° hook into the concrete opened up. Figure 3.27(f) is also a case of poor lapping at floor level. Figure 3.29 shows that beam bars (especially bottom ones) may pull-out from corner joints, if not anchored there by a 90°-bend or hook. Figure 3.35 depicts several shear failures of columns or walls having only widely spaced perimeter ties and indeed in some cases poorly closed ones. Obviously, the members in Fig. 3.35 have not been protected from shear failure by capacity design. Finally, the two corner joints in Fig. 3.47(b) and (c) had no horizontal reinforcement.

Columns without engineered earthquake resistance have normally been designed only for gravity compression with a nominal eccentricity. So, they are not only undersized and poorly detailed, but also have low flexural and shear resistance against lateral loads. By contrast, the beams of seismically deficient buildings normally have substantial flexural and shear resistance thanks to their design for factored gravity loads. So, unlike column failures which abound, beam failures are rare. They are mostly limited to bar pull-out of the type of Fig. 3.29, as anchorage of bottom bars at the supports is of little importance for gravity loading. This type of failure reduces the moment resistance of the frame and increases its flexibility, but poses little threat to life safety.

Faced with the poor detailing and obvious lack of engineered earthquake resistance of old concrete members and the deficiencies of the structural layout, the engineer carrying out a seismic evaluation of an existing building will contrast them with current knowledge and design practice for new buildings (including his/her own practice) and may come to the conclusion that the building is doomed in the event of a strong earthquake. This is human, but it may be premature. Each building has its own personality, reflecting that of the original designer and his/her perceptions, which may be different from the preconceptions of the engineer carrying out the evaluation. He/she should also look for possible sources of overstrength (see last paragraph of Section 6.2.1) that may partly make up for the member and system deficiencies. Although good judgment and experience are of prime importance for the seismic evaluation of an existing building, only a meticulous detailed assessment, e.g., along the lines of Section 6.5, can point out its real deficiencies and overstrengths and guide properly the decisions for its retrofitting.

6.3 The Predicament of Force-Based Seismic Assessment and Retrofitting

In the past detailed seismic assessment of individual buildings was force-based, mimicking design of new buildings. Capacity-demand comparisons were carried out at the member level in terms of forces, with internal force demands and resistances determined according to code provisions for the seismic design of new buildings. A prime example is the ENV version of the relevant part of Eurocode 8 (CEN 1996), according to which assessment had to be carried out by checking whether the provisions of the part of the ENV-Eurocode 8 for the design of new buildings for one of three Ductility Classes (H, M or L) were met. So, all members were first examined for fulfilment of detailing and minimum/maximum reinforcement rules of the three alternative DCs. If all of them satisfied those of one of the two upper DCs, they were also checked for fulfilment of the corresponding capacity design rules. If these rules were met, the value of the q-factor for which the building qualified was determined, according to its structural system and regularity. Otherwise, the engineer would classify the building as DC L and be content with its low q-factor value.[1] With the value of the behaviour factor q known, the design spectrum could be entered and linear analysis, of the lateral force or of the modal response spectrum type was employed to determine the design internal forces, E_d, of members (including P-Δ effects and the applicable capacity-design calculations). Design values of member resistances, R_d, were also determined, according to the relevant rules of Eurocode 2, as modified by Eurocode 8, if DC M or H was applicable. If $E_d > R_d$ was not satisfied, strengthening of the building was required.

Assessment of existing structures by checking compliance with a standard for the design of new ones is neither rational nor practical. An old structure is very unlikely to meet the very stringent requirements of modern codes for structural regularity, ductility at the local level (member detailing) and at the structural level (control of inelastic response through capacity design), continuity of the load path, etc., and

[1] The ENV-Eurocode 8 did not provide for "secondary" members, to allow exempting some elements from full satisfaction of the requirements of DC M or H for detailing, capacity design, etc.

qualify for a Ductility Class that uses ductility and energy dissipation capacity for earthquake resistance. If it doesn't, and unless it is a low-rise building with large concrete walls, it is also very unlikely to have the lateral strength necessary to resist elastically the seismic action (i.e., with $q = 1.5$). In this way all old structures may be found to be inadequate and to need retrofitting. If so, to comply with a code for new structures, practically all structural elements would have to be upgraded to meet all the resistance and detailing requirements of that code. Then the cost of retrofitting would exceed that of replacement. The owner is then very likely to decide to do nothing and continue using (or living in) a building that is now known to be unsafe.

6.4 Seismic Performance Requirements and Criteria for Existing or Retrofitted Buildings

In view of the predicament of conventional force-based approaches, the performance requirements, criteria and procedures adopted now for existing or retrofitted buildings differ from those in current codes for new construction. Recognising the much higher total cost of retrofitting (including the indirect cost of disruption of occupancy) compared to new construction, the new approach adopts more flexible requirements, which address the real intent of the owner and of the code-maker. The new requirements are served by more rational and less conservative criteria, limiting retrofitting to the cases where it is really needed and making it more cost-effective. Note that the pragmatic new attitude does not rely on the presumably shorter remaining service life of an existing building, as on that basis a building could be evaluated as adequate for the lower seismic action corresponding to a remaining life of a few years, after the eventless end of which the positive evaluation might be renewed for another period, etc., which does not make much sense.

The new requirements are fully and explicitly performance-based, in the sense of Section 1.1.2. A multi-level performance menu is provided, for the owner (or the competent authority) to choose the performance objectives that fit not only the importance, use and occupancy of the building, but also his/her means and intent. For example, an owner may be prepared to pay more, to avoid any disruption of the operation of the facility after a rare earthquake. By contrast, another owner may just have the means and the wish to avoid collapse under an occasional earthquake, retaining the option of retrofitting to a higher performance objective if financing conditions improve in future (see Section 1.1.2 for the meaning and definitions of the various performance and hazard levels).

The new criteria serving the performance objectives are also more rational and less conservative than the prescriptive ones associated with the performance requirements of current codes for new buildings:

– Poor detailing combined with low force resistance in many members is not a problem, provided that the system of lateral-load-resisting elements, old, retrofitted or new, assures global stability. Some members may be explicitly allowed to develop large, permanent post-ultimate-strength deformations, provided that their gravity-load capacity is not impaired. These may be "secondary members", at a number and with a contribution to lateral resistance which are more relaxed compared to new buildings or not capped at all. They may even be "primary" ones, under the condition that the system as a whole meets the performance objectives. This of course entails checking member deformation demands against their capacities.

– Sources of earthquake resistance and energy dissipation in the existing and in the retrofitted structure that are normally neglected in the design of new buildings, are explicitly taken into account. Examples include the positive effects of non-structural elements (e.g. masonry infills) and the redistribution and reduction of seismic demands thanks to nonlinearities in the structural system and the foundation, without artificial limits. This of course entails modelling and analysis at a higher level of sophistication than in the simple, yet conservative, approaches commonly used for new designs.

As these new trends become established through successful application in assessment and retrofitting projects, they will start affecting codes for the design of new structures as well. This represents a reversal over the past tradition, where procedures and codification for existing structures followed and emulated those for new ones.

6.5 Performance- and Displacement-Based Seismic Assessment and Retrofitting in Eurocode 8

6.5.1 Introduction

Part 3 of Eurocode 8 (CEN 2005a) is unique among all EN-Eurocode parts in many respects:

1. It is the only one in the whole set of 58 EN-Eurocode that deals with existing structures.[2]
2. It is essentially the first standard in Europe on seismic assessment and retrofitting. So, as there is no previous experience in European practice with codified seismic assessment and retrofitting, Part 3 of Eurocode 8 is an experiment. It is not sure yet how it will work in engineering practice.
3. Unlike all other EN-Eurocodes, which will eventually apply for all structures within their scope (i.e., to all new structures), Part 3 of EN-Eurocode 8 will not apply to all structures in its scope, i.e., to all existing buildings, but only to those that the owner – or local Authorities – have decided to seismically assess and possibly retrofit. So, this EN-Eurocode addresses only the structural aspects of

[2]As a matter of fact, an effort for an Annex to EN1990: "Basis of Design" on "Assessment and Retrofitting" has been launched in 2008.

seismic assessment and retrofitting and will apply once the requirement or will to assess a particular building has been established. The conditions under which seismic assessment of individual buildings – possibly leading to retrofitting – may be required are beyond the scope of Part 3 of EN-Eurocode 8. The initiative for seismic assessment and retrofitting lies with the owner, unless a national or local programme is undertaken for seismic risk mitigation through seismic assessment and retrofitting.

4. As a consequence of the other peculiarities, the extent and scope of the normative (i.e., mandatory) part of Eurocode 8-Part 3 is limited, covering only the general rules on:

   – the performance requirements and criteria,
   – the applicability conditions of the four analysis methods,
   – the type of verifications for ductile and brittle modes of behaviour and failure, and
   – the collection of information for the assessment and its implications, etc.

   All material-specific aspects and details (including expressions for modelling and verifications) can be found in Informative Annexes, for:

   – concrete structures,
   – steel or composite structures, and
   – masonry buildings.

   The information in these Annexes is not binding. It is up to National Authorities (but not to individual designers or owners) to adopt or not this information or even replace it with reference to other, national, sources of information.

6.5.2 Performance Requirements

Section 1.1.3 has described the three performance levels provided in CEN (2005a), called there Limit States:

1. “Damage Limitation” (DL);
2. “Significant Damage” (SD); and
3. “Near Collapse” (NC).

As pointed out in Section 1.1.3, it is not necessary to fulfil all three Limit States under the corresponding the “hazard level”. The country, through its National Annex – or the owner, if the country doesn’t make a choice – will decide how many and which of the three should be met. National authorities will also choose the “hazard levels” corresponding to these Limit States, depending on the prevailing level of risk tolerance and the socio-economic conditions. National authorities may well leave the choice open for the owner to make.

It is hoped that national authorities will choose the performance requirements for existing buildings in their territory so that they encourage owners to retrofit their property, while the population of buildings to be retrofitted is acceptable for the society and the national economy. As a matter of fact, the intent of the Eurocode is to let the owner make all these choices (of course after listening to advice from the designer or an engineering consultant), depending on his/her means, priorities and needs. For example, an owner may afford only to upgrade the very poor seismic resistance of his/her residence for collapse prevention in an occasional earthquake, while another may have the interest and means to retrofit his/her facility to be operational even after a rare event.

Unlike its practice for most Nationally Determined Parameters, the Eurocode itself does not a give a recommendation for the "hazard levels" corresponding to the three Limit States. It mentions, instead, in a note that the performance objective recommended as suitable to ordinary new buildings is a 225 year earthquake (20% probability of exceedance in 50 years), a 475 year event (10% probability in 50 years), or a 2475 year one (2% probability of being exceeded in 50 years), for the DL, the SD or the NC "Limit State", respectively. As a matter of fact, countries and owners should avoid adopting blindly this note, because in most seismic regions of Europe the 475 year earthquake is not much stronger than the 225 year one. So, if the 225 year earthquake is indeed chosen for the "Damage Limitation" Limit State, it will govern the whole assessment and/or retrofitting sufficiently to make it at best uneconomical or sometimes unfeasible.

Note that an earthquake with a mean return period of 475 years, mentioned in the above note of Part 3 of Eurocode 8 as suitable for the "Significant Damage" (SD) level in ordinary new buildings, is also the choice recommended in Part 1 of Eurocode 8 for the "local-collapse prevention" level of new designs. It appears, therefore, that these two parts of Eurocode 8 do not recommend any differentiation in the "performance objective" that has to do with Life Safety between new and existing buildings. However, such differentiation is made through the "compliance criteria" specified in these two parts for this single performance level. Although there is no direct correspondence of the criteria, those in Part 3 for existing buildings are less demanding than what Part 1 provides for new ones.

Performance differentiation of essential or large occupancy buildings from ordinary ones is effected as in new buildings, i.e., by multiplying the seismic action with the "importance factor", which has the recommended values given in Part 1 for new buildings (see Section 1.1.1).

6.5.3 Information on the As-Built Geometry, Materials and Reinforcement

A prime prerequisite for the seismic assessment or retrofitting of an individual building through analysis of a fairly detailed structural model is to have all the information needed for setting up the model: the topology of the lateral-load-resisting system, the cross-sectional dimensions, reinforcement and relevant material properties of its members, the gravity loads acting concurrently with the seismic action and the corresponding masses, etc.. If infill walls play an important role in the seismic response and will be included in the model, their topology, thickness and openings, as well as the masonry properties, should also be known. Ideally, all this information will be available in as-built drawings and material test reports from the time of construction and should be confirmed through spot-checks and lab testing of few samples. However such information is often not available, or turns out to be incomplete or unreliable. In that case, missing information is collected through a survey of the building and a campaign of in-situ measurements and lab tests on samples.

If the building as-is has certain earthquake resistance and is assessed to find out whether retrofitting might be avoided, it is essential to have good knowledge of the amount and detailing of the reinforcement and of the material properties in at least the critical structural and non-structural members. However, it is in such a building that collection of this information through exposure of reinforcement and extracting samples for lab tests is harder, as in all likelihood it is occupied and used at the time. Moreover, any damage to members that will not be retrofitted in the end should be fully redressed and not just patched-up. In other cases it is clear from the outset that the as-is building is doomed unless drastically upgraded, e.g., by adding major new components, such as concrete walls, to provide most of the required earthquake resistance. Then its structural model, although still valuable for identifying the specific deficiencies to be remedied by retrofitting, is mainly used as the basis for studying different retrofitting schemes. In such situations the performance of the retrofitted building is not very sensitive to the amount and detailing of the reinforcement or the material properties in the existing structural and non-structural members. The same applies for (vertical) members suffering from severe reinforcement corrosion and to be concrete-jacketed anyway. Unfortunately, these are also the cases where collection of information is easiest, as the building may be evacuated anyway before the retrofitting and any damage due to exposure of reinforcement and material sampling will be corrected anyway during retrofitting. At any rate, plenty of judgement is needed to adapt the scope and depth of the campaign of destructive in-situ measurements and sampling to the specific conditions and needs of a project. To this end, the in-situ collection of data should be carried out – or at least controlled – by the engineer in charge of the subsequent seismic assessment and retrofit design. It should never be delegated to another team that follows blindly a protocol producing lots of redundant data, while possibly leaving out crucial information.

Needless to say, the more extensive and reliable the information, the smaller is the uncertainty and the closer is the analysis model to reality. The uncertainty is reflected in the values of the partial safety factors, γ_m, for the calculation of member capacities and of any model uncertainty factors, γ_{Rd}, involved in the verifications of the existing or of the retrofitted building. So CEN (2005a) distinguishes three different cases, depending on the amount and reliability of the information available for the as-built structure:

1. "Limited knowledge";
2. "Normal knowledge";
3. "Full knowledge".

These three cases are defined as follows:

1. In "limited knowledge" permanent loads and masses, as well as member and infill geometry should be derived either:

   – from drawings of the original construction and of any subsequent modifications, validated in-situ visually and with spot-measurements, or
   – through a full campaign of in-situ measurements of geometrical quantities.

   Original information on the amount and detailing of reinforcement or the materials used is not essential. Indirect information and judgement may be used for them, instead. One can make default assumptions for the materials on the basis of the codes applicable and the practice prevailing at the time of construction (for the steel, after visual identification of its grade among the ones used at that time). These assumptions should to be spot-verified or calibrated in-situ with one material sample per floor for each type of member (column, beam or wall). Past codes and practice may also be the basis for the estimation of the amount and detailing of reinforcement through simulation of the original design. In other words, to figure out the reinforcement the engineer tries to put him/herself in the position of the original designer and contractor. The results of the simulated design should be confirmed or calibrated through spot checks of a small fraction of the total number of structural members of each type, recommended in CEN (2005a) as at least 20%. Needless to say, all this requires good knowledge of past codes and design/construction practice, as well as experience and judgement. For example, attention and spot-checking should focus on the important primary elements (especially those of the most critical storey) and may be more lax for secondary ones or for infills. Spot-checking should be prolonged if it has given large scatter or significant deviations from the default assumptions for materials or from the outcome of the simulated design. Extraction and testing of concrete cores should be supplemented with an extensive campaign of indirect measurements (with concrete hammer or ultrasounds), calibrated against the core test results.
2. "Normal knowledge" is the reference case for the available information. It entails knowledge of structural topology, cross-sectional dimensions, infills and amount and detailing of reinforcement either from:

   – original construction drawings (including any subsequent modifications), confirmed by in-situ checks (as in "limited knowledge"), or
   – (if original construction drawings are unavailable or not confirmed by the spot-checks) a full campaign of in-situ measurements, with exposure of reinforcement in a large fraction of the total number of structural members of each type, recommended in CEN (2005a) as at least 50%.

Material properties are derived either:

- from the original specifications, verified in-situ with few samples (Eurocode 8 recommends one per floor for each type of member), or
- through in-situ sampling and testing of some samples (Eurocode 8 recommends two per floor and member type).

Although masonry infills are not specifically mentioned by Eurocode 8 in this connection, it makes sense to estimate their strength properties and Elastic Modulus from the properties and geometry of the masonry blocks and mortar using semi-empirical expressions (e.g., those in Eurocode 6).

3. For "full knowledge", the engineer should draw information for the permanent loads, the topology of the structural system, the member cross-sectional dimensions, the amount and detailing of reinforcement and the location and geometry of infill walls from detailed original construction drawings, confirmed by checking at least 20% of all members per member type (column, wall, beam or infill). If the outcome of the spot checks does not fully agree with the drawings, then it is as if these drawings were not available. Lacking original drawings, a thorough survey of the structure is carried out, including exposure of reinforcement in almost all structural members of each type ((CEN 2005a) recommends at least 80%). Material properties are inferred either:

   - from test reports at the time of construction, verified by few samples ((CEN 2005a) recommends one per floor and member type), or
   - from in-situ testing, material sampling and lab tests of several samples ((CEN 2005a) recommends three per floor and member type).

   If masonry infills are considered in the assessment, certain sampling and testing for shear and compressive strength and for Elastic Modulus makes sense.

In all cases, whenever the scatter of test results in-situ or in the lab (as measured by the coefficient of variation) is large, or there is evidence that supervision during construction has not been so meticulous, the engineer should use judgment to possibly extend sampling and testing beyond the minimum required.

According to CEN (2005a), the mean value properties of existing materials should be used, as directly obtained from in-situ tests and any additional sources of information, after "correction" with a "confidence factor". The "correction" should always be safe-sided. Mean material properties used in the calculation of a capacity of an existing member (yield moment, shear resistance, chord rotation or curvature at yielding or at ultimate, etc.) to be compared to a demand, are divided by the "confidence factor". By contrast, if these mean material properties are employed for the calculation of a moment resistance from which capacity-design action effects are derived for a brittle mode of failure through equilibrium (see Section 1.3.6), then they are multiplied by the "confidence factor". Similarly, the yield stress considered for existing longitudinal bars at lap splices or bar anchorages is the mean value from in-situ tests and any additional sources of information, multiplied by the "confidence factor".

The values of the "confidence factor" recommended in CEN (2005a) are:

1. For "limited knowledge": 1.35;
2. for "normal knowledge": 1.20;
3. for "full knowledge": 1.00.

No confidence factor is applied on the nominal strength of new materials added for the purposes of retrofitting. If an existing member is modified, e.g., by FRP- or concrete-jacketing, a "confidence factor" is applied only on its old materials. Moreover, although such a distinction is not clear in CEN (2005a), it makes sense to apply to each old material (steel, concrete, or infill masonry) the value of the "confidence factor" corresponding to its own level of knowledge. For instance, member sizes may be well known and concrete may have been thoroughly sampled and lab-tested, while the amount and detailing of reinforcing bars may be rather unknown, or bars of a certain diameter or the infill material have not been sampled and tested at all.

According to CEN (2005a) "limited knowledge" may support only linear analysis. Note that the applicability of an analysis approach does not really depend on the level of knowledge available for the as-built structure. The restriction of the use of nonlinear analysis methods only to the cases of "normal" or "full knowledge" stems mainly from the concept of harmonised accuracy: it does not make much sense to use sophisticated and complex modelling and analysis with poor and highly uncertain input data. The real purpose, though, of the restriction is to motivate engineers who prefer using a more advanced method of analysis as less conservative, to collect also more information about the as-built structure. Note that, if the level of knowledge associated with different aspects of the as-built structure (e.g., the loads, the material strengths, the quantity and detailing of reinforcement, etc.) is different, it is the lowest level of knowledge for any one of these aspects that determines the type of analysis allowed.

6.5.4 Seismic Analysis and Models

6.5.4.1 Seismic Analysis Procedures and Applicability Conditions

As pointed out in Sections 4.1.2 and 4.11.1, the prime (if not only) aim of an analysis for displacement-based seismic assessment or retrofitting is the calculation of the deformation demands in structural members. Part 3 of Eurocode 8 (CEN 2005a) provides to this end the full menu of analysis options described in Chapter 4:

– the two linear options: linear static (Section 4.3) and modal response spectrum analysis (Section 4.4), and
– the two nonlinear ones: nonlinear static or "pushover" analysis (Section 4.6.1) and nonlinear dynamic analysis (Section 4.6.2).

In seismic design of new buildings linear analysis uses the design response spectrum incorporating the behaviour factor q. In displacement-based assessment or retrofitting, by contrast, linear analysis – when applicable – uses the 5%-damped elastic response spectrum. In CEN (2005a) member inelastic deformation demands (e.g., chord-rotations) are derived directly from such an analysis, applying the equal-displacement rule at the member level. The sole condition for this simplification is to have uniform chord-rotation ductility demands throughout the building. As pointed out in Section 4.11.1, uniformity of inelastic chord-rotation demands over the building is evaluated on the basis of the demand-to-capacity ratio, *D*/*C*, in *flexure*, where *D* is the bending moment at the end of a member due to the seismic action and the concurrent gravity loads from linear analysis and *C* the corresponding moment resistance,[3] calculated on the basis of the axial force due to gravity loads alone and using mean-value properties of old materials from in-situ tests and any additional source of information, or nominal values for new materials. Linear analysis is not allowed, if the maximum *D*/*C*-ratio in all "primary members" exceeds its minimum value over all such elements that have $D/C > 1$ by more than a factor in the range of 2–3 (as a Nationally Determined Parameter) with a recommended value of 2.5. Those sections around beam-column joints where plastic hinging is ruled out on the basis of the sums of beam or column moment resistances, ΣM_{Rb} or ΣM_{Rc}, respectively, are presumed to remain elastic and neglected in the check of the variation of *D*/*C* throughout the structure. As noted in Section 4.11.1, although there is no limit on the absolute magnitude of *D*/*C* for the applicability of linear analysis, there will always be at least one section in the entire structure where *D*/*C* slightly exceeds 1.0, making the relative limit on *D*/*C* essentially an absolute one.

If this applicability condition of linear analysis is not met, only nonlinear analysis is permitted. Recall, however, that (CEN 2005a) allows nonlinear analysis only when we have "normal" or "full knowledge" of the as-built structure. So, if only nonlinear analysis turns out to be applicable, or if the engineer wants to include in the structural model the contribution of "secondary members" to the lateral strength and stiffness (see below), he/she has no choice but to collect enough information for at least "normal knowledge".

The rules and procedures for linear or non-linear analysis are those described in Sections 4.3, 4.4 and 4.6 with reference to Part 1 of Eurocode 8 (CEN 2004a).

No matter which method of analysis is applied, we should take into account torsional effects due to the accidental eccentricity and simultaneous seismic action components according to Sections 4.8 and 4.7, respectively.

6.5.4.2 Modelling Aspects

For modelling, the reader is referred to Sections 4.9 and 4.10. Points which are of special importance in the context of an analysis for seismic assessment and retrofitting are highlighted and elaborated further in the present section.

Throughout the structural model mean values of material properties should be used. For old materials, as derived from in-situ tests and any additional sources of

[3]If the equal displacement rule indeed applies, *D*/*C* is about equal to the demand value of the chord-rotation ductility ratio, μ_θ.

information without modification by the "confidence factor" of Section 6.5.3. For new materials, as estimated on the basis of the nominal values of their properties.[4]

Part 3 of Eurocode 8 places special emphasis on the effective elastic stiffness of members to be used in linear or nonlinear analysis. It recommends for that purpose the secant stiffness to the yield-point from Eq. (3.68), instead of 50% of the uncracked gross-section stiffness, which is the default for force-based design in Part 1 of Eurocode 8 (see Section 4.9.2). For the purposes of Eq. (3.68) M_y and θ_y may be computed according to Sections 3.2.2.2 and 3.2.3.2, respectively, with the effect of any lap-splicing or FRP-wrapping taken into account on the basis of Sections 3.2.3.9 or 3.2.3.10, respectively, and that of concrete jacketing according to Section 6.8.2.3.

As pointed out in Section 6.5.6, in order to have a successful (i.e., positive) seismic assessment of the as-is building or evaluation of the retrofitted one, every single member in the model, "secondary" or "primary", should meet all relevant verification criteria. Verification criteria for brittle modes of behaviour (i.e., in shear), include a margin against failure (in that case, against significant loss of force resistance). Verification criteria for ductile modes (namely in flexure), also in general entail a certain margin from the expected value of ultimate deformation (conventionally identified with a drop in peak force resistance to less than 80% of the maximum possible capacity). Therefore, there is no real need to incorporate in nonlinear force-deformation models of members any strength decay due to this conventionally defined failure. If indeed failure takes place, the verification criteria will in all likelihood be violated, signalling the need for (more effective) retrofitting. When in the end all verification criteria are met, all members will be at sufficient distance from ultimate conditions (identified with a drop in peak resistance) to justify using a nonlinear force-deformation model with a non-decreasing primary loading branch. For example, Eqs. (4.82) and (4.85) may be used for the constant post-yield hardening ratio of the uniaxial M-φ or M-θ curve in primary loading. In these expressions M_u may be computed according to Section 3.2.2.5, M_y and φ_y from Section 3.2.2.2, θ_u and θ_y according to Sections 3.2.3.5 and 3.2.3.2, respectively, and φ_u from Section 3.2.2.4. Minor differentiations of the information in Annex A of CEN (2005a) from the contents of these latter sections in Chapter 3 have been pointed out: in Section 3.2.2.10 for φ_u, in Section 3.2.3.2 for θ_y and in Section 3.2.3.5 for θ_u. Sections 3.2.3.9, 3.2.3.10 or 6.8.2.3 provide means to take into account the effects of any lap-splicing, FRP-wrapping, or concrete jacketing, respectively, on M_u, M_y, φ_u, φ_y, θ_u and θ_y and point out also any differentiation from the information in Annex A of CEN (2005a).

Except wherever slippage of longitudinal bars from a joint or a foundation element is prevented or restricted through positive means of fixity at the end section or at a short distance inside the joint or the foundation element, model parameters should be determined assuming that such slippage does take place (i.e., using $a_{sl} = 1$ in Eqs. (3.66), (3.72), (3.78), etc.).

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[4]The value f_{ck} + 8 MPa, from which the elastic modulus of concrete is calculated according to Eurocode 2, is indeed a conventional mean value of the compressive strength of a concrete with nominal strength f_{ck}.

For any one of the simple models mentioned above we need to know the applicable values of the member shear span and axial load. As pointed out in Sections 4.10.1.3 and 4.10.1.4, even in a nonlinear analysis the member effective stiffness should stay constant throughout the course of the analysis. So, it should be based on the member axial force due to the gravity loads concurrent with the seismic action and on a constant value of the shear span at the member end(s) where a plastic hinge might form. Section 4.10.1.4 has suggested and Section 4.10.5.1 adopted (at point 2) the following values of the shear span:

(i) At each end of a beam or column framing into a joint with another element: half the clear length from one such joint to the next within the plane of bending, neglecting any intermediate nodes along this clear length where the beam or the column may be connected to another member lying in a plane at right angles to the plane of bending.
(ii) The shear span of a beam ending at an indirect support on another beam may be taken equal to its full clear span.
(iii) For each wall segment between successive floors: 50% of the height from the bottom section of the storey to the top of the wall in the building.

Because there is a single effective stiffness for each member, the average secant-to-yield-point stiffness at the two end sections in positive or negative bending should be used. Beams or columns connected at intermediate nodes with other members that have no stiffness within the plane of bending (e.g., a girder connected with cross-beams or girders, a column with two-way frame action in certain storeys and one-way action in others, etc.) have a single effective stiffness value throughout the full length between successive nodes where the member in question frames into elements with certain stiffness within the plane of bending. That effective stiffness is determined using in Eq. (3.68) the yield moments at the two end sections of this full length and as shear span 50% of the clear length between them.

During the course of nonlinear analysis any parameter of the member's nonlinear force-deformation relation other than its effective stiffness, such as the yield moment and the post-elastic primary loading branch derived from it, may be taken to vary with the axial force. Model parameters are not overly sensitive to the axial force value, at least for the smooth variation it exhibits even during a response-history analysis. By contrast, the shear span at each member end may vary wildly, depending on the relative magnitude and sign of the end moments and – in beams – on the concurrently acting transverse loads. So, it is strongly recommended to continue basing all parameters of the member nonlinear force-deformation relation on the initially adopted values of the shear span at the two member ends.

Note that the fixed-end moments at beam end sections due to the concurrent transverse loads induce zero chord rotation there. Moreover, the shear span value used in the calculation of the member's single effective stiffness normally differs from the moment-to-shear ratio at an end section at the time it first reaches its yield moment, M_y, during the calculated response. For these two reasons we may have significant disparity between the end's chord rotation at first yielding from the analysis on one hand, and the value of θ_y from Section 3.2.3.2 on the other. To avoid ambiguities, yielding of an end section should be considered to take place when M_y is attained there, no matter how much the concurrent chord rotation from the analysis, $\theta_{y,act}$, differs from the θ_y-value used in the calculation of the effective stiffness from Eq. (3.68). Plastic deformations are taken to start from that chord rotation. For member assessment the demand value of the plastic chord rotation should be compared with the plastic part of the ultimate chord rotation, θ_u^{pl}, i.e., with the part of the ultimate chord rotation, θ_u, in Eqs. (3.72), (3.78) and (3.79) beyond θ_y (as this part is modified for the effect of any lap-splicing or FRP-wrapping according to Sections 3.2.3.9 and 3.2.3.10, respectively). It is also that demand value of plastic chord rotation that should be divided by the value of θ_y from Eqs. (3.66) to give a $\mu_\theta{}^{pl}$-value for use in the verification of the member in cyclic shear through Eqs. (3.114), (3.115) and (3.127). If it is the total ultimate chord rotation, θ_u, from Eqs. (3.72), (3.78), (3.79), (3.80) and (3.84), etc., or a fraction thereof (see Section 6.5.6 and Table 6.1) that is compared to the demand for the purposes of member verification, that demand should be the value from the analysis plus ($\theta_{y,act}$.– θ_y), with $\theta_{y,act}$ being the chord rotation from the analysis when M_y is attained there and θ_y the value from Eqs. (3.66).

