On the Eurocode 8 limited damage criteria for non-structural elements – Analysis and requirements

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SUMMARY:
Eurocode 8 Part 1 provides criteria on the story drift ratio to limit the damage due to frequent seismic event. Although the regulation is simple, its practical applicability is impeded in lack of standard definition for classification of non-structural elements.
The paper focuses on the behaviour and damage limitation of deformation sensitive non-structural elements: gypsum wall, infill masonry. Based on cyclic test results from the literature, hysteretic behaviour of the elements is characterized and major parameters are determined. The elements are classified with respect to ductility, deformation sensitivity and degree of damage. It is concluded from the results that type and degree of damage is not correctly reflected by the ductility classification. Based on the results, the required repair work and expenses, different damage levels are defined for each element. Analysis and damage evaluation approaches are finally illustrated through numerical example of a multi-storey hotel building.

Keywords: non-structural elements, damage limitation, expected damage, drift ratio, ductility.

1. INTRODUCTION

In the recent period, design to limit damages cause by seismic events as well as research on economical restoration procedures attracted more attention due to series of powerful earthquakes in the seismic regions. One of the fundamental requirements of the European standard for seismic design, Eurocode 8 is the damage limitation requirement reflected in terms of interstorey drift limitation due to a frequent seismic event (95-year return period). By fulfilling the limitation the amount of damage can be kept on a low level and after a seismic event the building can keep its original function.

In the Eurocode 8 Part 1, EC8-1 (EN1998-1) the limitation of interstorey drift depends on the behaviour of the non-structural elements of the building. The non-structural elements have to be categorized as brittle, ductile or isolated elements from the structure (in a way it does not interfere with the structural deformations). The brittle or ductile behaviour means small or large deformation capacity. Practical problem of the regulation is that the code does not provide any support for practicing engineer on what basis can a non-structural element be categorized as brittle or ductile. Further shortcoming is that such classification neglects the fact that ductile behaviour – where excessive deformations are experienced in the non-structural elements – does not necessarily ensure economical repair of the damaged parts; consequently it does not represent the amount of probable damages and the repair costs. Furthermore, differences can be found in the displacement analysis of the structures when comparing EC8-1 to international codes, indicating that Eurocode is strict with many aspects.

The final goal of our research is to answer the arising question on the limited damage criteria, with special respect to the classification of non-structural elements, the acceptable drift values and to the displacement analysis. This paper first illustrates and compares the current Eurocode and international regulations. Secondly, based on experimental results available in the literature, seismic performance evaluation of various non-structural elements is completed and proposal is given for the acceptable damage levels and corresponding drift ratios. An illustrative example of 7-storey building calls the attention on the importance and differences experienced with different displacement analysis and damage evaluation methods.

2. CURRENT EUROCODE REGULATION

EC 8-1 prescribes two fundamental requirements depending on the probability of occurrence: a) no-collapse requirement corresponding to the ultimate limit state and b) damage limitation requirement corresponding to the serviceability limit state requirement.

The ultimate limit state requirement includes, that the primary objective is the protection of human life during a rare seismic event through the prevention of local or global building collapse. According to EC 8-1 an earthquake with 10% probability of exceedance in 50 years (475-year mean return period) can be considered as a rare seismic event. While such a seismic event significant damage and moderate permanent deformation may occur in the building, but the structure must preserve the capacity and posses sufficient stiffness as well as strength to withstand further aftershocks. The repair costs can go beyond an economical level. In the sense of the second requirement, the damage limitation requirement, no permanent deformation can occur on the structure nor on any of its elements and no significant stiffness and strength reduction can be a result of a frequent seismic event. According to EC 8-1 an earthquake with 10% probability of exceedance in 10 years (95-year mean return period) can be considered as a frequent seismic event. During such an earthquake the non-structural elements can be damaged, but these can be easily and economically restored. The fundamental requirement can be considered fulfilled if the limitations listed in Table 2.1 apply for the maximum calculated interstory drifts. The parameters in the table are: h is the storey height; d_{r} stands for the design interstorey drift calculated for the 475-year seismic event; ν is the reduction factor taking into account the lower return period of the seismic action associated with the damage limitation requirement (in case of importance class I and II ν = 0,5 and in case of importance class III and IV ν = 0,4). In different standards the design interstorey drift is typically indicated with the drift ratio that is the quotient of the displacement difference and the storey height: d_r / h (%).

Table 2.1. Damage limitation criteria in EC 8-1

	Limitation
Non-structural elements of brittle materials attached to the structure	$d_r \cdot \nu \leq 0{,}005 \cdot h$
Ductile non-structural elements	$d_r \cdot \nu \leq 0{,}0075 \cdot h$
Non-structural elements fixed in a way so as not to interfere with structural deformations	$d_r \cdot \nu \leq 0{,}010 \cdot h$

3. INTERNATIONAL SPECIFICATIONS

In the following, drift limitations criteria of various international specifications are compared. Documents involved in the study are: Uniform Building Code (UBC, 1997), NEHRP (NEHRP, 2001), ASCE 07-10 (ASCE 07-10), ANSI/AISC 341-05 (ANSI, 2005), FEMA-445 (FEMA-445, 2006), FEMA-356 (FEMA-356, 2000). Table 3.1 compares the various international specifications.

The first observation is that even if the same probability of occurrence is considered in the displacement analysis, the calculated displacements are different when using different codes. Note that while in Eurocode the displacement behavior factor q_{d} (similar to the displacement modification factor C_{d}) in most cases equals to the behaviour factor q (similar to the response modification factor R), American standards usually prescribe lower value for C_{d}. Accordingly, the deformation level that shall be taken into account for the damage criteria calculation is higher in case of the European standard. This may be interpreted as the different codes require different probability of exceedance for the damage analysis.

Table 3.1. Comparison of drift criteria of international specifications

Code	Design force	Drift	Modified drift	Limitation of interstorey drift
EC 8-1	$a_g