Unlike in new buildings, the contributions of "secondary members" to strength and stiffness against lateral loads may well be included in the structural model. Part 3 of Eurocode 8 (CEN 2005a) essentially requires including them, if the analysis is nonlinear. If their contributions are neglected, as allowed by CEN (2005a) in linear analysis, we cannot determine the seismic deformation demands against which "secondary members" should be verified (see Section 6.5.6.1 and Table 6.1). In that case, we should carry out two linear analyses per horizontal component of the seismic action, namely those referred to as analysis no. 1 and 2 in Section 4.12.3 (see also models no. 1 and 2 in Section 4.12.5.1). The seismic deformation demands in the "secondary members" of storey *i* from analysis no. 2 (including the "secondary members") is then multiplied by the ratio of interstorey drifts of that storey from analysis no. 1 (neglecting them) to those from no. 2. The outcome may be taken as the seismic deformation demands in the "secondary members" and compared to the corresponding capacities.

As pointed out in Section 4.12.5.2, if the cyclic degradation of strength and stiffness in "secondary members" is thought to be indeed much larger than in "primary" ones, it can be included in a nonlinear model via a descending post-yield branch (with negative slope) of the force-deformation curve in primary loading. In nonlinear dynamic analysis the hysteresis rules may instead include degradation of strength with cycling (see Section 4.10.1.6).

6.5.5 *Estimation of Force Demands by Capacity Design In Lieu of Linear Analysis*

Brittle mechanisms of behaviour, such as shear, are assessed in terms of forces. Linear analysis is not appropriate for the estimation of internal force demands in the inelastic regime, even when it is applicable for the estimation of member inelastic deformation demands. When the relevant applicability conditions are met and member inelastic deformation demands are indeed estimated for simplicity from linear analysis, we should resort to capacity design calculations of the internal force demands on members going into the inelastic range.

According to CEN (2005a) any moment resistance involved in the estimation of internal force demands through capacity design may be calculated using the axial force due to gravity loads alone, neglecting the one induced by the seismic action. After all, this latter value is unknown in the framework of linear analysis.

6.5.5.1 Shear Forces in Beams or Columns

Taking into account the possibility that under the seismic action of interest and the concurrent gravity loads plastic hinging in a column may take place at both, or just one, or none of the two end sections, i =1, 2, its shear force is estimated from the following modification of Eq. (1.12):

$$V_{\mathrm{CD,c}} = \frac{M_{\mathrm{c1}} + M_{\mathrm{c2}}}{H_{cl}} \tag{6.1}$$

where:

– if $$M_{\mathrm{Ed,ci}} > M_{\mathrm{R,ci}} \min\left(1; \frac{\sum M_{\mathrm{R,b}}}{\sum M_{\mathrm{R,c}}}\right)_i, \text{ then } M_{\mathrm{ci}} = M_{\mathrm{Rd,ci}} \min\left(1; \frac{\sum M_{\mathrm{R,b}}}{\sum M_{\mathrm{R,c}}}\right)_i \quad (i = 1{,}2) \tag{6.2a}$$

– if $$M_{\mathrm{Ed,ci}} \le M_{\mathrm{R,ci}} \min\left(1; \frac{\sum M_{\mathrm{R,b}}}{\sum M_{\mathrm{R,c}}}\right)_i, \text{ then } M_{\mathrm{ci}} = M_{\mathrm{Ed,ci}} \quad (i = 1{,}2) \tag{6.2b}$$

$M_{\mathrm{Ed,ci}}$ in Eqs. (6.2) denotes the moment at column end section i (=1, 2) from the linear analysis for the seismic action of interest and the concurrent gravity loads. $M_{\mathrm{R,ci}}$ is the moment resistance there calculated from the mean material properties from in-situ tests and any additional sources of information. $M_{\mathrm{Rd,ci}}$, appearing only in the 2nd part of Eq. (6.2a), is the moment resistance from these mean material properties times the relevant "confidence factor" corresponding to the "knowledge level" applying to this case. The ratios of sums of moment resistances, $\Sigma M_{\mathrm{R,b}}$, $\Sigma M_{\mathrm{R,c}}$, refer to the end sections of those beams and columns, respectively, that frame into the joint at column end i (=1, 2). H_{cl} is the clear height of the column.

By the same token, the shear force at cross-section x along the part of the beam nearest to beam end 1 (the other end denoted as 2) is estimated from a modification of Eq. (1.9a):

$$\max V_{\mathrm{CD,b1}}(x) = \frac{M_{\mathrm{b1}}^{-} + M_{\mathrm{b2}}^{+}}{L_{cl}} + V_{\mathrm{g}+\psi\mathrm{q,o}}(x) \tag{6.3}$$

where:

– if
$$M_{\mathrm{Ed,b1}}{}^{-} > M_{\mathrm{R,b1}}{}^{-}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_1, \text{ then } M_{\mathrm{b1}}{}^{-} = M_{\mathrm{Rd,b1}}{}^{-}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_1 \quad (6.4\mathrm{a})$$

– if $M_{\mathrm{Ed,b1}}{}^{-} \leq M_{\mathrm{R,b1}}{}^{-}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_1$, then $M_{\mathrm{b1}}{}^{-} = M_{\mathrm{Ed,b1}}{}^{-}$ (6.4b)

– if
$$M_{\mathrm{Ed,b2}}{}^{+} > M_{\mathrm{R,b2}}{}^{+}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_2, \text{ then } M_{\mathrm{b2}}{}^{+} = M_{\mathrm{Rd,b2}}{}^{+}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_2 \quad (6.5\mathrm{a})$$

– if $M_{\mathrm{Ed,b2}}{}^{+} \leq M_{\mathrm{R,b2}}{}^{+}\min\left(1;\frac{\sum M_{\mathrm{R,c}}}{\sum M_{\mathrm{R,b}}}\right)_2$, then $M_{\mathrm{b2}}{}^{+} = M_{\mathrm{Ed,b2}}{}^{+}$ (6.5b)

In Eqs. (6.4) and (6.5) $M_{\mathrm{Ed,bi}}$ (i=1, 2) is the moment at beam end section i (=1, 2) from the linear analysis for the seismic action of interest and the concurrent gravity loads. $M_{\mathrm{R,bi}}$ is the value of moment resistance there from the mean material strengths from in-situ tests and any additional sources of information and $M_{\mathrm{Rd,bi}}$ its value from these mean material properties times the relevant “confidence factor” corresponding to the “knowledge level” applying to this case. Hogging moments (superscript (–), tension at the top flange of the beam) are considered at end section 1 which is closer to the beam cross-section x where the shear force is calculated and sagging ones (tension at the bottom flange, superscript (+)) at the opposite end, 2. All moments enter in these expressions as positive, no matter whether they are hogging or sagging. The same for the shear force $V_{g+\psi q,o}(x)$ at cross-section x in a simply supported beam subjected to the concurrent gravity loads, $g+\psi q$.

6.5.5.2 Shear Forces in Walls

Capacity-design shears along the height of a multistorey wall cannot be established from the moment resistances of beams framing into it at floor levels, because the (unknown) lateral forces transferred to the wall by the floors enter also into the equilibrium condition. So, for all types of walls, “squat” (with $h_w/l_w>2$) or “slender” ($h_w/l_w > 2$), Part 3 of Eurocode 8 allows calculating capacity design shears from the following modification of Eq. (1.14):

– if $M_{\mathrm{Ed,w}}\ (z=0) > M_{\mathrm{R,w}}\ (z=0)$, then $V_{\mathrm{CD,w}}(z) = \left(\frac{M_{\mathrm{Rd,w}}(z=0)}{M_{\mathrm{Ed,w}}(z=0)}\right) V_{\mathrm{Ed,w}}(z)$ (6.6a)

– if $M_{\mathrm{Ed,w}}\ (z = 0) \leq M_{\mathrm{R,w}}\ (z = 0)$, then $V_{\mathrm{CD,w}}(z) = V_{\mathrm{Ed,w}}(z)$ (6.6b)

where z is elevation above the base, $M_{\mathrm{Ed,w}}(z)$ and $V_{\mathrm{Ed,w}}(z)$ are the moment and the shear from linear analysis for the seismic action of interest and the concurrent gravity loads, $M_{\mathrm{R,w}}(z{=}0)$ is the value of moment resistance at the base section from the mean material properties from in-situ tests and any additional sources of information and $M_{\mathrm{Rd,w}}(z{=}0)$ its value there from mean material properties times the relevant "confidence factor" corresponding to the "knowledge level" applying to that case.

According to the reasoning in Section 1.3.6.4 about Eq. (1.15), Eq. (6.6a) may be seriously on the unsafe side for "slender" walls in the nonlinear response regime, as it neglects higher mode effects on wall shears after yielding at the base. To avoid being on the unsafe side, for "slender" walls (i.e., where $h_{\mathrm{w}}/l_{\mathrm{w}} > 2$) with $M_{\mathrm{Ed,w}}(z) > M_{\mathrm{R,w}}(z{=}0)$, Eq. (6.6a) may be replaced by the following adaptation of Eq. (1.15) to the present conditions:

$$V_{\mathrm{CD,w}}(z) = \sqrt{\left(\frac{M_{\mathrm{Rd,w}}(z=0)}{M_{\mathrm{Ed},w}(z=0)}\right)^2 + 0.1\left(\frac{S_a\left(T_C\right)}{S_a\left(T_1\right)}\right)^2}\, V_{\mathrm{Ed},w}(z) \tag{6.7}$$

As in Eq. (1.15), $S_{\mathrm{a}}(T_1)$ and $S_{\mathrm{a}}(T_{\mathrm{C}})$ are the elastic spectral accelerations at the period of the fundamental mode in the horizontal direction closest to that of the wall shear force and at the corner period, T_{C}, of the spectrum, respectively.

6.5.5.3 Shear and Bond in Joints

By referring to the provisions for DC H beams in Part 1 of Eurocode 8, Annex A of Part 3 bases the calculation of shear force demands in joints on plastic hinging in the beams, Eqs. (3.134) and (3.135), discounting the possibility of column hinging. This is safe-sided. It makes more sense, however, to consider both possibilities and use the full set of Eqs. (3.134), (3.135), (3.136), (3.137), (3.138), (3.139) and (3.140) in Section 3.3.3.1 for the shear force and stress demand in a joint. Note that, if the moments from the linear analysis, M_{Eb}, M_{Ec}, are such that $\sum M_{\mathrm{Eb}} < \sum M_{\mathrm{Rb}}$ and $\sum M_{\mathrm{Ec}} < \sum M_{\mathrm{Rc}}$, then the nominal shear stress in the joint, v_{j}, may be estimated either:

- from Eq. (3.135) and a joint horizontal shear force, V_{jh}, calculated using in Eq. (3.134) $\sum M_{\mathrm{Eb}}$ instead of $\sum M_{\mathrm{Rb}}$, or
- from Eq. (3.140) and a joint vertical shear force, V_{jv}, calculated using in Eq. (3.139) $\sum M_{\mathrm{Ec}}$ instead of $\sum M_{\mathrm{Rc}}$.

According to the spirit of Part 3 of Eurocode 8 (CEN 2005a), the value of f_{y} to be used in Eqs. (3.134), (3.136) and (3.138) is the mean yield strength of beam bars estimated from in-situ tests and any additional sources of information times the relevant "confidence factor" corresponding to the "knowledge level" applying to the case in question. By referring to the rules for DC H beams in Part 1 (CEN 2004a), however, the letter only introduces a multiplicative factor of $\gamma_{Rd} = 1.2$ on f_y, to account for strain-hardening.

CEN (2005a) does not explicitly mention bond along beam bars crossing or anchored at beam-column joints, which is also a force-controlled mechanism. At any rate, Section 5.4.1 and Eqs. (5.10) in particular apply for their maximum diameter. The values $\gamma_{Rd} = 1.0$ and $k = 0.75$ may be used in these expressions. The mean tensile strength of concrete inferred from in-situ tests and any additional sources of information should be used there, divided by the "confidence factor" corresponding to the "knowledge level" applying to the particular case. The mean yield strength of beam bars from in-situ tests and any additional sources of information times the relevant "confidence factor" should be used there as f_{yd}. By the same token, ν_d should be calculated using the mean compressive strength of concrete as estimated from in-situ tests and any additional sources of information times the relevant "confidence factor". Note, however, that in existing buildings beam bars crossing or anchored at beam-column joints are very unlikely to fulfil Eqs. (5.10) applied according to the present paragraph. If they don't, we should keep in mind, first, that the problem cannot be easily fixed by retrofitting and, second, that the consequences of slippage of beam bars through or from joints are not so catastrophic. They amount to an apparent increase of beam flexibility due to fixed-end rotations and to a cap on the moment resistance of beams anchored at a joint without a bend.[5] If we want to take this latter effect into account, we may compute the beam moment resistance on the basis of a steel yield stress multiplied by the ratio of the bar diameter from Eq. (5.10b) to the actual diameter. The effective elastic stiffness of the beam from Eq. (3.68) does not need to be revised. However, the ultimate chord rotation at the beam end framing into such a joint is governed by the uncontrolled fixed-end rotation due to bar pull-out and cannot be computed anymore from Sections 3.2.3.4 or 3.2.3.5.

6.5.5.4 Transfer of Seismic Action Effects to the Ground

CEN (2005a) does not explicitly mention force-controlled mechanisms other than shear in beams, columns, walls or joints. Loss of bearing capacity under (part of) the foundation during an earthquake amounts to permanent deformations of the soil, leading to a permanent tilt and/or settlement of the building. Therefore, in essence it may be considered as a deformation-controlled, "ductile", mode of behaviour and failure. However, procedures for the estimation of permanent soil deformations in an earthquake and displacement-based verifications of the ground in the

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[5] A bar passing through an interior joint, even when it slips along its length within the joint, will be stabilised in the beam at the other side of the joint. Beam bars anchored at a joint through a 90° bend or a 180° hook rely on it for ultimate stabilisation against pull-out.

framework of an integrated, fully-coupled seismic design of the superstructure-foundation-soil system are still far away. The transfer of action effects to the ground is still checked as force-controlled and will do so for the foreseeable future.

Tie-beams and foundation beams of an existing building may be treated like the beams of the superstructure, allowed to enter the inelastic range and develop plastic hinges. So, if analysis is linear, these beams may be verified in flexure on the basis of elastically estimated chord-rotation demands[6] and in shear on the basis of forces derived according to Section 6.5.5.1. The bearing capacity of the underlying soil, however, should be checked in terms of forces. As long as we don't discover in the end that there is bearing capacity failure, the foundation soil may be considered elastic. The rules presented in Section 2.3.4 for the capacity design of the foundation may be applied for the calculation of the seismic action effects (forces and moments) transferred to the ground by the foundation system. This entails multiplying all seismic action effects transferred to the ground according to linear analysis by a factor a_{CD}. For individual footings Eq. (2.15a) may be adopted to the present conditions as follows:

$$- \text{ if } M_{R,y} \geq M_{Ed,y} \quad \underline{\text{and}} \quad M_{R,z} \geq M_{Ed,z,} \quad \text{then } a_{CD} = 1 \tag{6.8a}$$

$$- \text{if } M_{R,y} < M_{Ed,y} \quad \underline{\text{or}} \quad M_{R,z} < M_{Ed,z,} \quad \text{then } a_{CD} = \min[(M_{Rd,y}/M_{Ed,y}); (M_{Rd,z}/M_{Ed,z})] \tag{6.8b}$$

where $M_{Ed,y}$, $M_{Ed,z}$ are the two orthogonal moment components at the base of the column or the wall from the linear analysis for the seismic action of interest and the concurrent gravity loads. $M_{R,y}$, $M_{R,z}$ are the values of the corresponding uniaxial moment resistances from the mean material properties from in-situ tests and any additional sources of information, and $M_{Rd,y}$, $M_{Rd,z}$ are those from these mean material properties times the relevant "confidence factor" corresponding to the "knowledge level" applying to the case in question.

For common foundations of $N > 1$ vertical elements (e.g. on a foundation beam, a box-type foundation, a raft, etc.) the simplification allowed in Part 1 of Eurocode 8 (CEN 2005a) for new buildings ($a_{CD} = 1.4$, Eq. (2.16)) is not meaningful, as in the present case the seismic action effects from linear analysis should be de-amplified ($a_{CD} \leq 1$). A sensible option is to determine from Eqs. (6.8) a value of a_{CD} at the base of each one of the jointly-founded vertical elements, indexed by i (i=1, N) and weight-average the individual values, $a_{CD,i}$, into a composite one:

[6]Note that the applicability of the equal displacement rule to buildings considered on compliant elastic ground, and to the elements of the foundation system in particular, has not been confirmed yet.

$$a_{\mathrm{CD}} = \frac{\sum_{i=1}^{N} a_{\mathrm{CD,i}} M_{\mathrm{Ed,i}}}{\sum_{i=1}^{N} M_{\mathrm{Ed,i}}} \tag{6.9}$$

where $M_{\mathrm{Ed,i}}$ is that moment component $M_{\mathrm{Ed,y}}$, $M_{\mathrm{Ed,z}}$ from linear analysis for the seismic action of interest and the concurrent gravity loads which gives the minimum ratio $M_{\mathrm{R,y}}/M_{\mathrm{Ed,y}}$ or $M_{\mathrm{R,z}}/M_{\mathrm{Ed,z}}$ at the base of vertical element i (hence governs plastic hinging there).

6.5.6 Verification Criteria for Existing, Retrofitted, or New Members

6.5.6.1 Overview of the Criteria

For a (as-is or retrofitted) building to meet a given performance level (Limit State) under the corresponding seismic action, all its members, "primary" or "secondary", old, retrofitted or new, should conform to the corresponding verification criteria. As pointed out in Section 4.12.1, different criteria are used for "primary" and "secondary elements", with more safe-sided ones applying to the "primary" ones. It is up to the engineer to evaluate the real importance of certain elements of the existing or retrofitted building for its earthquake resistance and (re-)classify some of them as "secondary", if they meet the criteria for "secondary elements" but not those for "primary" ones.

The criteria follow the general verification format, $E_{\mathrm{d}} \leq R_{\mathrm{d}}$ (cf. Eq. (1.3)), but using for E_{d} and R_{d}:

- Deformations in the "ductile" modes of behaviour and failure (i.e., for flexure).
- Forces for the "brittle" ones (i.e., for shear).

The normative text of Part 3 of Eurocode 8 (CEN 2005a) describes these criteria in very general terms:

- At the Damage Limitation (DL) Limit State, structural elements, ductile or brittle, should stay below yielding;
- At the Limit State of Significant Damage (SD), ductile elements should not exceed certain "damage-related deformations" and brittle ones their "conservatively estimated strengths"; and
- At the Near Collapse (NC) Limit State, ductile elements should stay below "appropriately defined ultimate deformations" and brittle ones below their "ultimate strengths".

The information given then in Annex A of CEN (2005a) for concrete members is very specific, as summarised in Table 6.1. Following the proposals in Fardis (1998, 2001) and Fardis et al. (2003), the deformation measure used in

Table 6.1 Compliance criteria for assessment or retrofitting of concrete members in Annex A of CEN (2005a)

Mechanism	Member	Damage limitation – LD	Significant damage – SD	Near collapse – NC
Flexure (ductile)	Primary Secondary	$M_E \leq M_y$ or $\theta_E \leq \theta_{y,act}$	$\theta_E \leq 0.75\ \theta_{u,m\text{-}\sigma}$ $\theta_E \leq 0.75\ \theta_{um}$	$\theta_E \leq \theta_{u,m\text{-}\sigma}$ $\theta_E \leq \theta_{um}$
Shear (brittle)	Primary Secondary	$V_E \leq V_{Rd,EC2}$ and $V_E \leq V_{Rd,EC8}/1.15$; joints: $V_{CD,j} \leq V_{Rd,j\ EC8}$ $V_E \leq V_{Rm,EC2}$ and $V_E \leq V_{Rm,EC8}/1.15$; joints: $V_{CD,j} \leq V_{Rm,j\ EC8}$		

these specific criteria is the chord-rotation at member ends The demand and the capacity measures in Table 6.1 are elaborated further in Sections 6.5.6.2 and 6.5.6.3, respectively.

6.5.6.2 The Demand Side of the Verification

The demand and its measures in Table 6.1 are due to the seismic action in question, plus the concurrent gravity loads:

1. The moment, M_E, and the chord-rotation, θ_E, are of interest only at sections where the member frames into others having stiffness within a plane normal to the vector of M_E.
2. The shear force demand V_E at the SD or NC Limit State is obtained from capacity design calculations according to Sections 6.5.5.1 and 6.5.5.2, if linear analysis is used. Otherwise, the analysis results are used.
3. $V_{CD,j}$ is the maximum shear force in the joint from capacity design calculations according to the first paragraph of Section 6.5.5.3.

It is necessary to clarify how θ_E is determined. The chord with respect to which the rotation of member end sections is measured should be fairly consistent with the shear span used for the calculation of the corresponding ultimate chord rotation, θ_u, at the two ends:

– In a member expected to be in counterflexure when it yields at one end, θ_E is measured with respect to the chord connecting its two nodes on either side of the expected point of inflection. More specifically, in beams or columns with end nodes where the member frames into elements with certain stiffness within the plane of bending, θ_E is measured with respect to the chord connecting these two nodes, no matter any intermediate ones with other members having no stiffness in the plane of bending. The length of this chord is twice the shear span determined according to (i) in Section 6.5.4.2.
– If the member is expected to be in single curvature when it yields at one end, θ_E is measured with respect to the chord connecting that end to a node at or near the expected point of inflection. For example, if a beam frames into a column at one end and ends at an indirect support on another beam at the other, θ_E may be

measured with respect to the chord connecting these two ends. In a wall θ_E is of interest at a storey's bottom section and is measured with respect to a chord connecting the node there to one about half-way the distance from the top of the wall. In all these cases the length of the chord is equal to the shear span determined according to (ii) or (iii) in Section 6.5.4.2.

For the implementation of the above in systems beyond full two-way frames in 3D, the post-processor of analysis results for member verifications should allow the user to define for each member the chord with respect to which chord rotations at the nodes of interest are defined within each plane of bending and then calculate θ_E from the nodal rotations and displacements of these nodes in the global co-ordinate system.

6.5.6.3 The Supply or Capacity Side

(1) M_y in Table 6.1 is the yield moment at the end section, as computed, e.g., from Section 3.2.2.2, taking into account FRP-wrapping of the end of the member, short lap splices there or short anchorage beyond the end section, according to Sections 3.2.3.10 and *Effect of Lap-Splicing on the Yield Properties* in Section 3.2.3.9, respectively. For concrete-jacketed members Section 6.8.2.3 applies. Biaxial effects on yielding may be taken into account according to Section 3.2.3.8. As M_y is in this case an indirect measure of a deformation capacity, namely of $\theta_{y,act}$ in (2) below, it is based on mean strengths of old materials divided by the confidence factor applicable and on the nominal strengths of new materials. Because of the confidence factor, it is generally different from the value used in Eq. (3.68) for the effective stiffness.

(2) $\theta_{y,act}$ in Table 6.1 is the chord-rotation demand at the time the value of M_y in (1) above is attained during the analysis. Note that, if the verification is in terms of deformations, Annex A of Part 3 of Eurocode 8 recommends using instead the value of θ_y from Eqs. (3.66) in Section 3.2.3.2 (corrected for FRP-wrapping, short lap splicing or anchorage and biaxial loading according to the sections mentioned in (1) above). As discussed in Section 6.5.4.2, if this suggestion is adopted we may have a mismatch with attainment of M_y and ambiguities about yielding.

(3) θ_{um} in Table 6.1 is the expected value of the ultimate chord-rotation under cyclic loading, calculated using mean strengths for old materials divided by the confidence factor and nominal strengths for new materials. According to Annex A of CEN (2005a) the reference approach for its estimation is via Eqs. (3.78a) or (3.78b), modified for: poor detailing through to Eqs. (3.79), but with coefficient 0.825 instead of $1/1.2 = 0.833$; for lap-splicing of longitudinal bars starting from the end section according to approach (i) in section *Effect of Lap-Splicing on the Flexure-Controlled Ultimate Deformation* in Section 3.2.3.9; for FRP-wrapping of a plastic hinge region via the approaches in Section 3.2.3.10 based on Eq. (3.92) with lap-splicing or (3.89) without. The application of the reference approach to concrete-jacketed members is described in Section 6.8.2.4. The second approach in Eurocode 8 for θ_{um} applies

only to members detailed for earthquake resistance. It does not have extensions for poor detailing, lap-splicing, FRP-wrapping or concrete jacketing. It employs Eq. (3.70a), with the full expression for θ_y from Eqs. (3.66) instead of the flexure-only term, $\varphi_y L_s/3$. It calculates φ_u as described under point (a) at the end of Section 3.2.2.10, giving two confinement options. A different expression for the plastic hinge length, Eq. (3.75) or (3.76) in Section 3.2.3.4, goes with each option.

Section 3.2.3.5 has pointed out that Eq. (3.78c) is an equally good alternative to the reference approach in Annex A of CEN (2005a), Eqs. (3.78a) or (3.78b). Section 3.2.3.10 has added two alternatives to Eq. (3.89) for the estimation of the effect of FRP-wraps over a plastic hinge region without lap-splicing. Section 3.2.3.4 has shown that Eq. (3.72), used with Eq. (3.73) or (3.74) for the plastic hinge length and with φ_u according to Sections 3.2.2.4 and 3.2.2.10, predicts θ_{um} much better than its Eurocode 8 counterpart, albeit worse than the reference one based on Eqs. (3.78). When extended according to approach (ii) in sub-section *Effect of Lap-Splicing on the Flexure-Controlled Ultimate Deformation* for members with lap-spliced longitudinal bars, Eqs. (3.72), (3.73) and (3.74) predict θ_{um} better than approach (i) in sub-section *Effect of Lap-Splicing on the Flexure-Controlled Ultimate Deformation* in Section 3.2.3.9, which is the counterpart of the reference approach. The extension of Eqs. (3.72), (3.73) and (3.74) to members with continuous bars and FRP wrapping according to sub-section *Members with Continuous Bars* in Section 3.2.3.10 is also as successful as the reference approach with Eqs. (3.89), (3.90) and (3.91). As pointed out at the end of sub-section *Members with Lap-Spliced Ribbed Bars* in Section 3.2.3.10, however, this extension is not so good for FRP-wrapped members with lap-spliced bars. Last, but not least, the approach of Section 3.2.3.4, Eqs. (3.72), (3.73) and (3.74), has not been extended to members with continuous bars but poor detailing. All things considered, the extended version of Eqs. (3.72), (3.73) and (3.74) is still not as good or general as the reference approach of Section 3.2.3.5 and its extensions.

As pointed out at the end of Section 3.2.3.8, for biaxial bending both the approach in Section 3.2.3.4 and that of Section 3.2.3.5 may be applied in Eq. (3.84).

(4) $\theta_{u,m\text{-}\sigma}$ in Table 6.1 denotes the mean-minus-standard deviation estimate of the ultimate chord-rotation under cyclic loading. Like θ_{um} above, it is calculated from mean strengths for old materials divided by the confidence factor and from nominal strengths for new materials. Annex A in CEN (2005a) replaces this estimate directly with:

(i) $\theta_{u,m\text{-}\sigma} = \theta_{um}/1.5$, if θ_{um} is obtained from Eq. (3.78a) in Section 3.2.3.5 and its extensions for poor detailing, lap-splicing, FRP-wrapping, etc.;

(ii) $\theta_{u,m\text{-}\sigma} = \theta_y + \theta^{pl}{}_{um}/1.8$, if one uses the value of θ_y from Eq. (3.66) in Section 3.2.3.2 and $\theta^{pl}{}_{um}$ from Eq. (3.78b) in Section 3.2.3.5, with their corresponding extensions for poor detailing, lap-splicing, FRP-wrapping, etc.

(iii) $\theta_{u,m\text{-}\sigma} = \theta_{um}/2$, for θ_{um} from Eq. (3.70a) but with θ_y from Eqs. (3.66) instead of $\theta_y = \varphi_y L_s/3$, with Eq. (3.76) for the plastic hinge length and φ_u calculated according to point (a) at the end of Section 3.2.2.10 with the confinement model in Eurocode 2 and CEB (1991), Eqs. (3.8), (3.9), (3.13) and (3.25);

(iv) $\theta_{u,m\text{-}\sigma} = \theta_{um}/1.7$, if θ_{um} is obtained from Eq. (3.70a) with θ_y as in (iii) above, with Eq. (3.75) for the plastic hinge length and with φ_u calculated according to point (a) in Section 3.2.2.10 with the confinement model of Eqs. (3.4), (3.5), (3.10) and (3.18).

In the light of the test-to-prediction statistics for cyclic loading quoted in Section 3.2.3.5 for unretrofitted members with continuous bars and elsewhere in Section 3.2.3 for members with lap-splices and/or FRP-wrapping, the values in (i) and (ii) above may be considered as close to mean-minus-standard deviation estimates, for the cases when θ_{um} as a whole is obtained from either one of Eqs. (3.78), or $\theta^{pl}{}_{um}$ is estimated from Eqs. (3.78b) and (3.78c), respectively (also for the extensions of Eqs. (3.78) for poor detailing, lap-splicing, FRP-wrapping, etc.). However, the statistics quoted in Section 3.2.3.4 right below Eqs. (3.75) and (3.76) show that the values in (iii) and (iv) above are higher than mean-minus-standard deviation estimates by 50 and 37.5%, respectively. The statistics for cyclic loading quoted in the Section 3.2.3.4 right below Eqs. (3.73) and (3.74) suggest that a mean-minus-standard deviation estimate of θ_u may indeed be obtained as:

(v) $\theta_{u,m\text{-}\sigma} = \theta_{um}/1.8$, for θ_{um} from Eq. (3.72), with a plastic hinge length from Eqs. (3.73) or (3.74) and φ_u according to Sections 3.2.2.4 and 3.2.2.10.

(5) $V_{Rd,EC2}$ in Table 6.1 is the monotonic shear resistance before flexural yielding according to Eurocode 2, as given in detail in sub-section *The Variable Strut Inclination Truss of the CEB/FIP Model Code 90 and Eurocode 2* in Section 3.2.4.2. It is computed using mean strengths of old materials divided by the confidence factor and nominal strengths of new materials, in all cases divided by the partial factor for the material.

(6) $V_{Rd,EC8}$ is the cyclic shear resistance after flexural yielding from Eurocode 8, i.e., from Eqs. (3.114) for cyclic diagonal tension, or Eqs. (3.115) and (3.127), for cyclic diagonal compression in squat walls or columns, respectively. Like $V_{Rd,EC2}$, it is computed from mean strengths of old materials divided by the confidence factor and from nominal strengths of new materials, always divided by the partial factor of the material.

(7) $V_{Rd,jEC8}$ is the cyclic shear resistance of the joint according to Part 1 of Eurocode 8, Eqs. (5.13) and (5.14) in Section 5.4.2 ((3.143), (3.144) in Section 3.3.3.1), from the mean strength of old steel and concrete divided by the confidence factor and from the nominal one for new materials, in both cases divided by the partial factor of the material. Equations (5.15) (or (3.142) in Section 3.3.3.1) may be used as alternative to (5.14) and (3.143), with f_y at the right-hand-side

taken equal to the mean yield strength of beam bars multiplied by the confidence factor and by $\gamma_{Rd} = 1.2$ for strain hardening, without material partial factor.

(8) $V_{Rm,EC2}$, $V_{Rm,EC8}$, $V_{Rm,jEC8}$ in Table 6.1 are the shear resistances as in (5), (6) and (7), respectively, but computed from mean strengths of old materials modified through the confidence factor and from nominal strengths of new materials.

Annex A of CEN (2005a) recommends for (6)–(7) the usual values of partial factors for concrete and steel: 1.5 and 1.15, respectively, and a value of 1.5 for FRP.

The ultimate deformation estimates in (3) and (4) above increase with increasing shear span, L_s, at the end in question and with decreasing axial force, N, in the member. When N increases, a member's shear resistance in (5) or (6) increases, while that of a joint in (8) may increase or decrease. The shear resistance in (6), $V_{Rd,EC8}$, decreases with increasing L_s and with increasing demand value of the chord rotation ductility factor, μ_θ, at the end in question. All these parameters vary during the seismic response. If the analysis is linear, the verifications take place in the end, using the initial values of L_s (e.g., as listed under (i)–(iii) in Section 6.5.4.2) and N (due to gravity loads alone) and the peak value of μ_θ from the analysis. If the analysis is nonlinear, it is preferable to check all verifications at each point during the response, using the instantaneous values of N and μ_θ. For L_s it is normally sufficient to use the current value of M/V only at beam ends and, as a matter of fact, only when they are in hogging bending. This is the most critical condition for the shear and the chord rotation (at least for the usual position of the slab and distribution of longitudinal reinforcement between top and bottom). Conditions are rarely critical at the sagging end, while the variation of M/V there may be such that absurd instantaneous capacity estimates come out. If a nonlinear static ("pushover") analysis is carried out, the value of M/V at column or wall end sections varies smoothly with increasing lateral forces and may be taken as the instantaneous value of L_s in capacity calculations. However, during a nonlinear dynamic response analysis the instantaneous value of M/V at these locations varies wildly and should never be taken as instantaneous value of L_s. For this type of analysis the biaxial failure criterion, Eq. (3.84) in Section 3.2.3.8, should also be checked at each point of the response, with the instantaneous values of the two chord rotation components. For the other methods, a final biaxial check with the peak component values suffices.

The shear verifications in Table 6.1 may be limited to the highest LS being verified, as they will then be met by default at any lower Limit State. Note also that the ratio between the required chord rotation capacities at the Significant Damage (SD) and the Near Collapse (NC) Limit States (LSs) is constant and equal to 0.75. Chord rotation demands estimated via linear analysis are proportional to the Peak Ground Acceleration (PGA) of the seismic action. So, the SD Limit State will govern, if the PGA of the corresponding seismic action is less than 75% of that applying for the NC LS. Conversely, if it exceeds that value. So, checking both LSs is redundant. This conclusion may be extended to the use of nonlinear analysis, provided that the ratio of the PGA values for the SD and NC LSs is not close to 0.75.

6.5.7 Masonry Infills in Assessment and Retrofitting

According to Part 3 of Eurocode 8, wherever there are no specific provisions for masonry infills, the pertinent ones of Part 1 (CEN 2004a) apply. According to one of them, if walls take at least 50% of the base shear from a linear analysis, the interaction of the structure with the masonry infills may be neglected. This may be taken to imply that it is allowed then to disregard the infills in the structural model. However, this is not always a safe assumption. Other implications for the analysis derive from the CEN (2004a) requirements highlighted in Section 2.1.13.2, namely that infills with strongly asymmetric or irregular layout in plan should be included in a 3D structural model and a sensitivity analysis of the effect of the stiffness and position of the infills carried out (e.g., disregarding one out of three or four infill panels per planar frame). If the layout of infills in plan is not so asymmetric or irregular to warrant including them in the analysis model, the requirement in CEN (2004a) to double the accidental eccentricity in an analysis that neglects them applies for existing buildings as well. The provisions in CEN (2004a) about infills with irregular distribution in elevation (see Section 2.1.13.3) address the problem in a fully force-based context. So, they can only guide the decision about including or not in the structural model heightwise irregularly distributed infills. If the value of η from Eq. (2.7) is less than 1.1, their heightwise distribution is not a sufficient reason for including them.

For modelling of masonry infills the reader is referred to Sections 4.9.8 and 4.10.2. Recall the different values proposed in Section 4.9.8 for the effective width of the diagonal strut of solid infill panels at the three Limit States of CEN (2005a).

Masonry infills included in the structural model seem more like "primary elements" than "secondary" ones, as they do not play a role in the support of gravity loads and contribute only to lateral stiffness and strength. Infill panels are naturally classified as "primary members", if engineered into parts of the lateral-force-resisting system for retrofitting, e.g., by adding overlays of shotcrete or strong mortar with curtains of light reinforcement. Moreover, if confined at all four sides by a fairly strong concrete frame, they may exhibit remarkable deformation capacity (but offer little energy dissipation). So, they may well be checked in terms of deformations.

Notwithstanding their effectiveness, CEN (2005a) does not explicitly cover masonry infills engineered to be part of the lateral force resisting system. It only mentions masonry infills in general, in relation to the Damage Limitation (DL) Limit State, stating that at that LS they should be checked on the basis of their interstorey drift capacity. This makes sense for the following reasons:

- At the Near Collapse LS we cannot rely on non-engineered infills. Even when we include them in the model to take into account their potential adverse effects, we will not bother to check their integrity, as falling hazards are accepted at this LS;
- At the Significant Damage LS we may include non-engineered infills in the model to also take into account any benefits derived from their contribution to lateral strength and stiffness. Then we rely on their fairly large deformation capacity

without explicitly checking it, since non-structural elements may be sacrificed at this LS.

– One of the targets of the Damage Limitation LS is to ensure that damage to non-structural components is minor (for infill walls, distributed cracking) and can be easily and economically repaired later. So, masonry infills should be explicitly checked at this LS.

CEN (2005a) does not give specific interstorey drift limits for infills at the Damage Limitation LS. Note, though, that the interstorey drift checks required under the Damage Limitation seismic action in the framework of the "q-factor approach" (see Section 6.5.8) include indirect reference to the pertinent provisions of Part 1 for new buildings. It is inferred from this that the target protection of infills in existing buildings under the Damage Limitation seismic action is achieved, if we meet the interstorey drift limits specified in Part 1 of Eurocode 8 for new buildings (listed in Section 1.1.3 under (i) and (ii)). Indeed, limit (i) in Section 1.1.3 is physically reasonable for the onset of damage in ordinary masonry infills. So, infills can be verified for the Damage Limitation LS by checking limits (i) and (ii) in Section 1.1.3. Note that these limits (especially (i)) often govern member sizes in frame buildings designed for earthquake resistance according to CEN (2004a), even though they are checked using the default value of 50% the stiffness of the uncracked gross concrete section. With their smaller member sizes, existing frame buildings will have larger difficulty in meeting these limits under the same Damage Limitation seismic action, especially when the realistic stiffness values from Eq. (3.68) are used. They may very well meet them, however, if the contribution of infills to lateral stiffness is taken into account. So, we should not attempt to verify the interstorey drift limits (i) and (ii) in Section 1.1.3 under the Damage Limitation seismic action, unless the infills are included in the model for this level of seismic action. This is a good practical reason to consider even non-engineered masonry infills as "primary members".

6.5.8 Force-Based Assessment and Retrofitting (the "q-factor Approach")

Part 3 of Eurocode 8 allows also force-based assessment and retrofitting using the q-factor. The purpose is two-fold:

– To allow a positive seismic assessment for buildings that may have been (recently) designed in accordance with EN-Eurocode 2 (CEN 2004b) and EN-Eurocode 8 (CEN 2004a) – be it on the basis more of (over)strength than of ductility – and avoid, therefore, embarrassing situations where a structure designed as new on the basis of one part of the suite of EN-Eurocodes is formally rejected by another when considered as an existing one.
– To facilitate retrofitting buildings by adding a new lateral-load-resisting system capable of sustaining the full seismic action. The new system may be designed in full accordance with CEN (2004a), considering all existing elements

as “secondary” even when their total contribution to lateral stiffness might exceed 15% of that of the added system.

Note that, to claim that it is a Eurocode assessment and retrofitting, a force-based one should comply with all relevant design provisions for new buildings in CEN (2004a,b).

The “q-factor approach” is a two-tier seismic assessment and retrofitting similar to a new design according to the EN-Eurocodes. It is based on two Limit States:

1. Damage Limitation, checked exactly as for new buildings, i.e., on the basis of the interstorey drift limits listed in Section 1.1.3 under (i–iii); and
2. Significant Damage, identical to the No-(local-)collapse performance level for new buildings and checked with the criteria and procedures of CEN (2004a).

The parallel with the design requirements for new buildings stops at the definition of the seismic action, as it is not necessary to adopt for Limit States 1 and 2 above the hazard levels specified in the National Annex to CEN (2004a) for the “damage limitation” and the “design” seismic action, respectively. The National Annex to Part 3 (CEN 2005a) applies instead, which may leave to the owner and the designer the choice of these hazard levels.

As in new designs, most demanding is the fulfillment of Limit State 2 above. This entails a linear analysis with the design spectrum in CEN (2004a), Eqs. (4.5) in Section 4.2.2. There we are always entitled to use $q = 1.5$. For this q-factor value safety verifications are limited to checking that in every single “primary element” internal force and moment demands due to the design seismic action plus the concurrent gravity loads do not exceed the corresponding force and moment resistances.[7] These resistances are computed as in new buildings, except that for old materials mean strengths are used divided by the confidence factor. Like the nominal strengths of new materials, they are divided in the end by the material partial factor.

To use a value of q higher than $q = 1.5$, one has to show that the building has the corresponding local and global available ductility according to Part 1 of Eurocode 8:

- For retrofitting with a new lateral-load-resisting system capable of sustaining the full seismic action, one has to choose a Ductility Class from Part 1 of Eurocode 8 (DC M or H) and use the associated q-factor value. The new system and its connection to the old one should meet all the detailing and capacity design requirements for the chosen DC.
- To find out what q-factor value can be used for the existing system as is, possibly with some of its members retrofitted or with certain new elements added, one may first check one-by-one the end sections of all “primary elements” where Part

[7] “Secondary elements” are checked as in new buildings, see Section 4.12.3.

1 of Eurocode 8 would require detailing for ductility and determine the value of the curvature ductility factor, μ_φ, for each one from its detailing. More specifically, at the end sections of each "primary beam" Eq. (5.4b) is inverted for the available value of μ_φ. Equation (5.8) is also inverted for μ_φ at the base section (connection to the foundation) of each "primary column" (taking $\omega_{vd} = 0$) and "primary wall". The minimum of all these so-determined μ_φ-values is used then to invert Eqs. (5.2) for the basic value of the behaviour factor, q_o. This exercise should be carried out separately for each horizontal direction. By going back to Table 1.1 (and the rest of Section 1.4.3.1 for the default value of α_u/α_1), one may find out which DC the building (as is or retrofitted) may claim to belong for the specific lateral-load-resisting system of the horizontal direction considered. If, for example, the q_o-value is between those of DC M and H, the building might qualify for DC M. This preliminary conclusion on a single potential DC for both horizontal directions should be confirmed, by making sure that every single "primary element" and its connections meet the prescriptive detailing rules and the capacity design provisions (Eq. (1.4) if applicable, as well as in shear, etc.) pertaining to this DC (see Tables 5.1, 5.2 and 5.3). It is only after this is confirmed that one can determine the final q-factor value (incorporating any reductions for irregularity in elevation) and calculate the internal force and moment demands due to the design seismic action on the basis of the design spectrum. Needless to say, it is very unlikely that the as-is building or a moderately upgraded version of it will meet all detailing, capacity design and strength verification requirements for DC M buildings (let alone DC H). However, this exercise may point out (hopefully few) "primary members" that need to be modified or downgraded to the class of "secondary", for the building to qualify for a Ductility Class higher than L.

For conformity with Part 1 of Eurocode 8 and the background of Eqs. (5.2), (5.4b) and (5.8), the analyses for the "damage limitation" and the "design" seismic action, should use the default stiffness of 50% of that of the uncracked gross concrete section. Seismic action effects should not be reduced by including non-structural infills in the model.

6.6 Liability Questions in Seismic Assessment and Retrofitting

The designer and the contractor of a building normally share the liability for property damage, injuries or casualties in the event of an earthquake, subject of course to any statute of limitations. The picture is not so clear, once a new engineer assesses the building as adequate, according to the performance targets set by the owner who hired him/her and/or the applicable codes and standards. Besides, if the assessment is followed by retrofitting, the designer and the contractor who carry it out enter the picture as well. At first sight, the new people bear the full liability for the building, especially if the original designer and/or contractor are not available anymore or are protected by statute of limitations. However, if the original designer has chosen a poor structural layout, its effects on seismic response and performance cannot be fully reversed through retrofitting. Moreover, owing to cost constraints or time pressures, or to limit disturbance and damage to the building for material sampling and exposure of reinforcement, the campaign of in-situ measurements may not reveal potential serious flaws in materials or detailing, for which the original contractor is responsible.

It may be difficult for the owner to find a designer and a contractor prepared to bear the full responsibilty and liability for the actions of others. In some cases, the new engineer of the retrofitting may choose to protect himself from future liabilities by taking a very safe-sided attitude. He/she may recommend retrofitting a building that would normally be assessed as meeting the performance targets, and/or choose a heavy and unduly costly retrofitting strategy. In addition to being a waste of resources, such an attitude does not serve well the cause of seismic risk mitigation through retrofitting of existing buildings, as owners may in the end be discouraged to go ahead with the implementation of a prohibitively expensive retrofit design. Therefore, competent authorities should make sure that the applicable legal framework for liability is perceived by designers and contractors of retrofitting projects as fair and equitable. Targeted programmes for the reduction of seismic risk by retrofitting existing buildings may well include special litigation schemes to resolve potential liability cases for the buildings to be rehabilitated.

6.7 Retrofitting Strategies

6.7.1 General Guidelines

The aim of retrofitting is to modify the seismic demands, E_d, and/or the capacities, so that all relevant elements of the retrofitted building fulfill the general verification inequality, $E_d \le R_d$, at all performance levels ("Limit States") under the corresponding seismic action (see Sections 6.5.6 and 6.5.7 for Eurocode 8). This goal may be achieved by adopting one of the following approaches or strategies, or even combining them:

1. By reducing the seismic *demands* on members and the structure as a whole;
2. By increasing the member *capacities*.

Each strategy may be implemented using one or more retrofitting techniques (*fib* 2003, Thermou and Elnashai 2006, Thermou et al. 2007b). Techniques serving mainly Strategy no. 2 are described in Section 6.8, while some of those employed for Strategy no. 1 are highlighted in Section 6.9. Each technique has its own advantages and drawbacks, scope and limitations of use and fits better in one of these strategies. The choice of the technique depends on:

a. the locally available materials and technologies;
b. cost considerations;
c. the disruption of use it entails and the duration of the works;
d. architectural, functional and aesthetic considerations or restrictions, etc.

One or more retrofitting technique(s) are normally chosen on the basis of considerations (a–d), etc. This choice and how it is implemented determines which retrofitting strategy is being adopted.

Each retrofitting intervention is a special case, with more than one appropriate solutions. So, generalisation of rules is neither possible nor advisable. With this in mind, there are some general (but not absolute) guidelines to follow, depending on the outcome of the assessment of the as-built structure:

1. If there is general deficiency in the building, retrofitting Strategy no. 1 above is more cost-effective, as it can reduce the seismic demands throughout.
2. If there are capacity deficiencies in just a few scattered members, it is more cost-effective to focus on them and upgrade their capacities with retrofitting Strategy no. 2.
3. If the deficiencies are concentrated in a single or few ("weak") storeys, they may be due to a vertical irregularity. Retrofitting Strategy no. 2 is an option, to upgrade the capacities of the members of these storeys. Retrofitting Strategy no. 1 could be adopted instead, to remove the irregularity by adding strong and stiff new elements from the ground to the weak storey(s) and beyond, or to strengthen and stiffen existing elements there to overshadow the irregularity and suppress storey-sway mechanisms.
4. If the deficiencies are concentrated at a single side of the building, they may be due to a torsional imbalance in plan. It may be chosen to stiffen and strengthen existing elements on that side or add new ones there, to balance the stiffness and strength (retrofitting Strategy no. 1). Alternatively, the deformation capacity and the shear strength of the members of the "flexible side" may be upgraded, to accommodate the larger demands on them (retrofitting Strategy no. 2).

In buildings with a large surface area and strongly irregular and asymmetric structural layout, it may even be chosen to introduce vertical joints at selected locations in plan, converting the building to a number of structurally independent regular units. Vertical elements should be provided in that case at both sides of each joint, for independent support of the corresponding horizontal elements. The width of the joint should be sufficient to prevent pounding, especially if there is large disparity in lateral stiffness between the parts being separated. Conversely, if the building is already separated by vertical joints into a number of structurally independent but asymmetric in plan units, it may be decided to join them into an integral structure, by providing structural continuity across the joints. In this way we avoid pounding between the units during their strongly torsional seismic response. More important, it is easier to come up with a structure with an overall balanced in plan lateral stiffness and strength. An example is given in Section 6.10.2. Note that, by the time of the retrofitting the joints would have served their primary mission, notably to accommodate the shrinkage of concrete. With shrinkage practically complete, future movements at the joints due to daily or seasonal temperature cycles alone will be minor and, if suppressed, they will not induce high stresses in the horizontal elements of the integral building. Besides, if the connection across a joint takes place at ambient temperatures markedly below the yearly average, these stresses will be compressive most of the time.

Regardless of its type and extent, a retrofitting intervention should in no way impair the safety or capacity of any part of the building, e.g. by introducing irregularities in plan or elevation, by shifting the deformation demands to inadequate components or other failure modes, etc. Upgrading the moment resistance of a member should never make it critical in shear. Strengthening of beams should not shift plastic hinging to columns.

6.7.2 Reduction of Seismic Action Effects Through Retrofitting

In this strategy seismic deformation demands on existing structural or (drift-sensitive) non-structural elements are reduced below the corresponding capacities, which may remain unchanged. Absolute displacements are also reduced, decreasing the likelihood of pounding with adjacent buildings. Shear force demands cannot decrease, unless the member in question is kept in the elastic region.

This strategy lends itself better for a multi-tier performance-based rehabilitation. It can prevent member failures in rare, strong earthquakes, while limiting structural and non-structural damage in frequent, moderate ones.

The most effective and common means for the reduction of seismic deformation demands is by increasing the global lateral stiffness. Normally this brings about an increase in global lateral strength, which, however, should be seen as a by-product and not as the main target of the retrofitting. In order of decreasing effectiveness, global lateral stiffness may be increased by:

1. Adding a whole new lateral-load-resisting system to take almost the full seismic action. This system may consist of steel bracing (see Section 6.9.3), new concrete walls (Section 6.9.2), new moment frames, or combinations thereof. The new elements are normally placed at the perimeter, to facilitate their foundation and to limit disruption of use of the building (under certain conditions, operation may continue during retrofitting). The new system can overshadow completely any irregularities in plan or elevation. Critical elements in this approach are the foundation of the new lateral-load-resisting system and the connection to the existing system for the transfer of inertia forces. This approach lends itself to application of force-based retrofitting according to Section 6.5.8. In that case the new system is designed for ductility in full accordance with a code for new buildings, while the existing elements are considered as "secondary" and verified as such.

2. Adding new elements (new concrete walls, see Section 6.9.2, or steel bracing, Section 6.9.3), to supplement the existing structural system. The new elements may be used to advantage to balance a strongly asymmetric layout in plan, or to eliminate soft/weak storeys. If the contribution of the added elements to lateral stiffness is large, this approach may be considered as a scaled-down version of approach 1. Then what has been said for it above still holds, except the applicability of Section 6.5.8 for force-based retrofitting.
3. Converting non-structural infill walls into structural elements, integrating them with the surrounding frame. The approach has many aspects in common with 3 above, but is not covered at all in this chapter, because there is not sufficient technical basis for the verification of infill walls as "primary" structural elements. Note that, if the overlays added to the infills contain curtains of light reinforcement, detailing aspects (e.g., the connection or anchorage of the reinforcement to the frame and how it affects the behaviour and the verifications; corrosion protection of the reinforcement, in view of the small thickness of the overlay, etc.) become very important.
4. Concrete jacketing, mainly of columns. This is closer to retrofitting for the purposes of increasing the capacities. Only when practically all columns are jacketed (e.g., when architectural reasons do not allow adding new walls or steel bracing, or when there is wide-spread reinforcement corrosion), it might also be considered as part of a strategy to reduce seismic demands, albeit not the most cost-effective and less disruptive one.

Depending on the case, the engineer may use approaches 2, 3 or 4 in the same retrofitting. The stiffening (and strengthening) should not be discontinued vertically at a level below the top, without considering the potential concentration of damage just above that level.

Reduction of mass is another means of reducing deformation and displacement demands, as in the pseudo-velocity controlled region of the spectrum, where the effective fundamental period of concrete buildings normally falls, seismic displacement demands are about proportional to the square root of the total mass. Except in special cases (e.g., when there is a single large mass at the upper floors), this approach is marginally effective by itself, but can be used to advantage in combination with other techniques or approaches. It can be implemented by removing heavy items (e.g., water tanks, heavy pieces of equipment, storage loads), by replacing heavy floor (or roof) finishings, cladding or partitions with lighter ones, or even by demolishing one or more top floors. Demolition of penthouses and upper storey setbacks also removes extreme irregularities in elevation and is sometimes worth considering. If the deficiencies identified in the existing structure are marginal, reduction and removal of masses throughout the building or, in extreme cases, complete removal of the upper storey(s), may make seismic upgrading of the rest unnecessary.

The introduction of base isolation and energy dissipation is also a means to reduce seismic deformation demands. These two techniques are not covered in this book at all, even for new buildings. For seismic upgrading, they are best suited for bridges, where they often represent by far the best option. However, for existing buildings normally they are not cost-effective. This is more so for base isolation, as inserting the isolation devices is technically challenging: one needs to cut one-by-one the vertical elements at the base, while jacking up the superstructure around them.[8] Base isolation provides not only safety to the building and its occupants under very strong and rare earthquakes, but also protection of building contents under any earthquake event. Therefore, if the facility is required to remain operational during the earthquake or be available for immediate occupancy afterwards, isolation may be the most cost-effective strategy, provided that the building is not slender but stocky (to avoid axial tension on the devices, due to the overturning moment) and the superstructure has large stiffness (to be considered as rigid, compared to the isolation system). At any rate, base isolation is a sophisticated and complex technique and its application requires not only specialised expertise, but also peer review of the design.

For energy dissipation to be effective, significant lateral seismic displacements are necessary. So, it can only be used in flexible structures, as a supplement of another system which does not significantly increase the global stiffness. Dissipation devices can be used together with base isolation, or can be inserted in braces of an added steel bracing system. In this latter case the displacement demands for the activation of the dissipation system are not concentrated at the base (isolation) level but distributed throughout the structure, possibly causing significant damage to existing structural and non-structural components. This technique also is sophisticated, requires specialised expertise and peer review of the design, and is costly, but less so compared to base isolation.

6.7.3 Upgrading of Member Capacities

The deformation capacity and shear strength of individual members may be significantly upgraded by FRP-wrapping, without modifying at all their stiffness. Concrete jackets also improve deformation capacity and shear strength, but increase stiffness as well. So, when applied to many elements they also reduce deformation demands, not only locally but also globally. Improvement of certain details (e.g., of poor connections between the floor diaphragms and the lateral-load-resisting system or within diaphragms, see Section 6.7.4) may also be considered to belong to this retrofitting strategy.

Modification of existing components uses up less floor area and does not require closing openings. So it is, in general, more convenient for the future functionality of the building than the addition of new elements, or of a large concrete volume to existing members to increase stiffness and reduce seismic demands. However, it may entail removal and replacement of finishing materials and often partial

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[8] A double foundation, sandwiching the isolation system, would normally be used in a new building.

demolition and reconstruction of partitions, lengthening the work and increasing its cost. Besides, modification of interior elements may disrupt use of the building. So, retrofitting via enhancement of member capacities alone makes more sense when deficiencies are limited to few members or connections or to part of the structure (a storey or one side in plan). It may also be adopted when it is not feasible to add new elements (e.g., because of architectural constraints) and/or provide a proper foundation for them.

Unless very specific and substantial deficiencies are identified in some beams, upgrading of existing members may be limited to vertical elements, possibly including their joints with the beams. Due to the integral connection of the beams with the slabs, upgrading a beam is technically more difficult than upgrading a column or a wall. Besides, experience from past earthquakes shows that damage in beams is far less common than in columns and its impact on global stability minor. Moreover, the design of beams for gravity loads normally provides sufficient top reinforcement at the supports (supplemented by the slab bars within a sizable effective flange width) and substantial shear reinforcement in the form of stirrups which are closed at the critical side (the bottom one). What is missing in such beams is continuity and anchorage of bottom bars over the supports. However, bar pull-out under sagging moments, if it occurs, only increases the lateral deformability of frames. Another weak point is the poor deformation capacity of the bottom flange in compression, in plastic hinges under hogging moments, which however, not always take place. Note that concrete jacketing of the columns into which a beam frames, improves, albeit indirectly, anchorage of its bottom bars and confinement of its bottom flange. Last but not least, the main hazard for existing buildings is posed by too much, rather than by too little, moment resistance of beams with respect to columns.

Existing components may sometimes be modified to improve not their own performance but that of elements they connect to. For example, a weak-beam/strong-column combination may be achieved by cutting beam longitudinal bars at the support by the column, provided that this is acceptable for gravity load resistance. Captive columns may also be set free by severing their connection to spandrel walls.

6.7.4 Completeness of the Load-Path

No matter which retrofitting strategy and technique he/she chooses, the engineer should check carefully the retrofitted structure for continuity of the load path(s). Transfer of inertia forces from the masses to the (primary) elements of the lateral-load-resisting system and from there to the foundation should be ensured. Note that inertia forces that may need to be transferred are proportional to peak floor accelerations, which are increased by global strengthening and stiffening.

Any connection within the floor system, between the floors and the lateral-load-resisting elements and between existing and new members should be checked in terms of forces, for the maximum possible demands that it may be required to transfer. Connectors and fasteners may exhaust their deformation capacity soon after yielding, as they are called to accommodate within their small size significant relative displacements of the components they connect. So, even though they normally consist of a ductile material (steel), they should be checked in terms of forces and protected from yielding. Connections likely to be subjected to cycles of tension and compression may fracture under forces below their nominal tensile capacity, if they buckle or are severely deformed in a previous compression half-cycle. Welded or bolted connections are inherently brittle. Steel parts in an existing connection that appear adequate in construction documents may have corroded in the mean time.

Connections between prefabricated elements, especially in diaphragms consisting of precast units, are potentially weak links in the load path. Thin and lightly reinforced toppings in precast floors or roofs may already be cracked over seams between the precast units, or may easily crack during the earthquake and break open. It is difficult and not cost-effective to ensure integrity of a precast floor or roof, topped or untopped, through retrofitting. It is better to replace such a floor with a proper cast-in-place concrete floor or roof, integral with the vertical framing elements.

Cast-in-place slabs are normally considered to provide a continuous load path. However, one-way slabs in old buildings may have little reinforcement in their secondary direction. Floors with one-way slabs may break open through the points of inflection of the supporting beams under gravity loads, because in old days beam longitudinal reinforcement was dimensioned without the shift rule and had short anchorage. So, it was curtailed near the inflection point, as this came out from the analysis for gravity loads.

6.8 Retrofitting Techniques for Concrete Members

6.8.1 Repair of Damaged Members

6.8.1.1 Scope of Repair Techniques

For completeness, repair of seismic damage without strengthening is addressed before going in detail in the application and design of various means of modifying existing members. Nowadays seismic damage almost invariably triggers upgrading of the deficient earthquake resistance of a building through global seismic retrofitting. So, the subject of repair is of interest only for those members which are not upgraded, but just restored to their pre-earthquake condition.

Retrofitting enhances one or more properties of a member which are important for its seismic behaviour and performance. By contrast, the target of repair is just to re-instate some original characteristics that may have degraded because of age and/or adverse environmental influences, or owing to an earthquake or other damaging event. When such deterioration or damage is minor to moderate, repair may be sufficient, provided that any necessary global upgrading of earthquake resistance is provided elsewhere or by other means. The effects of severe damage, with disintegration of concrete inside the stirrups and/or buckling or fracture of the reinforcement, cannot be fully redressed by simple repair. Repair is then usually followed by certain upgrading of the damaged zone (e.g., through jacketing with concrete of FRP), overshadowing any residual effects of the damage.

Repair may comprise one or more of the following:

- Replacement of buckled or fractured bars.
- Injection of cracks with epoxy or sometimes grout.
- Replacement of concrete that is loose, or has spalled, or has been removed to replace bars.

These measures are also applied, as appropriate, to damaged members before retrofitting with concrete or FRP jackets. In such cases it is not essential to inject fine cracks (e.g., less than about 0.3 mm wide), while spalled or removed concrete may be replaced with the cast-in-situ concrete or the shotcrete of the jacket.

Some practical information about these repair measures is given below.

- Replacement of reinforcing bars: Longitudinal bars that have fractured or suffered visible buckling are often replaced over a certain length. After removing the concrete around this length, the old bar is cut and its two ends are butt- or lap-welded to a new piece of bar. This is not a trivial operation and should be undertaken only when the old bar is essential. Otherwise, we just do not rely on it anymore.
- Replacement of concrete: Any loose concrete should be removed. If a bar is broken or has buckled, or has been partially exposed owing to spalling or disintegration of concrete, any still sound concrete around the length affected should also be removed to provide sufficient room for the repair mortar to surround the bar. The replacement material is commonly epoxy- or cement-based nonshrink grout mortar, with sand, pea-gravel or coarser aggregates, depending on the size and depth of the cavity. The mortar is normally trowelled against the substrate or against the previous mortar lifts, without applying an epoxy-based bonding agent as a primer. Formwork is redundant.
- Injection of cracks: If properly carried out, injection fully reinstates the continuity of the material and the tensile strength and cohesion across a crack. A common low-viscosity epoxy can be used to fill cracks wider than about 0.1 mm and up to few (2 or 3) mm. Cracks narrower than 0.2–0.3 mm may not be worth injecting, as the depth of penetration of the epoxy is uncertain and the epoxy is fairly expensive. For crack widths from 2 or 3 mm to 5 or 6 mm, medium-viscosity epoxies are more appropriate. Wider cracks, up to 20 mm, should better be grouted with cement.
- Before injection or grouting, loose material is removed from the trace of the crack on the concrete surface. For epoxy injection, the trace is fully sealed with epoxy paste, leaving only surface-mounted plastic nozzles as injection ports. Ports should be not further apart along the crack than the distance the epoxy can travel before it hardens. This distance depends on the width of the crack and the viscosity of the epoxy at the application temperature. If the member is thicker

than this distance, injection should take place through cracks on opposite faces. The injection starts from the lowest level where a crack appears in the member and continues upwards. It stops when the epoxy bleeds out from another port. The current port is then sealed by bending and tying the nozzle and injection continues from the next port. In the end, after the epoxy fully sets, the ends of the nozzles may be cut flush with the finished surface of the repaired member.

The procedure for cement grouting is similar, but the injection pressure is much lower.

6.8.1.2 Effectiveness of Repair for Strength, Stiffness and Deformation Capacity

There are very few cyclic tests in the literature on RC members repaired and retested without strengthening (Fardis et al. 2005). In these tests repair was limited to replacement of the crushed concrete shell of the plastic hinge with epoxy mortar, non-shrink concrete, fibre-reinforced concrete, etc. Data on 15 such tests on walls and 18 more on columns are given in (Fardis et al. 2005). The expressions in Section 3.2 may be applied to the repaired member, assuming that the strength of the repair concrete used in the shell of the plastic hinge (typically higher than that of the original concrete) applies to the whole element. The test-to-prediction ratio for the yield moment M_y (from Section 3.2.2.2) has median values of 1.005, 1.035 and 1.015, for columns, walls or overall, comparing well to those quoted for virgin members in Section 3.2.2.2 under *Comparison with Experimental Results and Empirical Expressions for the Curvature*, but the coefficient of variation of about 26% is markedly higher. The medians of the test-to-prediction ratio for the chord rotation at yielding θ_y (from Eqs. (3.66)) are 1.26, 1.66 and 1.27 for columns, walls or overall, while the corresponding coefficients of variation are 24, 40.5 and 38%. The test-to-prediction ratio for the secant stiffness to the yield-point (from Eq. (3.68)) has medians of 0.79, 0.54 and 0.725 for columns, walls or overall and coefficients of variation of 32.5, 58.8 and 45%, respectively. For 15 of the repaired columns carried to flexural failure, the test-to-prediction ratio for the ultimate chord rotation, θ_u, has median values of 0.675, 0.72, 0.705 or 0.705 and coefficients of variation of 32.5, 32.5, 32.5 or 54%, for θ_u computed from Eqs. (3.78a), (3.78b), (3.78c) or (3.72), (3.73) and (3.74), respectively. The corresponding medians for the 15 walls are 0.7, 0.695, 0.825 or 0.97 and the coefficients of variation 55%, 58.5%, 53% or 58.5%. For the total of 30 repaired specimens carried to flexural failure, the test-to-prediction ratio for θ_u has median values of 0.69, 0.71, 0.74 or 0.805 and coefficients of variation of 43.5, 45.5, 43 or 57.5%, for θ_u computed from Eqs. (3.78a), (3.78b), (3.78c) or (3.72), (3.73) and (3.74), respectively.

Although based on limited data, the above comparisons show that, even when carried out as carefully as in a research lab, repair re-instates fully only the yield moment (and hence the moment resistance), failing by 25–30% to recover the secant stiffness to the yield-point and the deformation capacity. Interestingly, repaired walls exhibit much larger loss of stiffness than repaired columns, but they fare a little better than columns at ultimate (although the difference is statistically insignificant). Although the small sample size normally reduces the apparent scatter, the dispersion of test results with respect to predictions is much larger than in virgin specimens, even for the yield moment which is recovered well on average. Apparently, not only the repair process and materials, but also the type and degree of the original damage, introduce significant additional uncertainty.

The final conclusion is that, no matter how carefully it is done, mere repair cannot be considered to fulfil its intent, notably the re-instatement of key properties of the member. It is especially disconcerting in the context of displacement-based assessment and retrofitting that, the repaired members will be subjected to increased seismic displacement and deformation demands than similar virgin ones, owing to their loss of stiffness, while having more difficulty copying with them, given their reduced deformation capacity. So, unless global measures are taken to drastically reduce seismic displacement demands, members that suffered major damage or failed in the first event, will do even worse in a future one, despite the repair.

6.8.2 Concrete Jacketing

6.8.2.1 Introduction: Advantages and Disadvantages of Concrete Jackets

Owing to their cost-effectiveness, concrete jackets are still the method of choice for seismic upgrading of individual concrete members. There are several reasons:

– Every engineer or contractor is familiar with the field application of concrete. Recall in this connection that retrofitting, and especially modification of existing members, does not lend itself to (even partial) prefabrication in shop. So concrete is the prime candidate, as it is the most common structural material for field fabrication and application.
– Concrete jacketing is the most suitable technique for retrofitting severely damaged members, as crushed and removed concrete is replaced while casting or shotcreting the jacket, while buckled or fractured bars do not need to be fully restored if replaced by the new reinforcement of the jacket. This aspect was even more important in the past, as only recent years have seen wide application of seismic retrofitting without seismic damage as the trigger.
– Structural concrete is versatile and can adapt to almost any shape, e.g., in order to fully encapsulate existing members and joints and provide structural continuity between different components (between a joint and the adjoining members, between members in adjacent storeys, etc.).
– A concrete jacket can, through the appropriate reinforcement, have multiple effects. It is the only means to improve at the same time:

  1. stiffness,
  2. shear strength,
  3. deformation capacity,

4. anchorage/continuity of reinforcement in anchorage or splicing zones,
5. moment resistance (even turning a weak-column/strong-beam frame into a strong-column/weak-beam one),
6. shear strength and bond in joints through which the jacket continues, and
7. protection of the old reinforcement from (further) corrosion.

– Stiffness and flexural resistance are enhanced by the increased cross-section and the added longitudinal reinforcement, which – very importantly and unlike for other retrofit techniques of individual components – can easily extend beyond the member end into and through the joint. The main contribution to shear strength, deformation capacity and anchorage or splicing of reinforcement comes from the added transverse reinforcement, which works in shear, against buckling and for confinement. The added concrete is also a factor there. The increased dimensions of a joint when a jacket continues into and past it, provide more length for bond along old bars going through the joint and improve the joint shear strength. They also make room for adding transverse reinforcement in the joint. Finally, if the jacket concrete is of sufficiently low porosity, it can prevent or arrest corrosion of old reinforcement even in carbonated concrete. As a minimum, it reduces markedly the mechanical and aesthetic consequences of any corrosion that may go on.

Concrete jacketing may be considered to serve at the same time both types of retrofitting strategies in Section 6.7. By increasing the stiffness (item no. 1 above) it reduces seismic displacement and deformation demands. Besides, it is very effective in enhancing the force and deformation capacity of the jacketed member (items no. 2–7).

From the technical point of view, the multiple effectiveness of concrete jackets is what mainly distinguishes them from the other seismic retrofitting techniques for individual concrete members, which cannot readily extend beyond the member end and into a joint region. Other techniques mainly enhance some or all of the properties no. 1–4 above, but normally not the flexural strength, the resistance of the joints themselves or the corrosion protection all along the member (items no. 5–7).

RC jackets have certain handicaps, compared to other member modification techniques:

– They considerably increase member cross sectional dimensions. This may be a serious drawback when space, especially floor area, is at a premium.
– They normally cause the largest inconvenience and the lengthiest disruption of occupancy, produce the largest amount of dust and debris (especially if shotcrete is used) and cause the most noise pollution and safety or health hazards for the workers.

As the factors where concrete jackets are at disadvantage to the competition become more and more important, the balance may soon turn against them, notwithstanding their present and future advantage in direct construction cost.

6.8.2.2 Detailing, Technological and Construction Aspects

The concrete overlay of the jacket should be at least 75–100 mm, to provide sufficient cover of the new reinforcement and space for 135°-hooks at tie ends (Fig. 6.1(a)). For this range of thickness shotcrete is more convenient. Thicker overlays are normally cast-in-place.

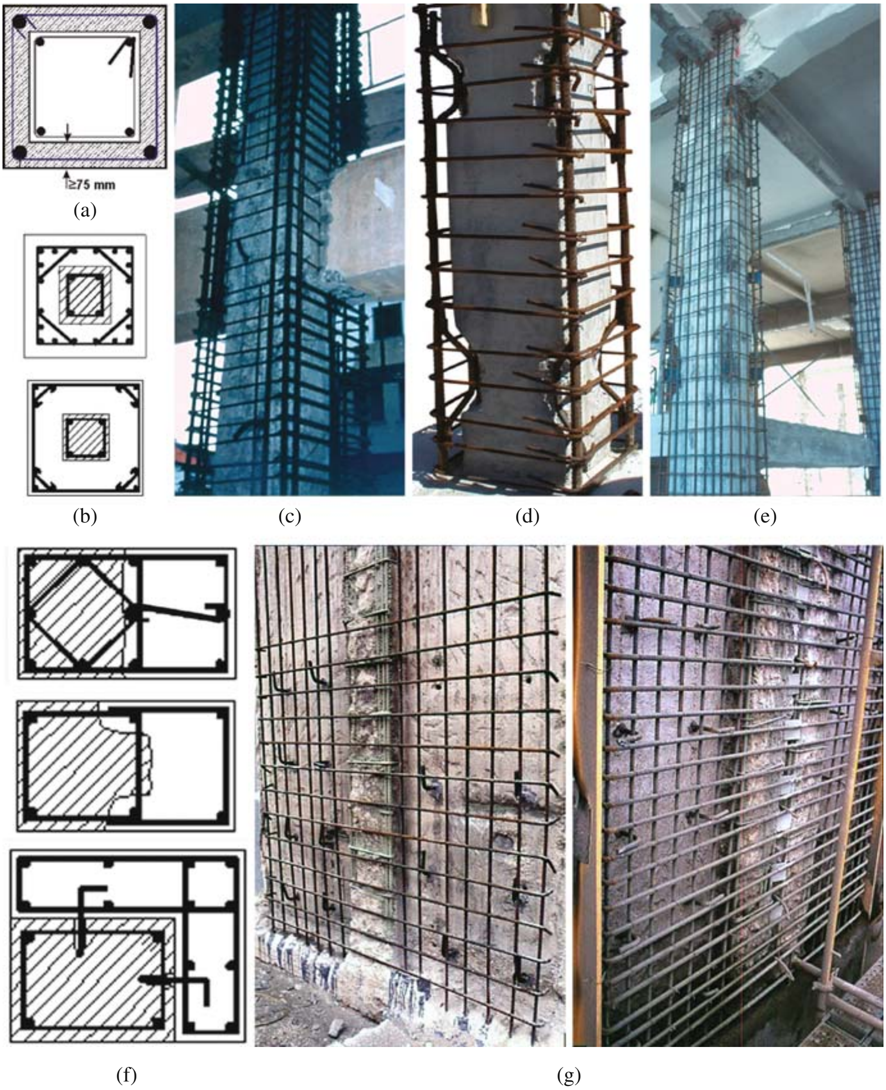


Fig. 6.1 Concrete jackets in columns: (**a**) simplest case; (**b**) jacket bars bundled near corners, engaged by cross-ties or octagonal tie; (**c**) jacket bars bundled at corners, dowels at interface with old column; (**d**) U-bars welded to corner bars; (**e**) steel plates welded to corner bars; (**f**) one- or two-sided jackets; (**g**) one-sided concrete overlay with single curtain of two-way reinforcement at exterior face of perimeter walls (cf. Section 6.10.2)

For the moment resistance of vertical elements to increase, longitudinal reinforcement should be continued to the adjacent storeys through holes or slots in the slab. To avoid perforating the beams on all sides of the cross-section into which a beam frames, jacket bars continuing through the slab should be concentrated near the corners of the new section, often in bundles (Fig. 6.1(b) and (c)). Jacket vertical bars may be anchored into a foundation element either:

– by enlarging the foundation element to accommodate anchorage of the jacket bars in new concrete there (possibly increasing at the same time the capacity of the foundation element to meet the larger moment demands from the jacketed vertical member), or
– by fastening (e.g. through epoxy) starter bars within vertical holes drilled in the foundation element, to be lap-spliced with the jacket vertical bars outside the plastic hinge that may form at the bottom of the retrofitted element.

If the target of the concrete jacket is just to enhance the deformation capacity of the member by offering confinement and anti-buckling action to the old member, to increase the shear strength and to remedy deficient lap splices without increasing the moment resistance, then the jacket does not need to continue past the joint into the next storey for a column or the next span for a beam. A gap of about 10 mm is recommended there, to avoid increasing indirectly the moment resistance of the member and, hence, the shear force demands in the member itself and the joint. Concentration of flexural deformation demands in a few-millimetre-length of the old member is not a concern. The gap serves there as a pre-existing wide crack. Within its length the compression zone in the old member is effectively confined by the concrete beyond the gap. At any rate, FRP jackets lend themselves better than RC ones for shear strengthening, enhancement of deformation capacity and improvement of deficient lap-splices without flexural strengthening.

A closed perimeter tie around the vertical bars of a column jacket restrains them against buckling, adds shear strength and confines the concrete. If we need multiple ties but do not want to drill holes and thread cross-links through the core of the old column, we can supplement the perimeter tie with an octagonal hoop outside the old column, which restrains buckling of any vertical bars close to, but not exactly at the corner. Instead of the octagonal tie, short corner ties at 45° to the perimeter may be used, engaging in 90–135° hooks the two bars adjacent to the corner bar (Fig. 6.1(b)). A diamond-shaped tie can be used only when the side of the jacketed column is at least twice as long as that of the old column. However, it is meaningless unless the jacket has mid-side bars to be restrained.

Confined boundary elements can be added to poorly detailed shear walls by jacketing the edges of the cross section. The ties around the new boundary element can come in two pieces: a straight one driven through a hole drilled in the web of the wall and a U-shaped piece around the edge of the wall, lap-welded to the first. A concrete overlay over one or preferably both sides of the web, with a curtain of horizontal and vertical bars, can provide additional shear strength.

Three-sided jackets are sufficient for beams integral with the slab. However, one-, two-, or three-sided jackets not fully surrounding an old column are much less effective than full jackets. Besides, their seismic behaviour is little known. If a full jacket around a column is not feasible, the old reinforcement should be exposed and the new ties welded to the old ones or bent around the old vertical bars (Fig. 6.1(f)). One-sided jackets are easy to add to the exterior face of perimeter members in a building. If the member is a wall, a one-sided jacket having full horizontal and vertical reinforcement and well connected to the old wall through dowels may play the role of an appropriately reinforced new wall (Fig. 6.1(g)). The main contributions of the old wall are its concrete and the connection it provides to the rest of the structural system and to the foundation for the transfer of seismic forces.

Past guidance documents for concrete jackets – and past practice alike – include measures for shear connection of the old and the new concrete. Connecting the (corner) bars of the jacket to the (exposed for this purpose) longitudinal bars of the old column, by lap-welding both to Z- or U-shaped steel inserts is commonly recommended and applied (see Fig. 6.1(d) and (e)). Alternatively, the surface of the old element may be roughened and/or dowels may be driven into it (Fig. 6.1(c) and (g)). The dowels are epoxy-grouted in holes drilled into the old element and protrude in the overlay of new concrete for almost its full thickness. As this thickness is usually small, the dowel is often bent at 90° for anchorage in the new concrete.

Welded steel inserts between the new and the old corner bars, as well as epoxy-grouted dowels, were perceived in the 1970–1980s as a means to engage the jacket in sharing the axial force of the column through shear forces in the welded steel inserts and the dowels. The concern about gravity load capacity was motivated by the past use of concrete jackets mainly – if not only – to repair heavily damaged columns, whose core had often partially disintegrated. This concern is reflected in past recommendations to use props, wedges and even jacks under the beams framing into the column, to relieve it from part of its axial load before jacketing. For undamaged or moderately damaged columns these concerns are not warranted. Experimental work has demonstrated that concrete columns subjected to large post-ultimate drifts and heavy damage in the concrete core can retain a large part of their gravity load capacity (Elwood and Moehle 2001).

As we will see in Section 6.8.2.3, welding the new corner bars to the old ones through steel inserts may improve the column cyclic chord rotation capacity (possibly because it delays or prevents bar buckling). However, positive connection of steels of different grade (and composition) may promote corrosion. So, it is not recommended here to connect the old and the new longitudinal bars by welding both to pieces of steel in-between.

Dowels at the interface have a larger beneficial effect on the ultimate chord rotation of the jacketed column and do not seem to have collateral negative effects. If placed, they have a geometric ratio about equal to $0.2f_{\mathrm{ctm}}/f_{\mathrm{yk}}$, which gives about 18 mm or 20 mm-dia dowels at 500 mm centres. The designer may choose to use dowels or not, taking into account their additional cost and time requirements. He/she may consider cost-effective to rely on friction alone for the shear at the interface, without connecting positively the new and the old concrete.

Friction is enhanced by the compressive stress building up normal to the interface, as the old member restrains shrinkage of the concrete overlay in the radial and circumferential directions. The restraint induces radial compressive stresses in the old and the new concrete and compressive circumferential ones in the old member and the ties, new or old, but tensile circumferential stresses in the jacket. This amounts to certain "active" confinement of the old element, even before any lateral loading. To get an idea of the magnitude of these stresses and of their dependence on various parameters, we consider for simplicity the old member as circular with radius R_o. A final total (drying plus autogenous) shrinkage strain ε_{cs} in a jacket with thickness t_j induces radial compression normal to the interface (confinement stress for the old member) equal to:

$$\sigma_r = \frac{\varepsilon_{cs}}{\left(1+\nu+\dfrac{2\left(\dfrac{R_o}{t_j}\right)^2}{1+2\dfrac{R_o}{t_j}}\right)\dfrac{1+\varphi_{\infty,j}}{E_{c,j}}+\dfrac{1}{\dfrac{E_{c,o}}{(1-\nu)\left(1+\varphi_{\infty,o}\right)}+\dfrac{E_sA_{sw}}{s_wR_o}}} \tag{6.10}$$

where $E_{c,o}$ and $E_{c,j}$ denote the Modulus of the old concrete and of the jacket, respectively, $\varphi_{\infty,o}$ and $\varphi_{\infty,j}$ their final creep coefficients[9] and ν the Poisson ratio of concrete. A_{sw}/s_w is the total cross-sectional area (old and new) of transverse steel per linear meter of the member, lumped for simplicity at the interface. Equation (6.10) applies only if the jacket does not crack under the accompanying circumferential tensile stress, which is equal to:

$$\sigma_t = \left(1+\dfrac{2\left(\dfrac{R_o}{t_j}\right)^2}{1+2\dfrac{R_o}{t_j}}\right)\frac{\sigma_r}{1+\dfrac{(1-\nu)\left(1+\varphi_{\infty,o}\right)}{E_{c,o}}\dfrac{E_sA_{sw,j}}{s_{w,j}R_o}} \tag{6.11}$$

In this case $A_{sw,j}/s_{w,j}$ refers to the transverse steel of the jacket alone, still taken for simplicity near the interface. In thin jackets around large members the stress from Eq. (6.11) eventually exceeds the jacket tensile strength, $f_{ctm,j}$. Cracks may then start along planes normal to the interface and parallel to the member axis. This will reduce the stress normal to the interface to almost zero. By contrast, for typical parameter values Eq. (6.10) gives normal stresses in the order of 1 MPa, which can markedly improve friction at the interface. A large percentage of transverse reinforcement in the jacket increases the compression from Eq. (6.10) and delays cracking of the jacket in the circumferential direction (see Eq. (6.11)). So, its role is vital for friction.

A rough interface enhances friction. For example, in a 1:1.5 scale two-storey, one-bay frame tested in Stoppenhagen et al. (1995) with sizeable concrete jackets around the heavily damaged columns of the original test specimen the interface

[9] The values of $\varphi_{\infty,o}$ and $\varphi_{\infty,j}$ depend on the age of the old concrete and the jacket, respectively, at the time shrinkage starts. For a cast-in-situ jacket this is the age at stripping of the formwork; for shotcrete it is zero.

was just roughened. The jacketed frame sustained storey drifts of 1.25% (which, if they are due to the columns alone, give a drift ratio of the clear column length of over 4%) without loss of the column force resistance and with apparent monolithic behaviour of the jacketed column. However, as we will see in Section 6.8.2.3, artificial roughening of the old surface is not essential. Monolithic behaviour of the jacketed columns and beams was apparently achieved in the tests in Alcocer (1992) and Alcocer and Jirsa (1993), even though no positive measures were taken to improve the shear capacity of the interface. The excellent performance of these specimens deserves special mention. The tests were carried out on four 1:1.5-scale 3D beam-column subassemblies retrofitted with column jackets (continuous through the joints), or – in one test – with beam and column jackets. The retrofitted subassemblies developed a cyclic lateral force resistance at a storey drift of 4% between 3.5 and 6 times that of the unretrofitted companions. In the retrofitted specimens the beams hinged, with a large part of the slab reinforcement fully contributing to the tension flange of the beam. Joint shear was critical, but did not lead to a drop in resistance even under bi-directional load cycles. No bond problems were observed along the length of beam or column bars within the joint, although it was limited to 18 bar-diameters (or 10 equivalent bar diameters for the bar bundles) in the columns and to 23 bar-diameters in the beams.

6.8.2.3 Strength, Stiffness and Deformation Capacity of Members with Concrete Jackets

The cyclic behaviour of a concrete-jacketed member to and beyond yielding and up to ultimate deformation is fairly complex, because it depends on the conditions at the interface of the jacket and the old member, etc. (Thermou et al. 2007a). This complexity notwithstanding, the dimensioning tools for practical retrofit design should be (almost) as simple as those for the design of new members. The so-called "factors of monolithic behaviour" have long been a popular means to this end (CEN 1996). They are conversion factors applied on the strength, stiffness, etc. of an "equivalent" monolithic member to approximate the corresponding property of the composite jacketed one. Values often used for these factors are based on scant test data, sometimes limited to a single experimental study comparing concrete-jacketed members to a monolithic reference specimen. Moreover, the conversion factor needed for practical retrofit design is not one to be applied on the "real" (i.e., experimental) value of the property of the monolithic member, which is also unknown. It should be a factor that multiplies a monolithic property computed by simple, yet fairly reliable means. The comprehensive portfolio of simple tools presented in Section 3.2 for the estimation of strength, stiffness and deformation capacity of monolithic concrete members may well serve as the reference for these conversion factors. To this end Fardis et al. (2005) and Biskinis and Fardis (2009) have compared the experimental strength, stiffness and deformation capacity of about 55 jacketed columns or walls from the literature (about 35 of which carried to flexure-controlled ultimate conditions) to those of an "equivalent" monolithic member, determined according to the rules in Sections 3.2.2.2, 3.2.3.2, 3.2.3.3 and 3.2.3.5 in accordance with Table 6.2.

Table 6.2 Characteristics of monolithic member considered "equivalent" to the jacketed one

I. Flexural resistance and deformation capacity, deformations at flexural yielding		
Case A: The jacket longitudinal bars are anchored beyond the member end section		
A1	Dimensions	The external dimensions of the section of the "equivalent" member are those of the jacket
A2:	Longitudinal reinforcement	The tension and the compression reinforcement are those of the jacket. The longitudinal bars of the old member are considered at their actual location between the tension and compression bars of the jacket: – they may supplement any longitudinal bars of the jacket between the tension and compression reinforcement and be included in a "web" reinforcement ratio, considered as uniformly distributed between the extreme layers of reinforcement; – in a wall, the tension and compression reinforcement of the jacketed member may be taken to include old vertical bars at the edges, as appropriate. Lap splices in the intermediate old reinforcement may be neglected. Any difference between the yield stress of the new and the old longitudinal reinforcement should be taken into account in all cases.
A3	Concrete strength	The f_c value of the jacket applies over the full section of the monolithic member, except for the 3rd term of Eqs. (3.66), where the f_c value of the concrete into which the longitudinal bars are anchored beyond the end section is used.
A4	Axial load	The full axial load is taken to act on the jacketed column as a whole, although it was originally applied to the old column alone.
A5:	Transverse reinforcement	Only the transverse reinforcement in the jacket is taken into account for confinement.
Case B. The longitudinal bars of the jacket stop at the end section		
B1	Dimensions, longitudinal reinforcement, concrete strength	M_y and φ_y (also in the 1st and 3rd term of Eqs. (3.68)) are calculated using the cross-sectional dimensions, the longitudinal reinforcement and the f_c value of the old member, neglecting any contribution from the jacket. The effect of lap splicing of the old bars is taken into account according to Section 3.2.3.9. The section depth h in the 2nd term of Eqs. (3.68) is that of the jacket.
B2	Transverse reinforcement	The deformation capacity, θ_u, is calculated on the basis of the old column alone, taken as confined by the jacket and its transverse steel. Confinement in Eqs. (3.78) or (3.72), (3.73) and (3.74) is taken into account with an effectiveness factor $a_s = 1.0$ and the value of $\rho_s = A_s/b_w s_h$ calculated using the value of A_s/s_h in the jacket and the width of the old column for b_w.
II. Shear resistance		
Shear resistance (even that without shear reinforcement, $V_{R,c}$, from Eq. (3.67), to determine the value of a_V in the 1st term of Eqs. (3.66)) and anything having to do with shear are calculated on the basis of the external dimensions and the transverse reinforcement of the jacket. The old transverse reinforcement may be considered to contribute to shear resistance only in walls, provided it is well anchored into the (new) boundary elements.		

The idea behind assumptions A3 and A4 in Table 6.2 is that, for common ratios of jacket thickness to depth of the jacketed section, when yielding takes place and a plastic hinge forms at the end section of the member, the compression zone there is almost fully within the jacket, carrying the full axial load. Also, it is the jacket that mainly governs shear resistance and bond along the longitudinal reinforcement of the jacket.

An asterisk is used here to denote a calculated value for the jacketed member, e.g., as $M_y{}^*$, $\theta_y{}^*$, $\theta_u{}^*$. Values calculated for the monolithic member according to the assumptions in Table 6.2 and Section 3.2 have no asterisk (M_y, θ_y, $\theta_u{}^{pl}$). Ratios of experimental values of M_y, θ_y, $EI_{eff} = M_yL_s/3\theta_y$ and θ_u for the tested jacketed members to values with the asterisk are shown in Fig. 6.2, separately for different types of jacket-to-old-member connection and for members which had been damaged by testing before been jacketed. Those specimens where the jacket longitudinal reinforcement did not continue past the member end and those with lap-spliced reinforcement in the original member are identified in Fig. 6.2. Other than that, they are lumped together with the specimens having continuous vertical bars in the original member. For tests not reaching ultimate conditions and for the two walls that failed in their unstrengthened storeys, an arrow pointing up signifies a test-to-prediction ratio greater than the value plotted.

The average value and ± standard-deviation estimates of the mean test-to-prediction ratios are shown in Fig. 6.2, separately for various groups of specimens with different types of jacket-to-old-member connection and with or no damage in the original column. The distance from the sample average to a certain reference value (e.g. 1.0), divided by the standard-deviation of the mean, is a criterion to decide whether the jacketed member's property in question may be taken equal to that calculated for the monolithic member according to Table 6.2 times that reference value. On this basis, the following simple rules are proposed for the yield moment, the chord rotation at apparent yielding and the ultimate chord rotation, $M_y{}^*$, $\theta_y{}^*$, or $\theta_u{}^*$, respectively, of the jacketed member, in terms of the values M_y, θ_y, $\theta_u{}^{pl}$ calculated for the monolithic member according to Table 6.2 (see also Bousias et al. 2007b, Biskinis and Fardis 2009):

$$M_y{}^* = M_{y,\mathrm{Sect.3.2.2.2}} \tag{6.12}$$

$$\theta_y{}^* = 1.05\theta_{y,\mathrm{Eq.(3.66)}} \tag{6.13}$$

(the main target being the effective stiffness at incipient yielding, computed from Eq. (3.68) as $EI^*{}_{eff} = M_y{}^*L_s/3\theta_y{}^*$), and

$$\theta_u{}^* = \theta_y{}^* + \theta_{u\,\mathrm{Eq.(3.78b)\,or\,(3.78c)}}^{pl} \tag{6.14a}$$

$$\theta_u{}^* = \theta_y{}^* + \theta_{u\,\mathrm{Eq.(3.72),(3.73)}}^{pl} \tag{6.14b}$$

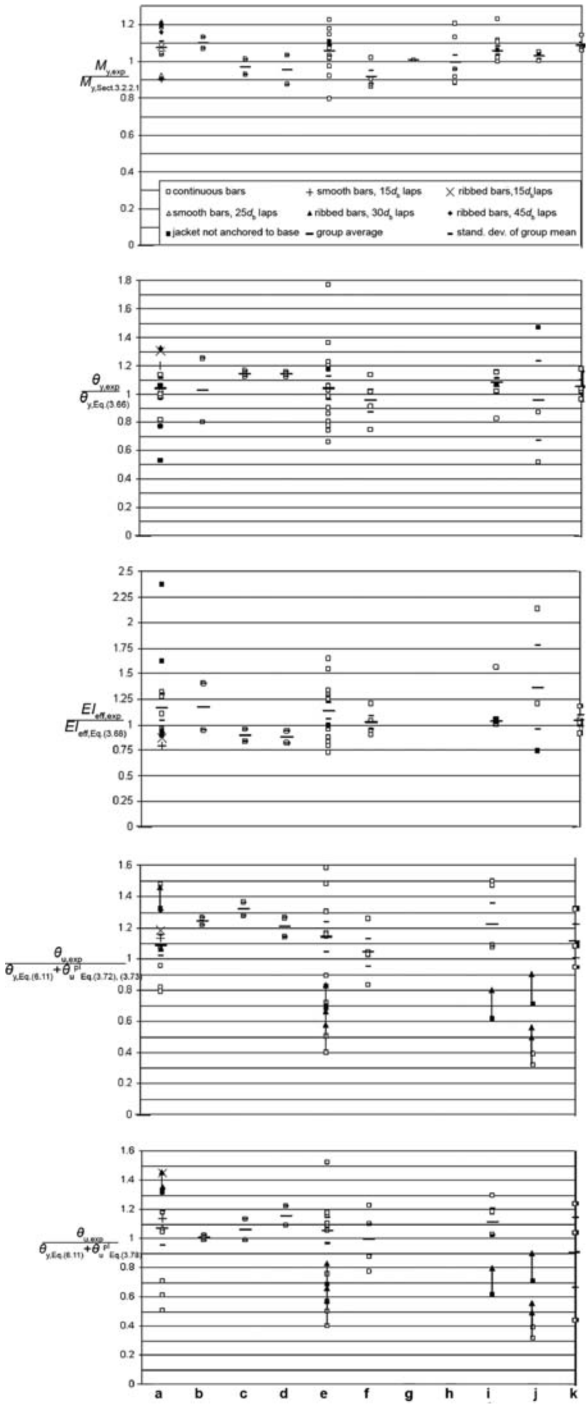


Fig. 6.2 Experimental value of the RC-jacketed member divided by the value calculated for the monolithic member according to Table 6.2 (*Key:*(**a**) no treatment of the interface; (**b**) no interface treatment, pre-damaged member; (**c**) welded U-bars; (**d**) dowels; (**e**) roughened interface; (**f**) roughened interface, member pre-damaged; (**g**) U-bars and roughened interface; (**h**) U-bars and roughened interface, member pre-damaged; (**i**) dowels and roughened interface; (**j**) dowels and roughened interface, member pre-damaged; (**k**) monolithic member)

Note that the value $\theta_y{}^*$ from Eq. (6.13) is used for the calculation of EI_{eff} and of θ_u at the denominator of the three lowermost plots in Fig. 6.2. Note also that the data cannot support a statistically meaningful effect of a pre-damage in the original column. So, Eqs. (6.12), (6.13) and (6.14) are proposed regardless of any such pre-damage. If the jacket longitudinal bars stop at the end section, Eqs. (6.12), (6.13) and (6.14) are used with assumptions B1 and B2 in Table 6.2.

Neglecting the effect of measures taken to enhance shear transfer at the interface of the old and the new concrete and grouping together the data in Fig. 6.2, the ratio of the experimental M_y to the prediction of Eq. (6.12) has a median of 1.035 and a coefficient-of-variation of 10.7%, compared to the values of 1.025 or 1.015 for the median and of 16.3% or 14.8% quoted in Section 3.2.2.2 under *Comparison with Experimental Results and Empirical Expressions for the Curvature* for M_y of monolithic beams/columns or rectangular walls, respectively. The median and the coefficient-of-variation of the ratio of experimental θ_y to the prediction of Eq. (6.13) are 0.99 and 23.5%, respectively, v the values of 1.025 or 0.995 and 32.1% or 33.7% given in Section 3.2.3.2 for the predictions of Eqs. (3.66) for θ_y of monolithic beams/columns or rectangular walls, respectively. Regarding the ratio of experimental EI_{eff} to the value predicted as $EI^*{}_{\text{eff}} = M_y{}^*L_s/3\theta_y{}^*$ (for $M_y{}^*$, $\theta_y{}^*$ from Eqs. (6.12) and (6.13)), the median and the coefficient-of-variation are 1.005 and 30.5%, to be contrasted to the values of 1.01 or 0.99 and of 32.3% or 47.1% quoted in Section 3.2.3.3 for the application of Eq. (3.68) to the database of monolithic beams/columns or rectangular walls, respectively. The median and the coefficient-of-variation of the experimental θ_u to the outcome of Eq. (6.14a) are 1.145 and 19%, v 1.0 and 42.4% for the predictions of Eqs. (3.78) for monolithic members. They become 1.08 and 24% for Eq. (6.14b), v 1.0 and 51.7% for Eqs. (3.72) and (3.73) and monolithic members. These comparisons show that average agreement with the experimental data is as good as that of the original expressions to the data they have been fitted, except for the ultimate chord rotation, where Eqs. (6.14) (mainly (6.14a)) are on the safe side. The small magnitude of the scatter for the jacketed members, although primarily due to the small sample size, is also re-assuring.

The data in Fig. 6.2 suggest that bonding measures at the interface of the jacket and the old member have a statistically significant effect only on the ultimate chord rotation. Equations (6.14) underestimate the measured θ_u-value of the few specimens with roughening of the interface and/or dowels. U-bars welded to the new and the old vertical bars have a beneficial effect on θ_u according to Eq. (6.14a) (maybe thanks to their anti-buckling action), but not according to (6.14b).[10] Even when no measure is taken to improve the interface between the old and the new concrete or connect the two materials there, Eqs. (6.14) undershoot the ultimate chord rotation of the jacketed member. It is therefore safe-sided for θ_u to use Eqs. (6.14) neglecting any favourable effect of a connection measures at the interface between the old and the new concrete. Interestingly, no systematic positive effect of

[10]Equations (6.14a) and (6.14b) point to opposite directions also about the effect of pre-damage on θ_u for (the just two) specimens without treatment at the interface.

roughening, dowels or welded U-bars on M_y and on the effective stiffness, EI_{eff}, has been found.

Note that slippage between the jacket and the old member is restrained in a real column under double curvature, but not in the cantilever-type specimens used in practically all tests from which the rules above were derived. Therefore, the effect of poor connection between the two layers would be even less in practice than in these tests.

According to Bousias et al. (2007b) there is no bias of Eqs. (6.12), (6.13) and (6.14) with respect to:

– the ratio of f_c or of the cross-sectional area of the jacket to those of the old member;
– the ratio of the mechanical reinforcement ratio in the jacket to that of the old member;
– the axial load, normalised either to the product of the full cross-sectional area of the jacketed section times the f_c-value of the jacket, or to the actual compressive strength of the jacketed section; and
– the ratio of the neutral axis depth at yielding to the thickness of the jacket.

This lack of bias supports assumptions A3 and A4 in Table 6.2, even when the compression zone extends beyond the jacket and into the old column.

None of the jacketed test specimens has shown any shear distress by the time it failed in flexure, which is consistent with the margin of at least 30% found between the shear resistance from Eqs. (3.114) and the maximum shear force applied.

6.8.2.4 Dimensioning and Verification of Jacketed Members According to Eurocode 8

On the basis of an earlier version of the database behind Fig. 6.2 and Eqs. (6.12), (6.13) and (6.14), Annex A in CEN (2005a) has adopted for the jacket the rules in Table 6.2, as well as Eqs. (6.12) and (6.14). However, for $\theta_y{}^*$ and for the effective stiffness at yielding resulting from it as $EI^*{}_{eff} = M_y{}^*L_s/3\theta_y{}^*$, Eq. (6.13) has been adopted only if the interface between the jacket and the old concrete is roughened. For no treatment of the interface, or for epoxy-grouted dowels alone, or for connection of the jacket bars to the old ones via welded steel inserts, Part 3 of Eurocode 8 adopted a softer response up to yielding:

$$\theta_y{}^* = 1.2\theta_{y,Eq.(3.66)} \tag{6.13a}$$

Faced with the scarcity of data on jacketed members failing in shear under cyclic loading, Annex A in CEN (2005a) has again adopted a cautious approach. It accepts for the jacketed member just 90% of the shear resistance computed for the "equivalent monolithic" member according to Eurocodes 2 or 8 (at points (5)–(8) in Section 6.5.6.3).

$$V_R^* = 0.9\, V_R \tag{6.15}$$

The values of M_y^*, θ_y^* and θ_u^* used in the verifications should be based on mean strengths of the old materials divided by the confidence factor and on nominal strengths of the new materials. These strength values enter in the calculation of the shear resistance of "primary elements" further divided by the pertinent partial factors. However, mean values of new or old materials, without a confidence factor, are used to compute M_y^*, θ_y^* entering in the calculation of $EI^*_{eff} = M_y^* L_s / 3\theta_y^*$, for the analysis.

6.8.3 *Jackets of Externally Bonded Fibre Reinforced Polymers (FRP)*

6.8.3.1 Scope of Seismic Retrofitting with FRPs

Externally bonded Fibre Reinforced Polymers (FRPs) are used in seismic retrofitting in order to enhance or improve (*fib* 2003, 2006):

a) The deformation capacity of flexural plastic hinges: A FRP jacket, with its fibres mainly along the perimeter of the section, is applied over the full length of the plastic hinge, to confine the concrete and prevent or delay bar buckling.
b) Deficient lap splices: A FRP jacket as in (a) above is applied over at least the full lap length, and
c) Shear resistance: A FRP overlay is applied, with the fibres mainly in the direction in which enhancement of shear strength is pursued.

Unlike concrete jacketing, which reduces through the added stiffness the seismic displacement and deformation demands, externally bonded FRPs only enhance the force and deformation capacities of the retrofitted member, serving therefore only retrofitting Strategy no. 2 in Section 6.7.1.

FRPs do not lend themselves for the enhancement of the moment resistance of members against seismic actions. The reason is that externally bonded FRPs with fibres in the longitudinal direction of a beam, column or wall cannot easily be continued into the joint beyond the member end section where the seismic bending moment is maximum. The moment resistance and the stiffness of a member can easily be enhanced instead through a concrete jacket (see Section 6.8.2), that can readily extend into a joint beyond the end of the member, providing continuity of the retrofitting between the member and the joint and – at the same time – strengthening of the joint itself. So, FRPs lend themselves only for selective modification of concrete members, notably of columns or walls, to improve their performance attributes listed above as (a–c).

Despite their high cost-to-weight ratio, externally bonded FRPs are becoming the material of choice in seismic retrofitting applications, owing to their:

- high strength-to-weight ratio,
- immunity to corrosion,
- easy handling and application (reducing labour costs and minimising disruption of use during installation) and
- very small thickness (minimising the loss in premium floor plan area, when the FRP is externally applied to vertical members).

For cultural heritage or historic buildings, whose architecture should not be altered by the intervention, externally bonded FRPs hold an advantage over any other technique: they can be made to have almost no impact on the external dimensions and appearance of structural elements. Moreover, their application is fully reversible.

That said, we should keep in mind certain drawbacks of the FRPs, such as their sensitivity to temperature and fire.

6.8.3.2 FRP Materials for Seismic Retrofitting

FRPs are relative new materials in seismic retrofitting and engineers are still not very familiar with them. For this reason, they are described in the present section at certain length.

The fibres of FRPs used for strengthening civil engineering structures are made of carbon, glass, or aramid, giving a FRP commonly termed CFRP, GFRP or AFRP, respectively.

Carbon fibres show the best stability under high temperatures and the best resistance to deterioration in acidic, alkalic or organic environments, including marine ones. They have high stiffness (Elastic Modulus) and tensile strength, but higher Modulus normally goes together with lower tensile strength and ultimate tensile strain. However, they are much more expensive than glass or aramid fibres (10–30 times more costly than E-glass fibres (*fib* 2007)).

Glass fibres are classified as (*fib* 2007):

- E-glass, which is popular as less costly;
- AR-glass, which is alkali resistant, but not available yet in sizes compatible with common thermosetting resins; or
- S-glass, which is stronger and stiffer than the other types.

Glass fibres, especially E-glass ones, are less expensive than carbon or aramid fibres. E-glass and S-glass fibres may lose up to 30–100% of their tensile strength in alkaline environments, especially at high temperatures.

Aramid is the term used for polymeric fibres appropriately processed to achieve high tensile strength-to-density ratio. Like E-glass and S-glass, these fibres may also lose up to 25–50% of their tensile strength in alkaline environments, but have good toughness and fatigue characteristics and are more tolerant to damage.

Fibres are linear-elastic up to failure, both in tension and in compression, with strength and Modulus in compression a little less than in tension. Aramid fibres are

Table 6.3 Typical tensile properties of fibres (*fib* 2003, 2006, 2007)

Fibre material		Elastic modulus (GPa)	Strength (GPa)	Ultimate strain (%)
Carbon	High strength	215–240	3.5–4.8	1.1–2.0
	Ultra high strength	215–240	3.5–6.0	1.5–2.3
	High modulus	350–500	2.5–3.1	0.5–0.9
	Ultra high modulus	500–700	2.1–2.4	0.2–0.4
Glass	E	72.5	1.9–3.4	2.5–4.5
	S	85–90	3.5–4.8	3.3–5.5
	AR	70–76	1.8–3.5	2.0–3.0
Aramid	Low modulus	60–80	2.8–4.1	4.3–5.0
	High modulus	115–175	3.4-4.2	1.5–3.5

the exception, being non-linear and ductile in compression with 80% less strength than in tension. Values of important mechanical properties of fibre materials are listed in Table 6.3. They apply for static loading in tension and for fibres not exposed for long to adverse environment. The manufacturer normally gives more representative values than those of Table 6.3, as well as information on their reduction due to adverse environmental exposures and long-term loading (which is not relevant for seismic retrofitting).

The fibres come in the form of flexible sheets (called also fabrics, or textiles), consisting of fibres mainly in one direction, or in two orthogonal ones, or in more directions, including oblique ones. In seismic retrofitting, particularly in buildings, the sheet is impregnated in-situ in a matrix, typically of a thermosetting polymer, that serves also as adhesive to the concrete substrate. The matrix binds the fibres together, transferring loads to them, and protects them in-situ from abrasion and adverse environmental effects. Having much higher – by one to two orders of magnitude – strength and Elastic Modulus than the matrix material, the fibres are the main stress-bearing component. The matrix governs only the shear properties of FRPs having fibres mainly in one or in two orthogonal directions, as well as the transverse modulus and strength of FRPs with fibres primarily in one direction.

The tensile strength and stiffness of the FRP (per linear meter) are typically derived from the corresponding values of the bare fibres (see Table 6.3), by multiplying them by the nominal thickness of the fibre sheet or fabric quoted by the manufacturer (typically a small fraction of a mm) and the number of plies (or layers) of sheets applied. Normally, it is not considered worth accounting for the efficiency of the fibre-matrix system and the sheet or fabric architecture, or for the small increase due to the contribution of the matrix. The FRP tensile strength and stiffness may also be obtained by multiplying the strength and Modulus of the FRP – given by the supplier of the materials for the specific combination of fibre sheet or fabric and matrix material used – by the nominal (and not the actual) thickness of the finished FRP specified by the manufacturer.

FRPs subjected to sustained stresses do creep and may ultimately fail by creep-rupture under stresses well below their short term strength. The time-to-rupture decreases when the temperature or the ratio of sustained stress to short term strength increase, or when the FRP is subjected to alkaline environment, UV light, or humidity, constant or not. The strength under sustained stresses is, however, of little relevance if the FRP is applied for seismic retrofitting alone. Its low-cycle fatigue behaviour is more important in that case. The structural response to a seismic action of the type considered in retrofitting normally includes an order of ten large cycles of almost constant amplitude. In such a scenario CFRP, AFRP and GFRP may lose about 5–8%, 5–6%, or 10%, respectively, of their short term static strength. The loss about doubles if 100 constant amplitude load cycles are applied (*fib* 2007). The reduction is small enough to be considered as covered by the safety factors applied on the FRP properties in the design of the retrofitting. Anyway, if expressions used for the cyclic capacity of FRP-retrofitted members have been calibrated or derived on the basis of cyclic tests, they are deemed to account for any low-cycle fatigue of the FRP.

The coefficient of thermal expansion of FRPs is dominated by that of the fibres, except in the transverse direction of unidirectional FRPs, where it is governed by the matrix. GFRPs have about the same coefficient of thermal expansion as concrete, except in the transverse direction of unidirectional GFRPs, where this coefficient is about double. CFRP and AFRP have very low but negative coefficients of thermal expansion (they shrink when temperature increases). If they are unidirectional, their coefficient of thermal expansion in the transverse direction is an order of magnitude higher than in concrete. Owing to the way CFRP and AFRP are externally applied to concrete members for seismic retrofitting, such disparities do not cause serious problems during service life.

Thermosetting polymeric matrix materials are epoxy, polyester or vinyl ester resins. Epoxy resins offer good wetting and bonding to the fibres and to various substrates and have rather long open time. They are more costly than polyesters or vinyl esters, but have better mechanical properties, low creep, little shrinkage during curing and good resistance to water, temperature and chemicals. Regarding alkali resistance and water absorption, they rate in-between vinyl ester (which is best) and polyesters (which are worst). Polyester resins have low viscosity. They cure fast but shrink a lot while curing. Vinyl esters have good wetting and bonding to glass fibres, high strength, excellent alkali resistance, low water absorption and – very important – moderate cost. So, they are often the matrix of choice for GFRP.

Polymers, especially polyesters, absorb water from a humid environment, suffering some deterioration of mechanical properties, including, very importantly, debonding between the matrix and the fibres. Temperatures over 60°C exacerbate these adverse effects of moisture (*fib* 2007). Being polymeric, aramid fibres also absorb water, suffering a reversible reduction of tensile strength and Modulus and an irreversible decrease of their fatigue strength. Such reductions can reach 15–25%.

UV radiation inflicts considerable damage to the mechanical properties of polymeric matrices and Aramid fibres. The loss in tensile strength after long exposure to UV light is negligible for CFRP and does not exceed 8% in GFRP. AFRP exhibits larger reductions without a clear limit (*fib* 2007). To avoid losing eventually the matrix, externally bonded FRPs should be shielded from direct sunlight, either through cladding or rendering with plaster, or by means of proprietary protection systems.

CFRPs are almost immune to chloride attack, even under high moisture conditions, but GFRP and AFRP are more vulnerable to chloride-moisture combinations (*fib* 2007). The most important chemical threat comes from alkalis and the alkaline environment of concrete. This threat is negligible for CFRP and GFRP with AR-glass fibres, moderate for AFRP and very serious for GFRP with E-glass and S-glass fibres, especially under high temperatures (*fib* 2007). Note, though, that the surface layer of existing concrete elements would most likely be carbonated (hence, non-alkaline) by the time the FRP is externally bonded to it for seismic retrofitting. This greatly reduces the risk of alkali attack from within.

At their "glass transition temperature" polymers soften from their glassy state to a rubbery one. That temperature ranges between 95 and 175°C for epoxy resins, from 70 to 100°C for polyesters and from 70 to 163°C for vinyl esters (*fib* 2007). In principle, it is prudent to select a matrix material that has glass transition temperature at least 30°C above the maximum expected service temperature (Karbhari et al. 2003). It should be kept in mind, though, that for FRP applied for seismic retrofitting it is the quasi-permanent (average or arbitrary-point-in-time) value of the temperature of the immediate environment of the retrofitted element that should be taken as concurrent with the level of seismic action considered in retrofitting.

The Elastic Modulus of CFRP, AFRP or GFRP decreases when the temperature increases above the glass transition temperature of the matrix. The reduction is reversible, provided that the temperature level causing chemical degradation of the polymer is not reached. The drop in Modulus of CFRP, AFRP or GFRP when the temperature increases from –20 to +60°C is about 10, 25 or 35%, respectively. Being organic, Aramid fibres suffer not only a reduction in Modulus when the temperature increases, but also a drop in tensile strength. However, at 180°C they still retain about 80% of their 20°C strength (*fib* 2007). Carbon fibres can resist temperatures of 800–1000°C with little loss in mechanical properties. Glass fibres do the same up to 300–500°C. However, the polymeric matrix will burn at about 150–200°C, governing the fire resistance of the FRP. This is a serious drawback only if the surface-bonded FRP is applied to strengthen the member just against gravity loads. Then its loss during fire may lead to direct structural failure and collapse. FRPs applied for seismic retrofitting alone can always be replaced if damaged by a fire. The earthquake resistance they originally offered can be fully re-instated for future use.

The specification of the polymeric matrix should include the range of temperatures appropriate for mixing, application and curing. The shelf-life of thermosetting polymers (i.e. the time for which the unmixed resin and the hardener can be stored with no degradation) is limited and depends on the storage temperature. It should be checked that shelf-life has not expired by the time of mixing for in-situ application. The effect of temperature on the resin pot-life (i.e., the time after mixing the resin and the hardener during which the viscosity is low enough for application) and on its open time (in this case, the maximum time available between the resin application to a fibre sheet and the attachment of the sheet to the substrate) should be taken into account.

With the continuous decrease in fibre prices, the polymer is becoming an important factor in the FRP cost. To reduce this cost and bypass the problem of poor fire resistance of polymers, polymer-modified cement-based mortars have been used as binders of the fibres and as adherents to the substrate (Triantafillou et al. 2006, Triantafillou and Papanicolaou 2006, Bousias et al. 2007c). Unlike resins, mortars cannot wet individual fibres. So, continuous fibre sheets need to be replaced by fabric meshes of long woven, knitted or even unwoven fibre rovings in two or more directions (Textile Reinforced Mortars, TRM). The quantity and spacing of rovings per direction can be tailored to the target mechanical properties of the textile and the ability of the mortar to penetrate the textile mesh.

6.8.3.3 Field Application of FRPs

In seismic retrofitting, particularly of buildings, FRPs are typically applied in situ with the "wet lay-up" (or "hand lay-up") method. In this flexible approach, a first coating of adhesive is spread over the appropriately prepared concrete surface. The dry fibre fabric, pre-cut at the desired size, is impregnated in place by pressing into the adhesive usually with a roller (Fig. 6.3(a,b)). The adhesive and air are squeezed out through the fibre sheet, taking care to avoid wrinkles in the FRP and bubbles of entrapped air. The next ply and any subsequent ones are applied in the same way, following fresh application of a layer of adhesive onto the underlying FRP layer. Alternatively, the fibre fabric is impregnated with the adhesive on the floor and then pressed in place against the previously applied layer (Fig. 6.3(c)). For continuous wrapping of the FRP around the concrete member, the adhesive is rolled onto the FRP layer applied last just ahead of the upcoming fibre sheet. A lapping of about 150 mm between the start of a continuous FRP wrapping along the member perimeter and its end is sufficient.

The number of FRP plies should be limited, e.g., to a maximum of five. As mentioned at the end of the part of Section 3.2.3.10 on *Members with Continu-*

(a) (b) (c)

Fig. 6.3 Hand lay-up of FRPs in situ: (**a**), (**b**) dry fabric impregnated in place (courtesy A. Ilki); (**c**) impregnation of the fabric right before placing (See also Colour Plate 15 on page 729) *ous Bars* and demonstrated by the last term in Eqs. (3.89), (3.90) and (3.91), the tensile strength of the FRP which is effective in concrete confinement is less than proportional to the total thickness of the FRP provided.

A smooth layer of non-shrink mortar (possibly polymer-modified) is often uniformly applied on the concrete surface. Its purpose is not to act as a bonding agent (polymers bond well to concrete), but to cover any roughness or asperities and provide a smooth, even final surface for the application of the lowermost FRP layer. If earthquake damage or other reasons of deterioration (e.g., reinforcement corrosion) has caused spalling or disintegration of the concrete in the element being retrofitted, the same non-shrink mortar is used to replace the spalled or loose concrete. The surface of the substrate on which the FRP is applied should be absolutely clean and dry. The FRP should not be applied while the substrate has water content more than 4% by weight or its temperature is below 5°C. Below 10°C, hardening of typical polymers stalls.

Before application of the FRP, sharp edges should be chipped off and rounded by applying a layer of non-shrink mortar to a corner radius of 20–30 mm. This is to avoid stress concentrations that may lead to premature FRP rupture and to extend the confining action of the FRP at the corner to a larger concrete volume (see. Eq. (3.28) and Fig. 3.17 in Section 3.1.2.4).

FRP wrapping should start 10–15 mm from the end section of the member (at the connection to a joint or a foundation element). The gap is to prevent bending of the member from causing the FRP to bear against the surface of the element or foundation into which the FRP-wrapped member frames. Such bearing is to be avoided, because it increases the force in the compression zone and the flexural capacity of the member, which in turn increases the shear force demand beyond the capacity design shear (see Sections 6.5.5.1 and 6.5.5.2). An unwrapped length of even 30 mm at the end of the member will not suffer from the lack of direct confinement by FRP, thanks to confinement afforded to it by the FRP-wrapped length of the member on one hand and by the volume of concrete of the transverse member or foundation element into which the member in question frames on the other.

Seepage of water into the FRP-concrete interface at the connection of FRP-wrapped members to the foundation should be prevented through appropriate sealants. If the lateral surface of the member needs to be continuously covered by FRP over its full length (e.g., for shear strengthening), evapouration of moisture should be allowed, e.g. through gaps of 30–50 mm between adjacently wrapped (typically 600 mm-wide) fibre sheets.

6.8.3.4 Material Partial Factor on the Tensile Strength of FRPs

Committee 440 of the American Concrete Institute (ACI 2003) has the equivalent of a material partial factor covering adverse environmental effects on CFRPs, AFRPs and GFRPs, with values of about 1.05, 1.15 and 1.35, respectively. This factor is in addition to (i.e., should multiply) the material partial factor that covers dispersion of the mechanical properties and creep-rupture effects, the value of which, according to ACI (2003), is between 1.5 and 1.8. The specifications of the Japanese Society of Civil Engineers (JSCE 1997) cover both adverse environmental effects and dispersion of mechanical properties through an overall material partial factor with values of 1.15 for CFRP and AFRP, or 1.3 for GFRP. Annex A to Part 3 of Eurocode 8 provides just a material partial factor against dispersion of mechanical properties, equal to 1.5, as if environmental deterioration would be prevented through appropriate measures.

6.8.3.5 Flexural Strength, Stiffness and Deformation Capacity of Members with FRP-Wrapping

For members wrapped with FRP along the end region where yielding and flexural plastic hinging takes place (to improve the deformation capacity of the plastic hinges and any deficient lap splices, see points (a), (b) in Section 6.8.3.1), Section 3.2.3.10 has presented models for the yield moment, the secant stiffness to the yield-point and the cyclic ultimate chord rotation, as affected by the FRP wrapping. That information is summarised here for convenience:

1. The yield moment of members with continuous bars is somewhat underestimated, if the effect of any FRP wrapping is neglected. The prediction improves if the confined concrete strength, $f_c{}^*$, from Eq. (3.27a) in Section 3.1.2.4, is used in this calculation. The same conclusions apply for the moment resistance.
2. In members with continuous bars the secant stiffness to the yield-point may be estimated from Eqs. (3.66), (3.67) and (3.68), neglecting the effect of confinement by the FRP.
3. The ultimate chord rotation of members with continuous bars may be estimated from Eqs. (3.78) in Section 3.2.3.5, provided that either one of the following is added to the exponent of the 2nd term from the end:

    (i) term $a_f \rho_f f_{f,e}$, with $\rho_f = 2t_f/b_w$ being the FRP geometric ratio parallel to the loading direction, a_f the effectiveness factor for confinement by the FRP given by Eq. (3.28) as a_n and $f_{f,e}$ the effective stress of the FRP from Eq. (3.89); or
    (ii) the term given by Eq. (3.90); or
    (iii) the term given by Eq. (3.91).

    Alternatives (ii) and (iii) are more accurate than (i), which is more safe-sided and has been adopted in Annex A of CEN (2005a).

    The so-predicted ultimate chord rotation is on the safe-side by about 5% on average, for members that are intact when retrofitted with FRP. It may be somewhat unconservative for members that had suffered serious damage and were repaired before been wrapped with FRP.
4. The ultimate chord rotation of members with continuous bars may be estimated equally well as in point 3 above, if Eq. (3.72) in Section 3.2.3.4 is used, with L_{pl} from Eq. (3.73) and ultimate curvature, φ_u, from:

– the plane-sections analysis in Section 3.2.2.4 modified to use a parabolic-trapezoidal σ-ε law for FRP-confined concrete, as in Lam and Teng (2003a,b) instead of a parabolic-rectangular one,
– the (Lam and Teng 2003a,b) model for the confined strength, Eq. (3.27a), and
– Eqs. (3.29) and (3.30) for the ultimate strain of FRP-confined concrete under cyclic loading.

5. If ribbed (deformed) longitudinal bars are lap-spliced over a length l_o starting at the end section and the member is wrapped with FRP all along the lapping, then:

   (i) the yield moment, the chord rotation at yielding and the member effective stiffness derived from them may be calculated with:

      a. both bars in any pair of lapped compression bars counting as compression reinforcement and
      b. with a maximum possible stress of the lapped tensile bars:

         I. obtained from Eq. (3.31) in Section 3.1.3.2, using there Eq. (3.32a), or
         II. equal to their yield stress times $l_o/l_{oy,min} \leq 1$, with $l_{oy,min}$ from Eq. (3.85a).

   (ii) the ultimate chord rotation, θ_u, is obtained from Eqs. (3.78b) or (3.78c), with θ_y corrected for the lap-splicing and the FRP-wrapping according to 5(i) above and the last term at the right-hand-side of Eqs. (3.78b) or (3.78c), $\theta_u{}^{pl}$, computed taking into account point 5(i)a above and then multiplied by $l_o/l_{ou,min} \leq 1$, with $l_{ou,min}$ from Eq. (3.92).

6. In members with serious damage (beyond yielding, to nearly ultimate deformation), FRP-wrapping adds very little to the effect of repair carried out according to Section 6.8.1 on yield moment (which is re-instated, anyway) and secant stiffness to the yield-point (which is reduced by previous damage to about three-quarters of the value estimated according to point 2 above). However, unlike mere repair, FRP-wrapping eliminates almost fully the adverse effect of such damage on the member's ultimate chord rotation, which seems to be nearly the same regardless of any damage suffered by the member before been wrapped.

According to Annex A of CEN (2005a) the effect of FRP wrapping on the yield moment, moment resistance and secant stiffness to the yield-point of members with continuous bars may be neglected (see points 1 and 2 above), while that on ultimate chord rotation may be found by applying approach 3(i) above to Eq. (3.78a). If ribbed longitudinal bars are lap-spliced starting at the end section and the member is wrapped with FRP all along this lapping, point 6(i)a and approach II in 6(i)b above apply for the yield moment, the chord rotation at yielding and the member effective stiffness. Point 6(ii) applies for the ultimate chord rotation.

Annex A of CEN (2005a) gives also alternative options for the dimensioning of the FRP for a target value of the curvature ductility factor of the retrofitted member and for clamping of short lap splices. Those alternative approaches, described in detail in *fib* (2003, 2006) have not been validated/calibrated on the basis of test results, at least to the same extent as the Eurocode 8 options highlighted in the present section and in Section 3.2.3.10.

The values of M_y, θ_y and θ_u used in verifications in the framework of retrofitting according to EN-Eurocode 8 should be based on mean strengths of the old materials divided by the confidence factor and on nominal strengths of the FRP. Mean values of all materials, without confidence factors, are used in the values of M_y, θ_y entering in the calculation of the effective stiffness for the analysis.

6.8.3.6 Cyclic Shear Resistance of FRP-Wrapped Members

Seismic strengthening of beams is easier in shear than in flexure, as it does not require intervention into the joint region or in the top slab. Shear strengthening can be implemented, instead, by side-bonding FRP straps or by U-jacketing with FRP over the three exposed sides of the beam, provided that the individual straps or the open ends of the U-jacket are sufficiently anchored at the beam sides or at their connection with the slab. Normally shear strengthening may be limited to the end regions of the beam where the shear due to the concurrent gravity loads, $g+\psi q$, (cf. 2nd term, $V_{g+\psi q,o}(x)$, in Eq. (6.3)) has large values. Moreover, if the shear force due to gravity is large compared to the seismic shear (1st term in Eq. (6.3)), nearly unidirectional FRP sheets or straps may be used at an angle $\alpha = 45^\circ$ to the beam axis. Such a shear strengthening is dimensioned as for gravity loads (*fib* 2003, 2006, Triantafillou 1998, Monti and Liotta 2005). As a matter of fact, Annex A to CEN (2005a) has adopted the general approach for shear strengthening for gravity loads (Triantafillou 1998, Monti and Liotta 2005) for the dimensioning of FRP straps or sheets that are wrapped around or U-jacket the beam, or are side-bonded to it at any angle to the member axis.

In gravity or seismic load strengthening alike, the contribution of FRP straps or sheets to the member resistance against diagonal tension is taken similar to that of shear reinforcement, except that the FRP cannot be taken to work with its full tensile strength. First, by not being ductile like steel but linear up to failure, the FRP cannot develop its full strength under the variable tensile strains along the diagonal crack. Second, it may exhibit its maximum contribution to shear resistance before the strut inclination rotates sufficiently to maximise that of transverse reinforcement. Third, the variable strut inclination model described in Section 3.2.4.2 under *The Variable Strut Inclination Truss of the CEB/FIP Model Code 90 and Eurocode 2* does not fully apply in this case, as the FRP bridging a diagonal crack may also exhibit a “dowel” type of resistance, instead of mere uniaxial tension. So, when a component due to the FRP is introduced in the variable strut inclination model according to the following generalisation of Eq. (3.94) (*fib* 2003, Triantafillou 1998, Triantafillou and Antonopoulos 2000):

$$V_{R,f} = \rho_f b_w z(\varepsilon_{f,e} E_f)(\cot\delta + \cot\alpha)\sin\alpha \tag{6.16}$$

it should employ an “effective strain”, $\varepsilon_{f,e}$, less than the fracture strain of the FRP. In Eq. (6.16) δ and α denote the inclination of the strut and of the main direction of the FRP, respectively, with respect to the member axis, ρ_f is the geometric ratio of the FRP in its main direction and E_f its Modulus. The value of the "effective strain" $\varepsilon_{f,e}$ depends on whether debonding (especially of side-bonded or U-wrapped FRP) may pre-empt fracture of the FRP in tension. In addition, for a given mode of FRP bonding (U-jacketing, side-bonding or full wrapping), $\varepsilon_{f,e}$ seems to increase with the ratio of the tensile strength of concrete, f_{ct}, that governs debonding, to the extensional stiffness of the FRP, $\rho_f E_f$, which controls the force demands placed on the FRP anchorage (*fib* 2003, Triantafillou 1998, Triantafillou and Antonopoulos 2000). In Triantafillou and Antonopoulos (2000) empirical expressions have been proposed for $\varepsilon_{f,e}$ as a function of $f_{ct}/\rho_f E_f$, of the mode of FRP bonding and of the fracture strain of the FRP. Such an approach has been adopted in ACI (2003) and JSCE (2001) as well.

It is prudent to cap $\varepsilon_{f,e}$, to allow the rest of the shear resisting components to develop their contribution before that of the (brittle) FRP is exhausted. To this end, the following upper limits have been proposed for the design value of $\varepsilon_{f,e}$:

– $\varepsilon_{f,e} \leq 0.007$ in JBDPA (1999);
– $\varepsilon_{f,e} \leq 0.006$ in *fib* (2003).

This also caps the inelastic tensile strain that the transverse reinforcement is allowed to develop and sets a lower limit on the strut inclination δ at exhaustion of the diagonal compression strength of the web. Note that the resistance for diagonal compression (web crushing) does not appreciably increase thanks to enhancement of the diagonal compression strength of concrete by FRP wrapping over the full lateral surface of the member (let alone of just the end regions or of part of the perimeter). Therefore, regardless of any FRP strengthening, the corresponding shear resistance, V_{Rmax}, still has the value given in Section 3.2.4.2 under *The Variable Strut Inclination Truss of the CEB/FIP Model Code 90 and Eurocode 2* for slender members under monotonic or cyclic loading. Then, by setting the value of V_{Rmax} from Eq. (3.97) equal to the sum of $V_{R,s}$ from Eq. (3.94) and $V_{R,f}$ from Eq. (6.16), we obtain the lower limit of δ in slender members with the main direction of the FRP at right angles to their axis, α=90° (cf. Eq. (3.98)):

$$\tan\delta \geq \sqrt{\frac{\dfrac{\rho_w f_w + \rho_f E_f \varepsilon_f}{n f_c}}{1 - \dfrac{\rho_w f_w + \rho_f E_f \varepsilon_f}{n f_c}}} \tag{6.17}$$

At this value of δ the shear resistance is (cf. Eq. (3.99)):

$$V_R = \sqrt{\left(\rho_w f_w + \rho_f E_f \varepsilon_f\right)\left(n f_c - \rho_w f_w - \rho_f E_f \varepsilon_f\right)} b_w z f_c \tag{6.18}$$

Equation (6.18) shows that the gain in shear resistance is less than proportional to the amount of FRP added for shear strengthening, especially as we approach the point of diminishing returns:

$$\rho_f E_f \varepsilon_f = 0.5 n f_c - \rho_w f_w \tag{6.19}$$

where the upper limit of shear resistance is reached:

$$V_R = 0.5 n b_w z f_c \tag{6.20}$$

A similar approach may be followed if the main direction of the FRP is not at right angles to the member axis, $\alpha < 90°$. The value of V_{Rmax} from Eq. (3.97) is set equal to the sum of $V_{\mathrm{R,s}}$ from Eq. (3.94) and $V_{\mathrm{R,f}}$ from Eq. (6.16), to solve (numerically) for the limit value of δ. Then, the shear resistance of the FRP-retrofitted member is obtained from Eq. (3.97).

The discussion above has mainly beams in mind. As pointed out in Section 6.7.3, the emphasis of Section 6.8 on modification of existing components for seismic retrofitting, is on vertical elements, mainly columns. Unlike beams, columns and walls are subjected to a constant shear force within each storey. So, if indeed shear strengthening is needed, it should be uniform throughout the height of the vertical element in a storey. Moreover, as the shear demand alternates between opposite values, the main direction of the FRP should be horizontal. The end result is a vertical element fully wrapped in (nearly) unidirectional FRP sheets essentially all along its length (see Fig. 6.12(b) for an example). If wrapping all around the element is neither essential (as, e.g., in rectangular walls) nor feasible (e.g., when not all sides are accessible), proper attention should be paid to the anchorage of the FRP near or around the edges of the section (see Section 6.10.2 for such examples). At any rate, the dimensioning of this FRP in shear may be carried out according to the general approach above, or to the more elaborate one in Triantafillou (1998) and Monti and Liotta (2005) adopted in Annex A of CEN (2005a). The focus of the rest of the present section is on the contribution of any FRP wrapping of the plastic hinge region to its resistance against diagonal tension failure ("ductile shear", see Section 3.2.4.3).

FRP-wrapping of a member's end region increases appreciably its cyclic deformation capacity, but does not delay yielding. So, the ductility ratio demand, $\mu_\theta{}^{\mathrm{pl}}$, may increase during inelastic cycling sufficiently for the FRP-wrapped plastic hinge to become critical in shear. Note that FRP-wrapping does not appreciably increase the cyclic shear resistance for diagonal compression failure after flexural yielding, given by Eq. (3.115) for squat walls (with $L_s/h < 2.5$) or Eq. (3.127) for squat columns (with $L_s/h \leq 2.0$). The very few available cyclic tests on FRP-wrapped squat columns failing by diagonal compression after flexural yielding show that Eq. (3.127) is indeed safe-sided. The question is then by how much FRP-wrapping increases the cyclic shear resistance after flexural yielding, over the value given for diagonal tension by Eqs. (3.114).

There are very few (about 10) cyclic tests of concrete members with FRP-wrapped ends that led to diagonal tension failure after flexural yielding. Their results suggest that the resistance to diagonal tension may still be obtained from Eqs. (3.114), provided that a term is added for the FRP contribution. One option is to base this term on the effective, average strength of the FRP all around the column according to the (Lam and Teng 2003a,b): $f_{\mathrm{fu,L\&T}} = E_{\mathrm{f}}\varepsilon_{\mathrm{fu}}$, with $\varepsilon_{\mathrm{fu}}$ about equal to 60% of the failure strain of tensile coupons (see Sections 3.1.2.4 and *Members with Continuous Bars*):

$$V_{R,FRP} = \frac{h-x}{2L_s}\min\left(N; 0.55A_c f_c\right) + \left(1 - 0.05\min(5;\mu_\theta^{pl})\right)$$
$$\left[0.16\max(0.5;100\rho_{tot})\left(1-0.16\min\left(5;\frac{L_s}{h}\right)\right)\sqrt{f_c}A_c + V_w + \rho_f b_w z f_{fu,L\&T}\right] \quad (6.21)$$

This proposal gives a test-to-prediction ratio in the 10 columns tested to ductile shear failure with an average of 0.99 and a coefficient of variation of 14.1%.

A simpler option, proposed in Biskinis (2007) and adopted in Annex A of CEN (2005a), is to add to the right-hand-side of Eqs. (3.114) a term $V_{\mathrm{f}} = 0.5\rho_{\mathrm{f}}b_{\mathrm{w}}z(E_{\mathrm{f}}\varepsilon_{\mathrm{u,f}})$ that does not degrade with inelastic cyclic displacements. In this case $\varepsilon_{\mathrm{u,f}}$ is the failure strain of the FRP from tension coupons and the factor of 0.5 reflects the linear reduction of the FRP stress over the section depth, from the failure strain at the extreme tension fibre to zero at the neutral axis. This simpler alternative gives as good a prediction as Eq. (6.21). The average of the test-to-prediction ratio in the 10 columns is 1.01 and the coefficient of variation is 12.9%.

6.8.4 Steel Jacketing

6.8.4.1 Scope and Construction Aspects

Steel jackets are more expensive than concrete ones. However, their technology is simple, familiar to the construction industry and readily available almost everywhere. So, it is the technique of choice for non-engineered emergency strengthening even hours after a damaging earthquake, to prevent collapse of heavily damaged buildings or give back to use moderately damaged ones during the aftershock period. Detailed assessment and retrofit design may take place afterwards. The steel jackets may be removed when retrofitting is implemented, or incorporated in a concrete jacket (as in Fig. 6.4(a)). Despite this advantage and the long history of surface-bonded steel plates in strengthening of RC members, they are being replaced fast by surface-bonded FRPs, which, although more costly, are much lighter, easier to apply and mechanically more effective.

Thin-walled steel jackets are most efficient and easier to apply around circular columns. There they usually come in two semi-circular halves fitting closely around the column and field-welded along two vertical seems. The gap between the jacket and the column is grouted with non-shrink mortar. Such jackets are considered so efficient, that they have even been proposed for retrofitting square or rectangular columns, using a large quantity of concrete (rather than mortar grout) to fill the gap between the jacket and the column. However, circular or elliptical steel jackets around square or rectangular building columns are neither practical nor aesthetically

(a) (b)

Fig. 6.4 Steel jackets built-up in situ with corner angles and horizontal straps

appealing. For such columns steel jackets are built up of four corner angles, usually epoxy-bonded to the concrete or just in gapless contact to it along the full length. Continuous thin steel plates or thicker horizontal steel straps or batten plates are welded to the corner angles (Fig. 6.4). Within the limitations imposed by the heavy weight of steel segments, continuous thin plates may be shop-welded to the corner angles into larger L-shaped pieces, fitting half the perimeter of the column and fillet-welded in the field. The 10–20 mm gap between the plate and the surface of the column is grouted with non-shrink mortar. Before been welded, straps or batten plates may be pre-heated in the field to 200–400°C, to exert some "active" confinement on the column after they cool down. Most of the benefit is gradually lost owing to concrete creep. So, it is neglected in design. Depending on the intended role of the jacket, the gap between the column and the straps or batten plates may be grouted with non-shrink mortar or left unfilled.

Steel jackets around columns enhance ductility through confinement (see Section 6.8.4.2), increase shear strength (Section 6.8.4.3) and improve deficient lap splices (Section 6.8.4.4). All these effects have to do with the action of the jacket in the transverse direction of the member. But as steel is isotropic, if the jacket consists of a continuous thin plate, it presents significant stiffness and strength in the longitudinal direction as well, which unavoidably affects flexural stiffness and moment resistance. The extent of this influence depends on how the steel jacket is connected to the concrete member and to the ones framing into it. However, steel jackets are normally not intended for flexural strength enhancement. Their continuation beyond the member end, although not so difficult as for FRP jackets, is not easy. Moreover, even though they can transfer forces beyond the member end by bolting or welding to other steel elements through the slab and by bearing against the concrete surface, they are not so effective in resisting cyclic flexure in composite action with the concrete member inside, as their thin walls may buckle. As a result, they are applied on the concrete surface so that they develop stresses mainly in the circumferential direction, with their effect on member flexural strength and stiffness minimised.

6.8.4.2 Confinement by Steel Jackets

The confinement of a rectangular section by the steel jacket may be calculated from Eqs. (3.23) and (3.25), as if the jacket (continuous or in straps) were internal hoops and ties, using as geometric steel ratio ρ_x or ρ_y in each transverse direction the cross-sectional ratio of the jacket in a vertical section of the column. Unless tied back into the column, thin steel plates or straps welded to the corner angles do not confine the sides of a rectangular section, because, being flexible, they bulge outwards. So, the confinement effectiveness factor within the section, a_n, may be calculated from Eq. (3.28) in Section 3.1.2.4, with the corner radius R replaced by the width, b, of the steel angle (where there is full contact of the angle and the column concrete). If continuous steel plates are welded to the corner angles, the confinement effectiveness factor along the member, a_s, may be taken equal to 1.0. If steel straps or batten plates are welded instead, a_s may be calculated from Eq. (3.20c), using there as b_{xo}, b_{yo} the external dimension of the section and as s the clear spacing, s_{cl}, of straps or batten plates, reduced by twice the corner angle width, b, according to a postulated 45° dispersion of confining action from the strap into the corner angle: $s = s_{cl}$-$2b$ (Dritsos and Pilakoutas 1992).

Friction between the corner angles and the column, owing to the confining forces developed there when the concrete column is approaching ultimate conditions, enhances the composite action of the column with the steel jacket and mobilises the jacket in the longitudinal direction, even when it is not continued into the joint beyond the member end. The resulting increase in stiffness and strength of the jacketed column is uncertain, but forms a second line of defense against loss of axial-load-capacity of the column in the post-ultimate range. Note that an enhancement of the column moment resistance may adversely affect the shear force demand on the column and the joint and the moment and shear input in the foundation. So, a few-mm gap should be provided between the end of the jacket and the end section of the column to prevent the jacket from bearing against the face of the element to which the column is connected and developing compressive forces that enhance the column moment resistance.

The confinement effectiveness of thin-walled steel jackets is further reduced by their Poisson expansion due to any longitudinal compressive stresses that may develop in the jacket through its partial or full composite action with the concrete column inside (see last paragraph). If there is full composite action in the longitudinal direction, the large Poisson ratio of steel will delay confinement until concrete approaches ultimate strength and its Poisson ratio exceeds that of steel. Cages of angles at the corners and welded transverse straps or butten plates do not suffer from reduction of confinement effectiveness due to Poisson effects. Such effects can be minimised also if the continuous thin-walled steel plate is replaced by sheets corrugated in the transverse direction of the member and welded along the corners of the section. Thanks to the very low stiffness of the corrugated sheet for axial compression in the member, confinement is not reduced by Poisson effects. Moreover, the large out-of-plane rigidity of the corrugated sheet almost eliminates outward bulging of the jacket and enhances confinement. Corrugated steel jacketing has been found to be very effective for the flexural deformation capacity of columns (Ghobarah et al. 1997).

6.8.4.3 Shear Strengthening Through Steel Jackets – Dimensioning According to Eurocode 8

When they aim at enhancement of the column deformation capacity or of deficient lap splices, steel jackets are normally applied only over the plastic hinge or the lap splice region. By contrast, those intended for shear strengthening extend over the full length of the member. The jacket is inactive in shear until a major diagonal crack develops in the concrete member. Relative displacement of the two pieces on either side of such a crack causes the member to bear against the jacket and activate it. From that point on the jacket resists all the additional shear force and controls the width of the original diagonal crack, as well as development of new ones or disintegration of the concrete core due to cyclic shear. To play this role the steel jacket should remain elastic (Aboutaha et al. 1999). The contribution of the jacket to the resistance in diagonal tension is added to that of internal ties and of the concrete.

The analysis of the limited available experimental information on shear critical steel-jacketed columns suggests that the jacket contributes to the resistance in diagonal tension of the jacketed part of the member with the following shear force:

$$V_{R,j} = \eta \frac{2\, t_{\mathrm{j}}\, b}{s} h f_{yj} \cot\delta \tag{6.22}$$

where:

- η is a jacket efficiency factor, with values between 0.4 and 1.0; η seems to have values close to 0.4 for continuous thin plates or corrugated sheets and higher than 0.5 for straps or batten plates welded to corner bars.
- t_{j} is the thickness of steel straps at right angles to the member axis,
- b is the width of the steel straps,
- s is the centreline spacing of steel straps (with $b/s = 1$ for a continuous steel plate),
- h is the depth of the concrete section in the direction of the shear force,
- δ is the inclination of the compression strut to the member axis,
- f_{yj} is the yield strength of the steel of the jacket.

Annex A of CEN (2005a) adopted Eq. (6.22) with $\eta = 0.5$ (as recommended in Aboutaha et al. 1999) and with the design yield strength of the jacket steel, $f_{\mathrm{yj,d}}$ (nominal strength divided by the partial factor for structural steel).

Equation (6.22) applies throughout the length of the member and its outcome should be added to the value of $V_{\mathrm{R,s}}$ from Eq. (3.94). In the plastic hinge the value of $V_{R,j}$ for $\delta = 45^\circ$ from Eq. (6.22) should be added to the right-hand-side of Eqs. (3.114). By remaining elastic the jacket prevents cyclic degradation of the resistance in diagonal tension within the plastic hinge. So, any reduction of V_R with cyclic ductility demand, μ_θ, in Eqs. (3.114) may be neglected.

The steel jacket does not increase markedly the resistance of the member in diagonal compression (against web crushing). Like for FRP-wrapping the corresponding shear resistance, V_{Rmax}, is still as given in Section 3.2.4.2 under *The Variable Strut Inclination Truss of the CEB/FIP Model Code 90 and Eurocode 2* for slender members under monotonic or cyclic loading, or by Eq. (3.127) for squat columns (with $L_s/h \leq 2.0$) in cyclic loading after flexural yielding.

6.8.4.4 Members with Short Lap Splices and Steel Jackets

A steel jacket consisting of a continuous thin plate or corrugated sheet can clamp deficient lap splices in rectangular columns. Its thickness, t_j, may be derived from a friction-based model for the clamping action. The model (Aboutaha et al. 1996a,b) is based on:

1. A postulated shear transfer area along the lap splice, consisting of strips with length equal to the lapping, l_o, and width not more than 1.5 bar-diameters ($1.5d_{bL}$) on either side of each spliced bar. The surface-area of the shear transfer area for one spliced bar is: $A_{sf} = l_o \min(3d_{bL}, s_b)$, where s_b is the spacing between lapped bars.
2. A friction coefficient over the shear transfer area equal to $\mu_f = 1.4$ for clamping by anchor bolts, or $\mu_f = 1.0$ otherwise.
3. Bond stress along the embedment length, l_{ab}, of anchor bolts equal to $v_b = 0.042\sqrt{f_c}$ (units MN, m).
4. Clamping by tie legs which are at right angles to the potential splitting plane and enclose the lap splices, based on the yield force of the tie legs per unit length of the member, $\Sigma A_{st} f_{yt}/s_t$.
5. Clamping by the (two) sides of the steel jacket which are parallel to the tie-legs in point 4 and the anchor bolts of point 3 above, based on the yield stress f_{yj} of the steel jacket.

   The model gives (Aboutaha et al. 1996a,b):

$$2t_j f_{yj} \geq v_{sf} b - \sum \frac{A_{st} f_{yt}}{s_t} - \frac{v_b \pi d_b l_{ab} n_{ab}}{l_o} \tag{6.23}$$

where:

- b is the column width parallel to the potential splitting plane, at right angles to the tie legs and the anchor bolts;
- A_{st}, s_t are the cross-sectional area and spacing of tie-legs along the member axis;
- d_b, n_{ab} are the diameter and the total number of anchor bolts over the entire area of lap splicing, bl_o;

v_{sf} is the shear stress demand over the shear transfer area along the splice length of a single bar:

$$v_{sf} = \frac{\gamma_{Rd} A_{sL} f_{yL}}{\mu_f A_{sf}} \tag{6.24}$$

with A_{sL} and f_{yL} denoting the cross-sectional area and the yield stress of a single lap-spliced bar and γ_{Rd} a model uncertainty factor, accounting for strain hardening in the spliced bars and having a suggested value: $\gamma_{Rd} = 1.25$ (Aboutaha et al. 1996a,b).

According to Aboutaha et al. (1996a,b), unless f_c exceeds 30 MPa, the steel jacket should be back-anchored into the column just above the end of the lap splice and at about one-third of the lap length from the base section where the lapping starts, with two bolts at third-points of the column side at each level. Intermediate (rows of) bolts between these two heights are not necessary for effective clamping of the lap splices.

Test results in Aboutaha et al. (1996a,b) have shown that very high cyclic deformation capacity of retrofitted columns (to drift ratios above 5%) can be achieved with the above rules.

Annex A in CEN (2005a) recognises clamping of deficient lap splices by steel jackets consisting of continuous steel plates, but provides no model for it. It only gives prescriptive guidance, based on the test results and recommendations in (Aboutaha et al. 1996a,b):

- The steel jacket should extend beyond the end of the lapping not less than 50% of the lap length (as extension by just 20% was found insufficient in Aboutaha et al. (1996a,b)).
- The jacket should be anchored to the faces of the column by at least two rows of bolts on column sides at right angles to the direction of loading. If the splicing is at the base of the column, one of these rows of bolts should be near the bottom end of the lapping and another at one-third of its length from the column base.

Note that discrete collars built-up of channel sections fitted around the column and bolted at its corners do not provide as effective a clamping of lap splices as jackets consisting of corner angles and continuous steel plates welded on them.

6.8.4.5 Resistance and Deformations of Steel-Jacketed Members at Yielding and Ultimate

Cyclic test results on steel-jacketed concrete members are limited in the international literature. Their analysis in *fib* (2003) leads to the following conclusions:

- With a 25–50 mm gap between the end of the jacket and the member end, the yield moment and the moment resistance of the jacketed member are equal to those of the end section of the original one.

- The secant-to-yield stiffness of the retrofitted member may be taken equal to that of the original column from Eq. (3.68), neglecting the effect of deficient lap splices that have been remedied according to Section 6.8.4.4.
- The flexure-controlled deformation capacity of the retrofitted member may be taken equal to that of the original one, again neglecting the effect of deficient lap splices corrected by retrofitting according to Section 6.8.4.4. The effect of confinement on the ultimate strain concrete and on the ultimate curvature of the end section may be taken into account according to Section 6.8.4.2, but may also be neglected in view of the limited experimental confirmation. Note, though, that empirical expressions for the plastic hinge length, L_{pl}, fitted to members without retrofitting should be used with caution for steel-jacketed members.

6.9 Stiffening and Strengthening of the Structure as a Whole

6.9.1 Introduction

Interventions at the structure's level to increase global stiffness and reduce seismic deformation demands throughout the system may be more cost-effective than universal upgrading of the capacities of the existing components, if disruption of occupancy and demolition and replacement of partitions, architectural finishes and other interior non-structural components are considered. This is particularly true for flexible buildings. However, they may be less convenient for the future functionality of the building, if they require reducing openings or take up valuable floor area.

6.9.2 Addition of New Concrete Walls

6.9.2.1 Construction of the New Walls and Connection to Existing Members

Adding concrete walls is perhaps the most common technique for seismic retrofitting of buildings. It is very effective for the control of global lateral drifts and the reduction of damage in frames and non-structural elements.

If the system of new walls takes the full seismic action according to Section 6.5.8, with the existing elements verified like "secondary" ones in a new building (see point 1 in Section 6.7.2), then the new walls are designed on the basis of forces and detailed as in new buildings. When retrofitting follows point 2 in Section 6.7.2 and the walls are verified in flexure on the basis of deformations, it is still good practice to detail them as in a new building, i.e. for flexural plastic hinging at the base. To this end, the plastic hinge zone at the base is provided with boundary elements near the edges of the section, well-confined and detailed for flexural ductility. The walls are also capacity-designed in shear throughout the height according to Section 6.5.5.2.

Full continuity of the wall across storeys increases its strength. Good anchorage of the new reinforcement is essential for strength and deformation capacity.

New walls may be conveniently introduced by infilling strategic bays of the existing frame, especially at the perimeter. If the wall takes up the full bay, it can incorporate the beams and also both columns as boundary elements. Then only the web of the wall is totally new. Sometimes it is shotcreted against a partition wall, which is then encapsulated within the core of the web. The new web should be fastened to the existing beams and columns all around the infilled panel through special connectors. The fastening of these connectors to the existing members and their embedment into the new concrete should be capable of fully transferring the web shear and the tensile capacity of the web reinforcement to the frame members. Poor detailing and lack of a proper load-path between the old members and the newly constructed parts of the wall may lead to reduced global ductility or brittle failure of web panels. Moreover, if there is no integral connection between the existing and the new, the behaviour is uncertain and the reliability of modelling and verification of the wall as a single, integral element is in doubt.

For integral behaviour, the new wall should be thick enough to encapsulate the existing beams and columns. In that case holes and slots should be drilled through the slab, for the vertical bars to pass from one storey to the next and for concrete to be cast from the top. The concrete that fills the slots plays the role of shear keys between the new wall and the slab. For fully integral behaviour epoxy-grouted dowels may be placed throughout the interface of the old concrete and the new, at about 0.5 m centres. Even when it does not encapsulate the existing beams, the new wall may have to do so for the columns, to provide the lacking confinement reinforcement, especially if the columns have short lap splices (see Fig. 6.5(a) for an example).

It is essential to ensure the transfer of inertia forces from the floors to the new walls. A wall created by infilling a bay of a frame may be considered as adequately connected to the floor diaphragms if it encapsulates the beams, or if its web panels are well fastened to the surrounding frame members and the floor slab is integral with the beams. Often "collector" elements may need to be added and designed for the transfer of floor inertia loads to the new walls. If the new wall is at the perimeter, a steel tie can be fastened to the side of perimeter beams to collect the floor loads and transfer them to the new wall, where the end of the tie is embedded for anchorage. In the example of Fig. 6.6 the steel tie is fastened by welding on steel plates anchored to the side of the perimeter beam. The collectors and their anchorage to the perimeter beams may be covered with shotcrete for protection. They should be dimensioned for seismic action effects based on capacity-design considerations, accounting also for higher-mode effects. Nonlinear response-history analyses of wall-frame systems show that such effects on peak floor forces are much larger than on storey shears (which are the cumulative effect of floor forces). Equations (1.16) in Section 1.3.6.4 and (6.7) in Section 6.5.5.2 are a good guide for the magnification factor to be applied on the seismic loads transferred from the floor to the new wall according to the analysis for the seismic action.

If it is not feasible to encapsulate the beams and columns of the frame bay, the new wall may be created by fully infilling with RC the space between them. The connection of the new web with the beams and the columns is more critical than when these members are fully encapsulated. Even with very good shear connection,

(a)

(b)

(c)

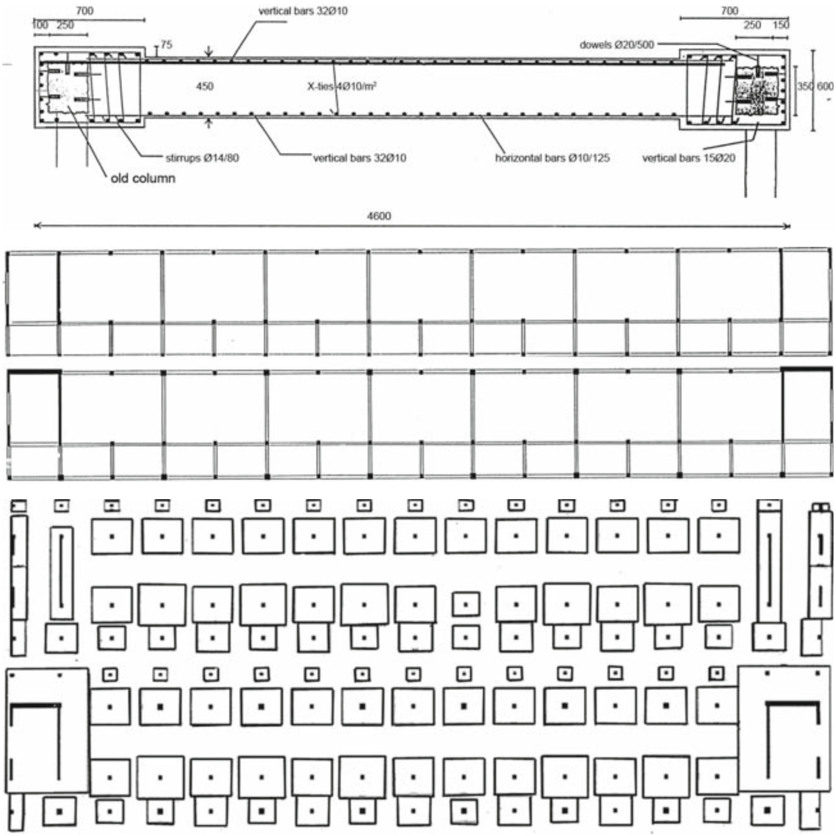


Fig. 6.5 3-storey building strengthened in long direction with two new walls: (**a**) cross-section of added wall; (**b**) plan of framing, before (*top*) and after retrofitting (*bottom*); (**c**) foundation plan, before (*top*) and after retrofitting (*bottom*)

integral behaviour of the old and the new cannot be presumed and the force and moment resistance or the deformation capacity of the system cannot be quantified with any certainty. Very instructive in this respect are the ultimate strength and deformation results of several one-bay, one-storey RC frames converted to walls by infilling them with RC of thickness (about) half the column width at right angles to the plane of the frame (JCI 2007). The infilled frames were invariably predicted to be shear-critical, with the connection of the new web to the surrounding members being the weak link. The force resistance derived from their predicted flexural capacity was always about double the predicted shear resistance. Companion monolithic wall specimens were tested for comparison. Their resistance was also governed by shear, although the margin between the predicted flexure or shear force resistances was narrower, as the connection was not anymore the weak link. In all types of specimens the experimental ultimate strength was always much higher than

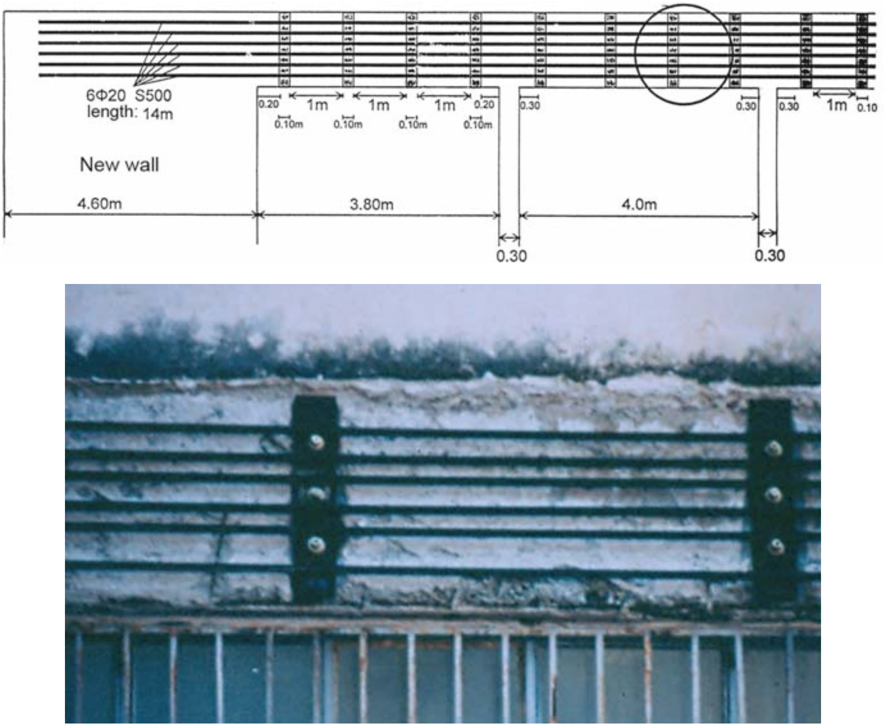


Fig. 6.6 Collector element of the wall in Fig. 6.5, fastened to the side of the perimeter beam (See also Colour Plate 16 on page 729)

the predicted shear resistance and closer to (but less than) the flexure-controlled prediction. The specimens resulting from conversion of a frame to a wall had on average an experimental ultimate strength equal to 92% of that of their monolithic counterparts for dowels connected to the old concrete through adhesive (14 specimens), or 87% for mechanical connection (3 specimens). Two wall specimens resulting from doubling the thickness of a thin web monolithic with the frame had experimental ultimate strength and ultimate deformation 97 and 105%, respectively, of those of their fully monolithic companions. In that case the new thickness of the wall was connected to the frame through dowels with adhesive. In a specimen of the same type but without shear connection other than what was provided by the original monolithic half of the wall thickness, the experimental ultimate strength and ultimate deformation were 83 and 50%, respectively, of those of the fully monolithic ones. Interesting and very important is the experimental ultimate deformation of the specimens resulting from conversion of a frame to a wall. On average it was 175% of that of their monolithic counterparts for shear connection through dowels with adhesive (14 specimens), or 270% for mechanical connection (3 specimens). The deformability of the shear connection seems to increase, therefore, very much the ultimate deformation at little expense of the ultimate strength.

As all the walls in JCI (2007) were shear critical owing to their thin webs and low slenderness (height-to-length ratio), they are not representative of multi-storey frames converted to walls by full-thickness RC-infilling. Their results are more of qualitative value, showing the general trends, the importance of the connection and the uncertainty of the behaviour.

It is not good practice to stop the new wall at a lower storey. For example, a three-storey building with walls added only to the first storey after the 1968 Tokachi-oki earthquake suffered heavy damage at the second storey in another earthquake in 1994 (Nakano 1995).

6.9.2.2 Foundation of New Walls and Impact of its Fixity on Wall Effectiveness

New walls should have proper foundation. As it typically has large cross-section, a new wall is expected to develop high seismic moments at the base. By contrast, its gravity load is low. Unless its foundation element incorporates existing footings (e.g., when the new wall is created by infilling a bay of a frame), its vertical load is not much larger that the self weight of the wall and the foundation element. At any rate, the most serious problem and drawback of the technique is the difficulty to transfer the wall base moment to the ground and the need of a major, costly and disruptive intervention to the foundation.

As emphasised in Section 2.2.2.3, isolated footings of large walls uplift and rock during the earthquake. Uplifting reduces the wall base moment well below the value obtained from constant foundation impedance. Although the rocking wall still acts as a stiff vertical spine and prevents storey mechanisms, rocking increases considerably the lateral drifts at floor levels and the chord rotation demands in beams, especially in those directly framing into the wall within its plane (those chord rotations will be about equal to the rotation of the wall base at the ground). If they are not retrofitted too, these beams may fail under such demands.

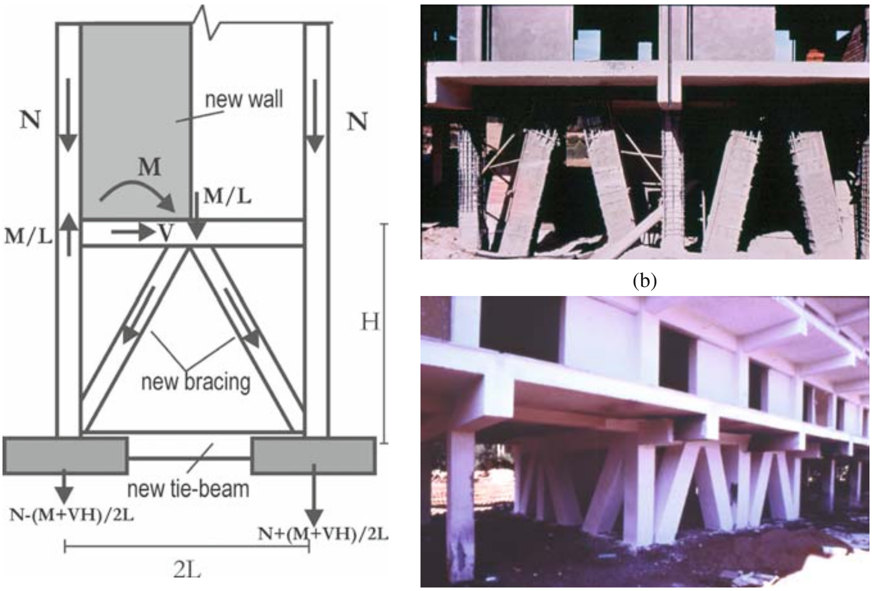
If we want the new wall to play its traditional role as a major element of lateral stiffness and strength fixed at the base, we should greatly reduce or even prevent uplifting and rocking. This can be achieved by one or more of the following:

a. by increasing the size of the new footing in plan, to increase its weight and the impedance of the underlying soil and/or to incorporate the footings of adjacent columns and mobilise their vertical load against the uplift (as, e.g., in Fig. 6.5(c));
b. by connecting the new foundation element to neighbouring ones through stiff and strong grade- or tie-beams (see Fig. 6.7 for an example); or
c. by tie-downs of the new foundation element (e.g., micropiles with large tensile strength).

Implementation of these solutions is so disruptive and costly that may discourage adding new walls to buildings not having already a stiff and strong (almost storey-high) foundation beam around the perimeter, to which a new wall could be conveniently anchored.

Fig. 6.7 New wall founded on foundation beam encapsulating footings of nearby columns

A system used by Tsiknias and Pittas (1992) to transfer the base moment of new walls to the soil without large uplifts and base rotations was to construct the new wall over half the bay width *L* of the frame with its base at the lowermost floor above the foundation and to connect that base to the existing spread footings of the



(a) (b) (c)

Fig. 6.8 Foundation of new wall through diagonal concrete bracing: (**a**) transfer of wall forces to the ground; (**b**) shotcreting of braces and of column jackets; (**c**) retrofitting completed two columns of the bay through Chevron (inverted-V) bracing (Fig. 6.8(a)). The moment M at the base of the new wall just above the Chevron bracing is converted into a couple of vertical forces, $\pm 2M/L$. One of them is applied to the joint of the Chevron bracing at beam mid-span and transferred to the two footings by tension and compression in the two bracings. The other force goes directly to the footing underneath, through the column. The end result is a couple of concentric vertical forces on the existing footings, $\pm M/L$. The axial load N in the existing footing prevents uplift, up to a value of the moment $M = NL$ at the base of the wall, much higher than the uplift resistance of an alternative footing large enough to accommodate the new wall and the existing column. The base rotation is equal to the differential settlement of the two footings divided by L and is low. So, the new wall is very effective in limiting lateral drifts. Implementation of this scheme requires jacketing the ground storey beam and the two columns to connect the Chevron bracing and accommodate the internal forces arising from it, possibly enlarging the footings for the connection to the bracings and the new tie-beam, if one is constructed (Fig. 6.8). The role of the tie beam is just to transfer to the adjacent footing the seismic shear in the event a footing uplifts.

Unless rocking of the base of the new wall is effectively prevented as highlighted above, it should be appropriately modelled in the analysis, to realistically capture its effects on the seismic deformation demands in the superstructure. As emphasised in Section 4.10.3, conventional rotational springs under a footing with constant stiffness, corresponding to full contact of the footing to the ground, are not sufficient. They account neither for the softening of the rotational spring due to uplifting nor for the accompanying vertical displacement of the centre of the footing. Such effects may be captured in nonlinear analysis (static or dynamic) by using a pair of nonlinear vertical springs at opposite ends of the footing that account for uplifting according to Eqs. (4.100), (4.101), (4.102), (4.103) and (4.104) in Section 4.10.3.

To help appreciate the effect of uplifting of the footing of a large wall, the results of a parametric study are presented here (Panagiotakos and Fardis 2001b). A 5-storey frame with five 5 m bays is assumed to be retrofitted by converting its central bay to a wall, incorporating the corresponding beams and columns. Nonlinear static analyses in 2D have been carried out. The one-component point-hinge model of Section 4.10.1.4 has been used and the assumptions in Section 4.10.5.1 relevant to the present case (no. 1–5 and no. 8) are employed for all beams and columns. Unlike the wall, all columns are fixed at the base. Two types of nonlinear models are used for the new wall:

1. A "conventional" one-component point-hinge model according to Section 4.10.1.4 and the relevant assumptions (no. 1–5) in Section 4.10.5.1. Bending takes place about the centroid of the wall section, without axial-flexure interaction.
2. A "new" model which differs from the "conventional" one only in that bending takes place about the current neutral axis. So, flexure induces axial deformations and vertical displacements of the centroid of the section.

For the wall footing, the model of Section 4.10.3, Eqs. (4.100), (4.101), (4.102), (4.103) and (4.104), is used.

Figure 6.9 shows a typical pattern of frame deformation, plastic hinging, etc., at an instant of the response when beam failures take place, using Model no. 2 for

(a)

(b)

(c)

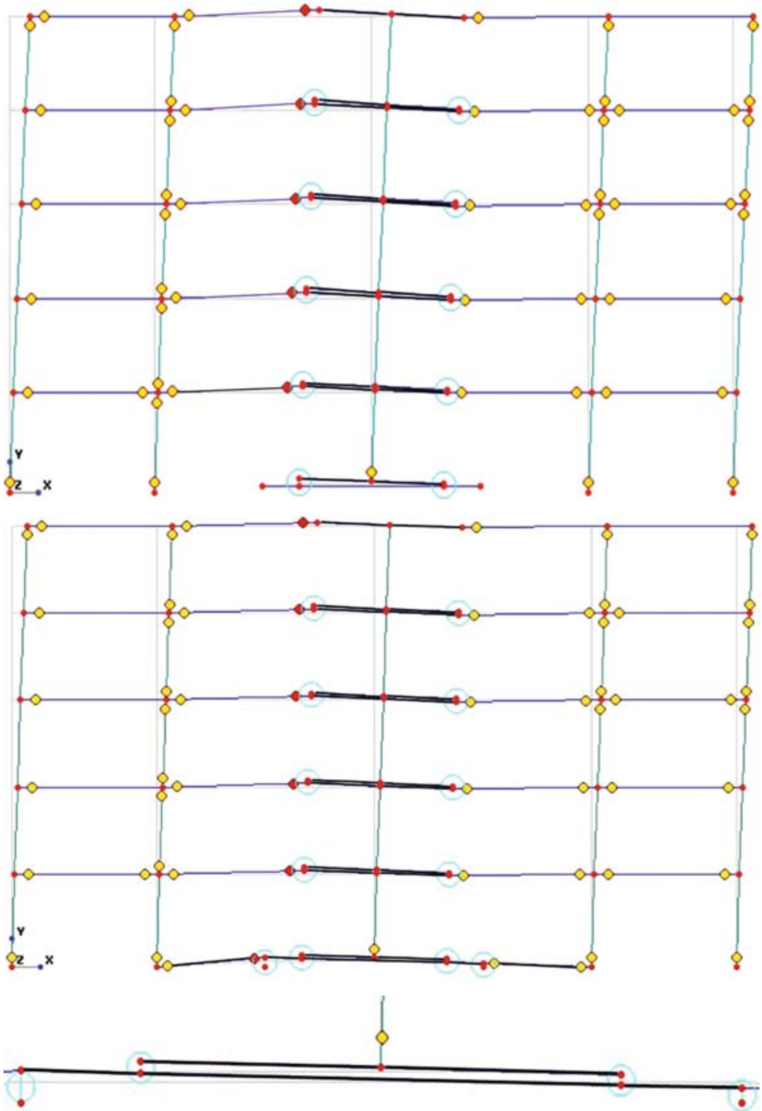


Fig. 6.9 Deformation and plastic hinging in retrofitted frame from nonlinear static analysis: (**a**) wall fixed at the base; (**b**) wall footing connected to tie-beams and uplifting; (**c**) detail of (**b**) at the base of the wall and the footing (*light-coloured circles*: plastic hinges; larger *dark circles*: flexural failure) (See also Colour Plate 17 on page 730)

the wall. When the wall is considered fixed at the foundation (Fig. 6.9(a)) there is a large upward displacement of the ends of beams framing into the "windward" side of the wall owing to the rotation of the wall section about its neutral axis. When the model includes uplifting (Fig. 6.9(b)) these upwards displacements are due to the rotation of the wall footing. In both cases these displacements drive these beam ends (including that of the tie-beam in Fig. 6.9(b)) to ultimate flexural deformation.

Figure 6.10(top), referring to a wall footing without tie-beams, compares:

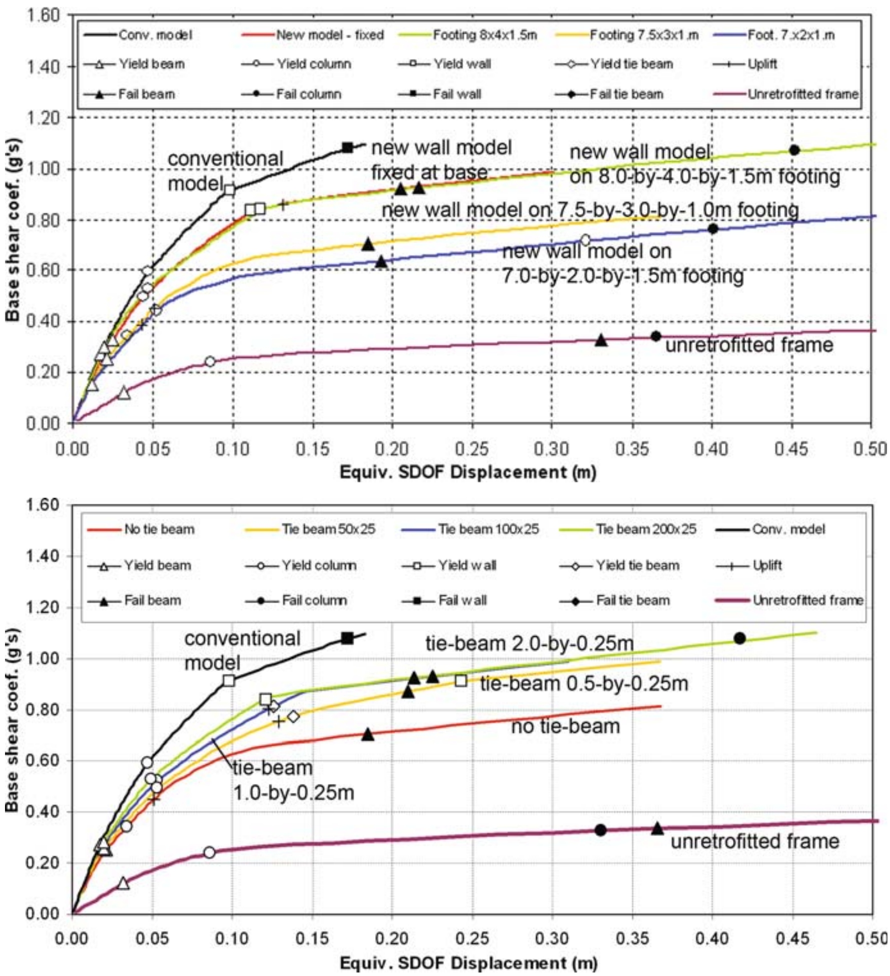


Fig. 6.10 Pushover response of retrofitted frame: (*top*) without tie-beams, using as parameters the model and the size of the footing; (*bottom*) with 1 m deep, 7.5 m-by-3 m footing, using as – parameter the size of the tie-beam (See also Colour Plate 18 on page 731)

- the four "pushover curves" obtained from wall Model no. 2 above and the model of Section 4.10.3, Eqs. (4.100), (4.101), (4.102), (4.103) and (4.104) for the footing, to
- the uppermost "pushover curve" given by Model no. 1 above with the wall fixed at the base, and the lowermost one for the unretrofitted frame.

The "new" wall model by itself produces overall a more flexible response, delaying wall yielding and preventing wall flexure failure (within the range of response considered here). A 1.5 m deep, 8 m-by-4 m footing uplifts shortly after wall yielding, but gives essentially the same overall response as full fixity at the base. Uplifting of the two smaller-size footings considered in Fig. 6.10(top) prevents wall yielding, but causes beam failure(s) a bit earlier.

Figure 6.10(bottom) uses as baseline the moderately-sized 1 m deep, 7 m-by-3 m footing and connects it to those of the adjacent columns via tie-beams of various sizes. A quite small tie-beam (0.5 m-by-0.25 m) is remarkably effective. It delays uplifting but allows yielding at the base of the wall later on (after the first flexural failure of a beam, however). Increasing further the size of the tie-beam delays almost indefinitely yielding at the wall base, but does not affect uplifting very much. Interestingly, in all cases the tie-beam yields right after the footing uplifts. A tie-beam as large as a deep foundation beam (2.0 m-by-0.25 m) provides almost full fixity of the wall, preventing uplift but accelerating yielding at the base of the wall.

The conclusions of this parametric study may be summarised as follows:

- A new wall is very effective for retrofitting a frame building.
- The effectiveness of the wall is grossly overestimated if its sections are assumed to rotate about its centroid in lieu of its neutral axis and/or if uplifting of the footing is neglected.
- A footing heavy enough, or a tie-beam sufficiently stiff, to prevent uplifting until the wall yields, are equivalent to wall fixity at the base.
- A normal-size footing without tie-beams is moderately effective.
- A normal-size footing with tie-beams of moderate or even small size has almost the same overall result as full fixity of the wall at the base.
- The sequence of events (yielding or failure of members, uplifting, etc.) is influenced markedly by the size of key foundation elements. Its prediction is affected very much by how we model the wall and the interaction of its footing with the ground.

6.9.3 Addition of a New Bracing System in Steel

6.9.3.1 Introduction

Adding diagonal bracings to selected bays in all storeys of a frame structure, or to just one or few weak storeys (e.g., to an open storey, see Fig. 6.11(c)), is effective for global strengthening and normally not as disruptive as adding walls. For convenience and minimal disruption the bracing is usually – but not exclusively – placed at the façades. Architectural constraints and openings may condition the layout of the bracings.

Normally the intervention to the foundation is minimal (see discussion in Section 6.9.2.2 about the concept in Fig. 6.8). The prime challenge in this technique is the connection of the bracings to the existing concrete elements.

Bracings are normally made of structural steel. Steel bracing increases markedly the lateral force resistance of a concrete frame, but not so much its lateral stiffness. Therefore it is not so effective for stiff buildings, such as wall or dual systems and masonry-infilled frames. The overall deformation capacity of flexible, non-ductile concrete frames may be considerably enhanced by steel bracing, provided that we prevent early brittle failures of braces and their connections, or of shear-critical concrete members.

Passive energy dissipation devices may be introduced in the bracing for supplemental damping. These devices normally require large response displacements. So, they may not be cost-effective if the bracing system increases the lateral stiffness considerably.

To design a seismic retrofitting with steel bracings, it is essential to have a good command of seismic design of steel (and composite, steel-concrete) buildings. This subject is outside the scope of this book. So, the following sections focus on aspects specific to seismic retrofitting with steel bracings, without venturing a comprehensive coverage of the subject.

6.9.3.2 Layout and Conceptual Design of Concentric Bracing Systems

Normally, steel bracing systems applied for retrofitting are concentric (see Fig. 6.11 for examples), contributing to lateral-load resistance and stiffness and to energy dissipation through the axial forces in their inclined braces. Their dissipative elements are only the tension braces.

Appropriate (concentric) bracing systems are those with:

1. X-diagonal ("cross-diagonal") bracings, along both diagonals of the braced bays. This is overall the preferable arrangement.

(a) (b) (c)

Fig. 6.11 Examples of retrofitting with steel bracings (University of Tokyo buildings)

2. Diagonal bracings, with a single diagonal per braced bay. The arrangement of bracings in different bays (or groups of adjacent bays) should give similar lateral resistance and stiffness for the two senses of the seismic action in the plane of bracing. For new braced steel frames, Part 1 of Eurocode 8 (CEN 2004a) restricts the difference of the total horizontal projections of the cross-sectional area of all tension diagonals for the two senses of action to less than 10% of their average value.
3. V-bracings (Fig. 6.11(a)) or inverted-V ones ("Chevron" in US), where a pair of inclined braces is connected to a single point near or at mid-span of a beam. Their advantages include the reduced unbraced length of the braces and the earlier mobilisation of their strength and dissipation capacity.[11] Chevron bracings in particular are most convenient for openings and passage through the braced bay. The horizontal member to which the two braces connect should (be strengthened to) resist a transverse force equal to the difference in the vertical force components of the tension and compression braces. For this purpose, the post-buckling force of the compression brace is normally conservatively taken as 30% of the yield force or one-third of the buckling load.

K-bracings, where the inclined braces are connected to columns at about their mid-height, should be avoided, because the unbalanced force after buckling of the compression brace mentioned above for V-bracings is applied to the columns and may cause them to fail.

If the sense of V-bracings is reversed in adjacent storeys (with an inverted-V in one storey and a V-bracing in the storey above, see Fig. 6.11(b)), the braces in consecutive storeys are almost continuous and there is no unbalanced transverse force on the beam. If the braces are connected to the top and bottom of a beam but, unlike in Fig. 6.11(b), their centrelines do not pass through the same (nodal) point, a concentrated moment is applied on the beam, equal to the sum of the horizontal projections of the tension and compression forces in the braces above or below, times the beam depth.

Composite action with the existing concrete frame and transfer of forces to the bracing are facilitated if the bracings are added within the plane of the frame, with as little eccentricity as possible to the centrelines of the existing beams and columns (Fig. 6.11(a) and (c)). Bracings placed outside the existing frame at the façade (Fig. 6.11(b)) intrude the least with the use of the building and may allow normal operation during retrofitting. However, their connection with the existing frame – usually through (a combination of) post-installed fasteners, grout mortar or even transverse post-tensioning through the concrete beams – is critical and tricky. When the exterior concrete columns are not flush with the beams but protrude from them, it is better to install the bracing between (the protruding parts of) adjacent columns,

[11] Braces yield at an interstorey drift ratio of $2\varepsilon_y/\sin 2\theta$, where ε_y is the yield strain of the brace and θ its inclination to the horizontal. So, the closer to 45° a brace is, the sooner the storey will mobilise its yield force and dissipation capacity.

bearing on them. Attaching in this case the braces to the column through a vertical steel member along its protruding part, helps the connection through interface shear.

Diagonal braces installed fully inside the bays of the existing frame should be supplemented with a frame ("rim") of horizontal and vertical steel members firmly attached to the surrounding concrete members. The rim helps the concrete frame resist the load effects (moments and shears) from any frame action that accompanies the truss action, as well as from the truss action itself (axial forces in the horizontal chords of the truss). Besides, its horizontal members may act as collectors, transferring the inertia forces from the floors to the bracing system. To this end they should be continuously fastened to the corresponding horizontal concrete members. The vertical members of the rim serve as a backup system for gravity loads, in case (some) concrete columns cannot sustain the imposed storey drifts and fail. For this, these members should be made continuous from storey to storey, via threaded rods or other elements passing through the slab.

For convenience of fabrication, the braces may have the same cross-section in several storeys. If their common size is dimensioned on the basis of the upper storeys, the lower storeys will develop the additional lateral force resistance needed through flexural action in the existing frame, possibly with some of its members acting compositely with new horizontal and vertical steel members surrounding the braces. If the braces are dimensioned on the basis of the lower storeys, inelasticity and energy dissipation demands will be concentrated there, owing to the overstrength of the upper storeys. To prevent such situations in new braced steel frames, Part 1 of Eurocode 8 (CEN 2004a) limits the maximum value of the brace overstrength ratio (: ratio of brace force resistance to demand from the analysis) to 1.25 times its minimum value anywhere in the structure.

6.9.3.3 Recommendations for the Design and Detailing of Braces

Experience from past earthquakes shows that steel frames with concentric bracing may fail prematurely by cracking and fracture of the braces and their connections after buckling. The key to good performance of such a system is its ability to withstand post-buckling cyclic deformations without premature fractures, through proper design and detailing of the braces to control the buckling and post-buckling behaviour and the associated adverse effects, like distortion and local buckling, early failure of welds, etc. Such phenomena may jeopardise the full tensile capacity of a brace, after it is straightened back during the next half-cycle of the response. The post-buckling response of braces, particularly of double-channel or double-angle ones, can be improved by welding closely spaced batten plates.

Local buckling in compression bracings precipitates fracture, owing to concentrations of strains or strain accumulation with cycling. To avoid local buckling, "compact" sections should be used, with low width-to-thickness ratios. For braces used in seismic design or retrofitting, the upper limit to this ratio should be considerably lower (e.g. by 50%) than for monotonic loads. For new steel buildings, Part 1 of Eurocode 8 (CEN 2004a) requires using:

– "class" 1 steel sections, if the q-factor 4 or more,
– "class" 1 or 2 sections, for q between 2 and 4, or
– "class" 1, 2 or 3 sections, with q between 1.5 and 2.

These "classes" are defined in EN-Eurocode 3, Part 1, depending on the shape of the section and its width-to-thickness ratio. For bracings in seismic retrofitting it is strongly recommended to avoid "class" 3 sections. If possible, "class" 1 should be used.

No matter whether compression braces are taken into account in the analysis for the retrofit design (as in V-bracings), or neglected (as may be the case in diagonal or X-braced systems, see Section 6.9.3.4), their slenderness should be capped. A sensible upper limit is the value of 2.0 imposed by CEN (2004a) to the non-dimensional slenderness $\bar{\lambda}$ (defined as the square root of the ratio of the member's yield force, f_yA, to its critical buckling load, N_{cr}) in new steel structures. CEN (2004a) sets a lower limit of 1.3 on $\bar{\lambda}$ for compression braces which are neglected in a linear analysis for the seismic action. This is to reduce the axial force that will inevitably develop in the (neglected) compression braces during the pre-buckling stage and prevent columns and beams from being overloaded with seismic action effects (much) higher than given by the linear analysis and damaged before the tension diagonals yield. If compression braces are included in a nonlinear analysis, there is little sense in observing a lower limit on $\bar{\lambda}$.

Realistic end restraint assumptions should be made for the effective unbraced length of braces. For X-braces welded to a common gusset plate at their midpoint(s) (ASCE 2007) recommends taking the effective unbraced length equal to half the total (diagonal) length of the brace, including the gusset plates. For other types of braces welded to gusset plates, an effective unbraced length equal to the total length of the brace is recommended for out-of-plane buckling, or 80% (90% for bolted connections) of that length for in-plane buckling.

A gusset plate connected to a brace susceptible to out-of-plane buckling should have clear length at least equal to twice its thickness, to limit restraint of brace plastic rotations in the post-buckling stage (ATC 1997).

6.9.3.4 Seismic Analysis and Design of the Retrofitting

The existing structure is already loaded by the quasi-permanent gravity loads before any steel bracing is added. It continues to support these loads fully, even when the earthquake comes.[12] If the analysis for the seismic action of interest is linear, one analysis should be carried out for the concrete frame alone under the quasi-permanent gravity loads and another for the retrofitted structure under the seismic action. The results are superimposed. If the analysis is nonlinear, quasi-permanent

[12]The seismic action will not cause serious axial distress to the existing vertical members (unless they fail). So, gravity loads will not be redistributed from the existing system to the steel bracing, when the earthquake comes.

gravity loads and the seismic action should be taken to act together. The quasi-permanent gravity loads are applied first, to provide the initial conditions. In that case an artificially high axial stiffness may be used from the outset for the existing vertical members, so that they monopolise the support of gravity loads. Note that, even though by the time the factored gravity loads may be applied the steel bracing will be in place, these loads should be taken to be resisted by the existing concrete frame alone. It is standard (and codified) practice for new steel or composite (steel-concrete) buildings with bracings to design the frame alone for the factored gravity loads and rely on the bracings only for earthquake resistance. V- or inverted-V braces are not taken to provide support to the beams at mid-span under gravity loads.

The most cost-effective approach for the design of the retrofitting with steel bracings is the standard one in Sections 6.5.4, 6.5.5 and 6.5.6, with deformation-based verification of ductile mechanisms in all members, old or new (including the bracing elements intended for energy dissipation) and force-based verification of brittle mechanisms. In this approach all members (including the existing ones, as well as the compression braces) are included in the model. If the analysis is nonlinear (typically static), compression braces may be modelled using an elastic-perfectly plastic force-deformation law in primary loading, with yield force equal to a small fraction of their buckling load – 20% is recommended in ASCE (2007) and ATC (1997). Notwithstanding this low limiting force in compression, the full buckling load should be assumed to develop in these compression braces when checking vertical members connected to these braces.

Table 6.1 still applies for the verification of the members of the concrete frame. All beams and columns around a bay where bracing is added should be considered as "primary elements". Table 6.4 presents the verification criteria for steel braces at the three Limit States (LS) given in Annex B of CEN (2005a) for diagonal braces in existing steel or composite (steel-concrete) buildings. Except at the Damage Limitation LS, the criteria for tension braces are more relaxed than for compression ones.

Table 6.4 Compliance criteria for steel braces in concentric bracing (CEN 2005a)

	Section	Damage limitation	Significant damage	Near collapse
Tension		$\delta_{E,t}{}^{1} \leq 0.25\delta_{y,t}{}^{2}$	$\delta_{E,t}{}^{1} \leq 7\delta_{y,t}{}^{2}$	$\delta_{E,t}{}^{1} \leq 9\delta_{y,t}{}^{2}$
Compression	Class 1	$\delta_{E,c}{}^{1} \leq 0.25\delta_{crit}{}^{3}$	$\delta_{E,c}{}^{1} \leq 4\delta_{crit}{}^{3}$	$\delta_{E,c}{}^{1} \leq 6\delta_{crit}{}^{3}$
	Class 2	$\delta_{E,c}{}^{1} \leq 0.25\delta_{crit}{}^{3}$ $0.5\, N_{pl,Rd}{}^{4} \leq N_{cr}{}^{5}$ $N_{E,c}{}^{6} \leq 0.8\, N_{pl,Rd}{}^{4}$	$\delta_{E,c}{}^{1} \leq \delta_{crit}{}^{3}$ $0.5\, N_{pl,Rd}{}^{4} \leq N_{cr}{}^{5}$	$\delta_{E,c}{}^{1} \leq 2\delta_{crit}{}^{3}$ –

[1] $\delta_{E,t}$, $\delta_{E,c}$ denote the axial deformation of the brace in tension or compression, respectively, from the analysis for the seismic action of interest without gravity loads.
[2] $\delta_{y,t}$ is the axial deformation of the brace at yielding in tension.
[3] δ_{crit} is the axial deformation of the brace at the critical buckling load, N_{cr}.
[4] $N_{pl,Rd}$ is the plastic resistance of the cross-section to normal forces.
[5] N_{cr} the critical buckling load of the brace.
[6] $N_{E,c}$ is the compressive axial force of the brace from the analysis for the seismic action of interest plus the concurrent gravity loads.

So, the limiting deformations of the compression braces normally govern. In reality, if the seismic action is likely to induce more than one half-cycle of large inelastic excursions, we cannot rely too much on the large limiting deformations of tension braces.

Braces yield at an interstorey drift ratio of $2\varepsilon_y/\sin 2\theta$, where ε_y is the yield strain of the brace and θ its inclination to the horizontal. This gives interstorey drifts between 0.4 and 0.6%, which is normally less than what causes storey yielding in a flexible concrete frame. The limit values in Table 6.4 for tension braces at the Significant Damage LS correspond to about 7-times these interstorey drift values, normally beyond what the members of a poorly detailed, deficient concrete frame can take, even when classified as "secondary" ones. By contrast, these members can easily take the interstorey drifts corresponding to the limit values given in Table 6.4 for compression braces at the Significant Damage LS. Another point is that the limit values in Table 6.4 for the Damage Limitation LS correspond to quite low interstorey drifts, namely between 0.1 and 0.15%. All this said, the limits in Table 6.4 derived from values in CEN (2005a) for steel or composite frames, may have to be assessed by applying them to actual retrofitting cases and studying their implications.

The alternative to the above standard approach for the design of the retrofitting with steel bracings is the force-based approach of Section 6.5.8. In that case Part 1 of Eurocode 8 (CEN 2004a), including its Sections on steel buildings (or composite, steel-concrete, wherever relevant), is applied according to Section 6.5.8. Note that, according to CEN (2004a), compression braces should be neglected in the (linear) analysis for the seismic action, except in V-braced systems. Note also that existing beams and columns around a bay where bracing is added cannot be considered as "secondary elements". As a matter of fact, in order to concentrate inelastic action and energy dissipation in the (tension) braces, CEN (2004a) wants some overstrength against the results of the analysis in the frame members. To this end, it requires frame members of new braced steel buildings be dimensioned for the seismic action effects (axial load and bending moment) from the analysis times 110% of the minimum value of the brace overstrength factor (ratio of brace force resistance to demand from the analysis) in all diagonals of the system, plus the action effects of the concurrent gravity loads. If the force-based approach of Section 6.5.8 is adopted, this rule should be applied to the existing members around bays where bracing is added, these members taken as "primary seismic".

6.9.3.5 Construction Issues

Normally there are more construction difficulties in retrofitting with steel bracings, than in construction of new steel or composite buildings.

Prefabrication of large steel subassemblies, as in new construction, is not easy. Most of the welding has to be done in situ and high quality full penetration welds are difficult to achieve. Weldments may end up being the weak link in the retrofitting.

The steel elements may be connected to the surrounding concrete frame through epoxy-grouted fasteners post-installed into the concrete. Normally this is done in phases:

- partial-depth holes are drilled in the concrete using the steel member's annuli as a template;
- the steel member is removed, to finish the drilling and clean the holes;
- the steel member is put back and fixed in place, after grouting the holes with epoxy and inserting the fasteners.

Sometimes interference with reinforcement or with connections between steel members may prevent drilling the holes for some fasteners to full depth. Another location may have to be sought for these fasteners, with new annuli drilled through the steel member's in the field.

In Japan, where retrofitting with steel braces is popular, some of the construction difficulties noted are by-passed using pre-fabricated bracing subassemblies, consisting of a heavy rectangular rim with X-, V- or inverted-V bracings inside. To accommodate variations in the dimensions of the existing bays where the subassemblies are placed, a tolerance of several centimeters is provided around the rim. The subassemblies are connected to the surrounding frame members through a system consisting of:

- closely-spaced headed studs, welded to the outside of the perimeter of the rim and protruding into the gap;
- post-installed fasteners in-between the studs, epoxy-grouted into the interior face of the surrounding frame;
- non-shrink mortar filling the gap between the rim and the frame.

Spiral reinforcement is often inserted along the mortar joint, between the fasteners and the headed studs. Tests in Yamamoto (1993) have shown that one-storey, one-bay frames retrofitted in this way can exhibit a lateral force resistance equal to the sum of the shear capacity of the frame and of the braces (: horizontal projection of the yield force of tension brace and of the buckling load of compression brace) at storey drift ratios over 3%. However, the size and strength of the steel braces could not increase indefinitely, without the horizontal mortar joint becoming the weak link in sliding shear. The horizontal failure plane extends then into the "wind-ward" column, which is in tension and may precipitate a flexure-shear failure of the column on the other side. The lateral force resistance is then the sum of: (a) the resistance of the mortar joint and of the "wind-ward" column in sliding shear, and (b) the shear resistance of the "lee-ward" column. The cyclic drift capacity of this failure mode is smaller, but in the tests in Yamamoto (1993) it was still between 2 and 3%.

An expensive alternative is to fully fill the gap between the steel rim around the bracing and the concrete members with epoxy resin (JCI 2007).

6.10 Application Case Studies

6.10.1 Seismic Retrofitting of SPEAR Test-Structure with RC or FRP Jackets

Section 4.10.5.2 has highlighted the two phases of retrofitting of the 3-storey SPEAR building:

- In the 1st phase, any damage inflicted to the unretrofitted structure was repaired; then:
  - the ends of all 0.25 m columns in all storeys were wrapped with two layers of uni-directional Glass FRP (GFRP) over 0.6 m from the face of the joint, for confinement and clamping of short lap splices (Figs. 4.13(b) and 6.12(a));
  - the full height of the large (0.25 m-by-0.75 m) column C8 was wrapped in two layers of bi-directional GFRP, for confinement and lap splice clamping, but mainly for shear strengthening (Fig. 6.12(b));

(a) (b)

Fig. 6.12 FRP-retrofitted SPEAR test structure: (**a**) column with FRP-wrapped ends during the response; (**b**) large column retrofitted in shear (See also Colour Plate 19 on page 732)

- the exterior faces of corner joints were strengthened in shear with two layers of bi-directional GFRP, without continuity with the FRP wrapping of the columns (Figs. 4.13(b) and 6.12).

- In the 2nd phase:
  - all FRPs were removed, and
  - the central columns of the two "flexible" sides (C2 and C6 in Fig. 4.14(a)) were RC-jacketed from 0.25 to 0.4 m square (Fig. 4.14(b)); the jackets had eight 16 mm-dia. vertical bars (three per side) and a 10 mm-dia. perimeter tie at 100 mm centres; jacketing virtually eliminated the largest of the two eccentricities between the (computed) centres of stiffness or resistance and the centre of mass and reversed the other eccentricity (Fig. 4.14(c) and (d)).

Each retrofitted version of the structure went through a test under a 0.20 g PGA bidirectional motion, for which predicted and measured displacements have been compared in Fig. 4.15 and predicted damage ratios were presented in Fig. 4.16. It was then subjected to a 2nd test with a PGA of 0.30 g. When the FRPs were removed after that test there was no visible damage. This is consistent with the predictions of the analysis in Section 4.10.5.2. The flexural damage ratios in the 0.30 g test are about 1.5-times those at the middle row of Fig. 4.16 (left) and are well below 1.0. The shear damage ratios are only slightly higher than the values at the middle row of Fig. 4.16 (right) and again do not approach 1.0. As shown in Fig. 6.12(a), the large storey drifts of the response took place mainly through fixed end rotations at the column end sections, due to (harmless) slippage of the (smooth) vertical bars from the joints. The performance demonstrates that even light FRP-wrapping of all members improves their deformation capacity sufficiently to enable an originally poorly designed and detailed, flexible and asymmetric structure withstand ground motions with a PGA as high as 0.30 g.

The flexural damage pattern of the version of the structure with the two central columns of the "flexible" sides RC-jacketed, was indeed as predicted in Fig. 4.16 (bottom, left). The jacketed columns went through the test unscathed, but the non-jacketed ones suffered very severe damage; especially the heavily loaded central column and those on the sides of the perimeter opposite to the two jacketed columns. The central column (C5 in Fig. 4.14) failed in flexure at the 2nd storey – as predicted at the bottom row of Fig. 4.16 (left) – as well as at the 1st storey. As a matter of fact, this is the column of Fig. 3.27(c). In the 0.30 g test that followed this column and C4 (the immediately most critical one according to Fig. 4.16) disintegrated completely. Nonetheless, their axial load was redistributed to other ones and the structure did not collapse. Consistently with the shear damage pattern at the right-hand-side of Fig. 4.16, there was no indication of shear effects in the damaged or failed members.

The conclusion of these tests is that in an asymmetric structure it is not sufficient to selectively upgrade the stiffness, strength and deformation capacity of critical columns. This may shift deformation demands to other elements and cause them to fail, if they have not been retrofitted.

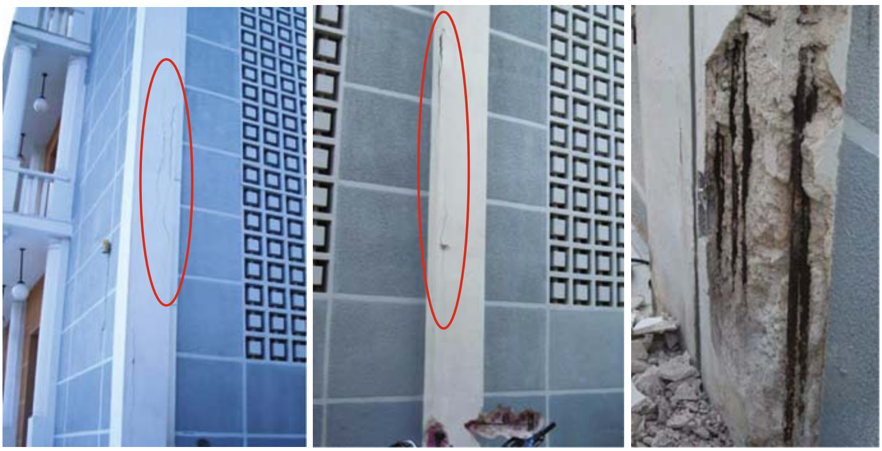
The qualitative agreement of damage may be considered as validation of the expressions in Sections 3.2.3.5 and 3.2.4.3 – adopted in Annex A of CEN (2005a) – for the flexure-controlled ultimate cyclic chord rotation and the degradation of shear resistance with cyclic loading.

6.10.2 Seismic Retrofitting of Theatre Building with RC and FRP Jackets and New Walls

The theatre facility presented already in Figs. 4.18 and 4.19 has been retrofitted to remedy the seismic deficiency pointed out in Section 4.10.5.3 as well as the extensive corrosion of the reinforcement in vertical members of the perimeter (Fig. 6.13). The design of the retrofitting (in mid-2005) was the first application of Part 3 of Eurocode 8 to a real building. The performance objective was to fulfil the Significant Damage Limit State (Life Safety performance level) for the 475 year earthquake, with a peak ground acceleration (PGA) of 0.36 g specified for ordinary buildings at the site in the current zonation map.

The seismic assessment of the building under a seismic action with a PGA of 0.10 g has been highlighted in Section 4.10.5.3. The design of the retrofitting and the evaluation of the retrofitted building through nonlinear dynamic seismic response analysis are presented here.

On the basis of the key features of the response of the unretrofitted building and of its assessment under the 0.10 g PGA motions, the following retrofitting measures were chosen (Fig. 6.14):



(a) (b) (c)

Fig. 6.13 Vertical cracks in perimeter members of theatre building (**a**), (**b**), due to reinforcement corrosion (**c**) (See also Colour Plate 20 on page 732)

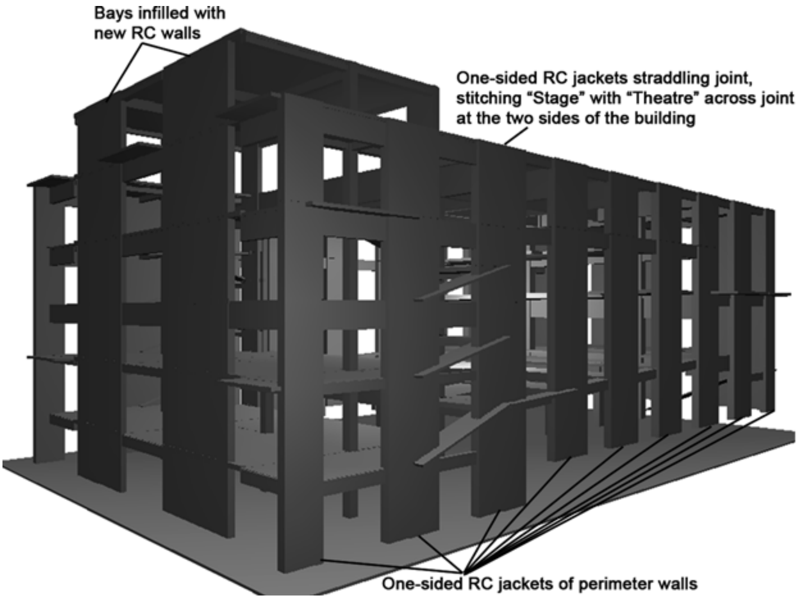


Fig. 6.14 Schematic of RC jackets and new walls in theatre building

1. The two parts of the building across the (practically zero width) expansion joint are connected into an integral structure with planwise balanced stiffness and strength, to avoid twisting of the individual parts and pounding at the joint. This is achieved as follows:

   (i) Each one of the two perimeter walls at the corners of the right-hand-side of the Stage part in Fig. 4.18 (left) is integrated with its counterpart across the expansion joint at the corners of the left-hand-side of the Theatre part in Fig. 4.18 (right), into a single rectangular wall straddling the expansion joint. Pairs of such walls are shown in Fig. 4.18 within continuous-line rectangles. One pair appears also at the right-hand-side of Fig. 6.1(g). The connection is effected through a shotcrete overlay on the exterior face of the two walls, with 18 mm-dia. horizontal bars at 100 mm centres continuous across the joint. Shear connection across the joint is enhanced by batten plates welded to the corner vertical bars of (the barbells of) the individual walls next to the joint.

   (ii) The roof diaphragms of the Stage and the Theatre parts are connected across the expansion joint all along their common length at the roof (i.e., outside the elevated central part – penthouse – of the Stage). This is done with a 0.3 m-deep and 0.8 m-wide RC belt, symmetrically cast over the joint and fastened to the roof slab via epoxy-grouted dowels (Fig. 6.15(a)). It has 14 mm-dia. closed ties across the joint at 100 mm centres and 16 mm bars along it.

   (iii) Each interior wall parallel to the joint at the right-hand-side of the Stage part in Fig. 4.18 (left) is connected to its counterpart across the expansion

Fig. 6.15 Features of the retrofitting of the theatre building: (**a**) RC belt straddling joint at roof level; (**b**) steel rods connecting interior walls across the joint; (**c**) long side of the building with finished wall jackets; (**d**) new bay-long walls at the back

joint at the left-hand-side of the Theatre part in Fig. 4.18 (right). Each pair of such walls is shown in Fig. 4.18 within a dashed-line rectangle. For the connection, two rows of 24 mm-dia. horizontal steel rods at 250 mm centres vertically (Fig. 6.15(b)) are placed in holes drilled through the short direction of the two walls and secured by anchor plates at their accessible vertical faces (i.e., opposite to the faces on the expansion joint). Although the rods act also as dowels across the joint, their main role is to transfer shear forces from one wall of the pair to the other. They have been dimensioned to cover a shear strength shortfall in one of the walls by the surplus in the other, both as obtained from the nonlinear seismic response analyses described later on.

2. The ten exterior shear walls on each one of the two long sides of the integrated building are strengthened and stiffened from the foundation to the roof with an exterior 150 mm-thick shotcrete overlay (see Fig. 6.1(g)). Note that reinforcement corrosion was limited to the perimeter walls or columns. Before applying the overlay, corroded bars were exposed, cleaned from rust and epoxy-coated against future corrosion. The original walls are 250 mm-wide. The ones at the

corners incorporate a 400 mm square column at one end of the cross-section (like a barbell, see Fig. 6.13(a)), while the intermediate ones have the 400 mm square column at mid-length (Fig. 6.13(b)). With the overlay the perimeter walls are converted into 400 mm-wide rectangular ones (compare Figs. 6.13(a) and (b) with 6.15(c)). The overlay is connected to the old wall via epoxy-grouted dowels. It contains one curtain of vertical and horizontal bars, with sufficient cover for corrosion protection in the mid-term and in sufficient quantity for the necessary flexural and shear resistance of the new, integral wall, even after the – at certain points complete – loss of cross-sectional area in the existing horizontal bars due to corrosion. The new vertical bars are epoxy-anchored at the top of underground perimeter walls or of bulky foundation beams. This is easy, as the top of these foundation elements is at about grade level and their exterior face is flush with the added concrete overlay (Figs. 6.1(g) and 6.15(c)).

3. Two new 500 mm-wide concrete walls are added, by infilling the 2nd and the 4th bay of the frame on the left-hand-side of Fig. 4.18, encapsulating existing columns and beams (see dashed-line ovals in Fig. 4.18). The main goal of these walls is to counterbalance the two over-one-bay-long walls at the opposite side of the integrated building. Another purpose is to strengthen the elevated central part – penthouse – of the Stage, to the top of which they continue. The finished walls are shown in Fig. 6.15(d).

Figure 6.16 depicts the average flexural damage ratio in the vertical members of the so-retrofitted building from the analyses under 56 spectrum-compatible bidirectional ground motions with a PGA of 0.36 g. The damage ratio is the maximum ratio of chord rotation demand to the concurrent capacity (as affected by the fluctuating axial load and shear span ratio) at the Near Collapse Limit State according to Part 3 of Eurocode 8. The demand-capacity ratios at the Significant Damage (Life Safety) Limit State may be obtained by multiplying by 4/3 the values in Fig. 6.16. The combination of the retrofitting measures highlighted above is sufficient for essentially all elements at the Significant Damage Limit State. The alarming damage ratios in four columns of the penthouse[13] (especially after the values Fig. 6.16 are multiplied by 4/3) are not a source of concern, because, unlike almost everywhere else in the building, all bays around the penthouse are infilled with strong masonry walls without openings (but a narrow door), which were omitted in the analysis model. The contribution of these infills to the lateral stiffness and resistance of the penthouse is more than enough to prevent the failure of the four columns predicted in Fig. 6.16.

As shown in Fig. 6.17, the measures described so far cannot prevent shear failure of the two large walls of the façade (see Fig. 6.18, one of these walls featuring also in Fig. 6.13(a) – left-hand side) or of the interior walls on either side of the expansion joint (those connected together in the transverse direction through steel

[13]These four columns are all at the three sides of the penthouse which are not retrofitted. The two new walls reaching the top of the penthouse are on the 4th side.

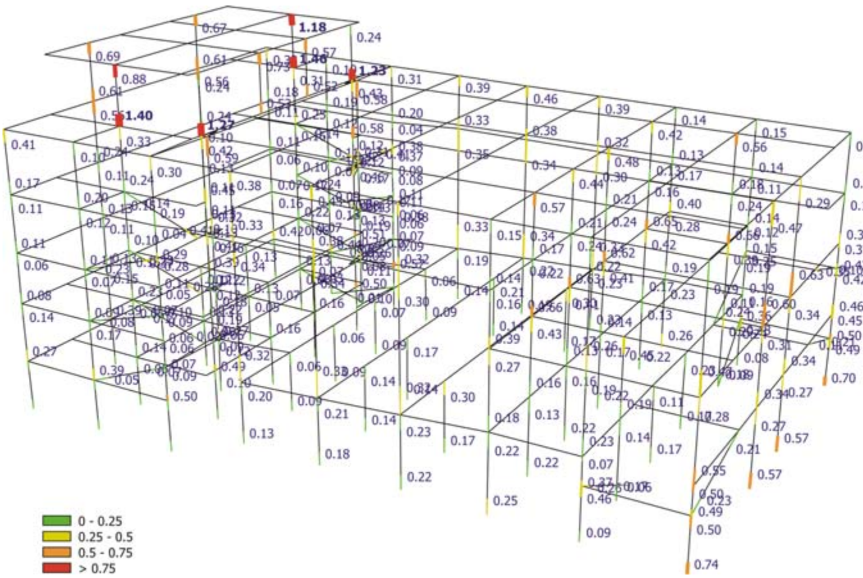


Fig. 6.16 Mean chord rotation demand in vertical members of the retrofitted building from seismic response analyses for 56 bidirectional ground motions at PGA = 0.36 g, divided by the corresponding chord rotation capacity for the Near Collapse Limit State (Kosmopoulos et al. 2007) (See also Colour Plate 21 on page 733)

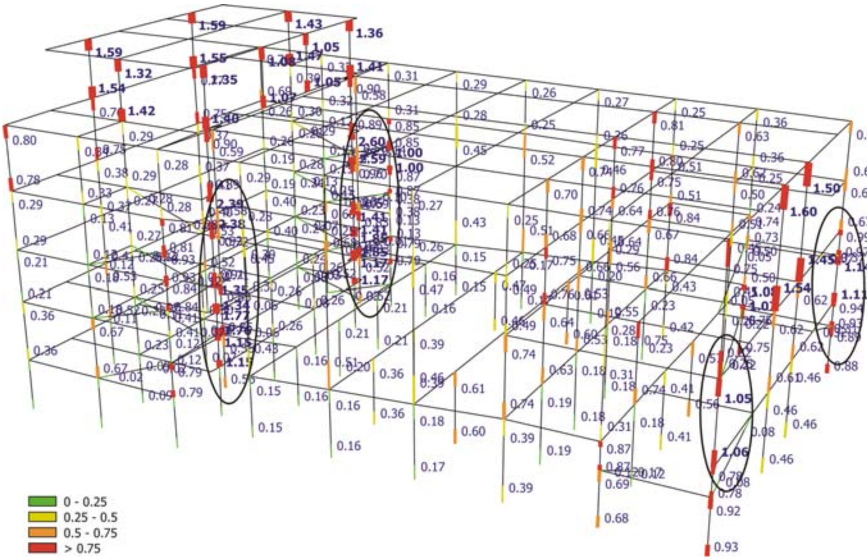


Fig. 6.17 Mean damage ratio in shear in the vertical members of the retrofitted building without the FRPs for 56 bidirectional ground motions at PGA = 0.36 g: shear force demand from the analysis divided by the corresponding capacity for any Limit State (Kosmopoulos et al. 2007) (See also Colour Plate 22 on page 734)

Fig. 6.18 Façade of theatre building with the two walls chosen for strengthening in shear with FRP

rods, see point 1(iii) above and Fig. 6.15(b)). It is not feasible to remedy these shear deficiencies via RC jackets or overlays, like in strengthening measure 1 above, because:

- Adding RC jackets or overlays to the façade is not architecturally acceptable.
- The owner does not welcome the inconvenience and debris produced by casting or shotcreting jackets around the interior walls on either side of the expansion joint.
- It is not feasible to access the foundation at these points in plan and at the façade, in order to connect the RC jacket or overlay and transfer its seismic moments and shears to the ground, without a prohibitively intrusive and costly operation (imagine in Fig. 6.18 the disruption entailed by exposure of the foundation of the walls of the façade);

The shear deficiency of the two large walls of the façade and of the two pairs of interior walls on either side of the expansion joint has been remedied by bonding horizontal Carbon FRP (CFRP) sheets:

- to the exterior face of the two large façade walls, see Fig. 6.19(a), and
- to the surface of the accessible long sides of the two pairs of interior walls (the sides opposite to the ones on the expansion joint), as shown in Fig. 6.19(b).

The six CFRP-retrofitted walls are shown in Fig. 4.18 inside continuous-line ovals.

The total thickness of CFRPs (equivalently, the number of CFRP plies of standard thickness) is dimensioned from the mean deficit of shear strength in the corresponding wall from the seismic response analyses for the 56 bidirectional ground motions with a PGA of 0.36 g, i.e., so that the shear damage ratios in Fig. 6.17 are reduced below 1.0.

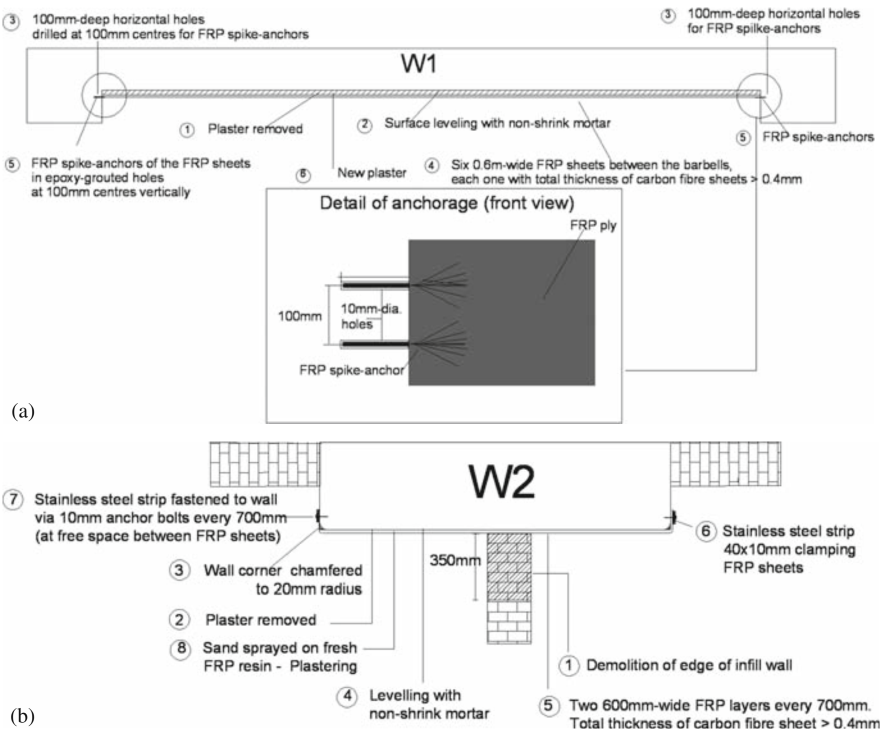


Fig. 6.19 One-sided shear strengthening with surface bonded horizontal CFRP sheets: (**a**) 3.5 m-long part of façade wall; (**b**) 1.6 m-long interior walls next to seismic joint (Kosmopoulos et al. 2007)

Numbers in circles in Fig. 6.19 show the sequence of operations for the application of the FRPs.

Witness in Fig. 6.19(a) the anchorage of the edge of the one-sided FRP sheets at the re-entrant corners of the web of the façade wall with the column-like barbell protruding from it, through 200 mm-long spike FRP anchors placed in 100 mm-deep holes filled with epoxy. The 100 mm-long part of the anchor outside the hole fans out within the epoxy layer between two successive FRP plies, to collect the forces of the FRP sheets and transfer them to the concrete.

Witness also in Fig. 6.19(b) the clamping of the edge of the FRP sheets applied on one side of the four interior walls. The edge is clamped by 40 mm-wide, 10 mm-thick stainless steel straps placed vertically next to the rounded corner of the section and fastened to the short side of the wall via anchor bolts driven into the concrete at the free space between adjacent horizontal FRP sheets.

Because the surface-bonded FRPs applied here for shear strengthening do not affect the stiffness or moment resistance of members, we do not need repeating the (nonlinear) analysis to evaluate their effect on performance. The FRPs can always be dimensioned after the analysis, for the shear strength deficit identified from it.

It is not considered worth remedying the shear deficiency shown in Fig. 6.17 at the two central columns of the façade at the top storey. This could be accomplished by wrapping the entire height of these columns with FRP, dimensioned to resist the deficit in shear strength of the column. That would entail demolishing vertical strips of the brick masonry wall on both sides of these columns (see Fig. 6.18) and restoring these walls after FRP wrapping. The contribution of these masonry piers to the in-plane stiffness and resistance of the façade is sufficient for the protection of these columns from an overstress given by an analysis that neglects the positive contribution of these wall piers.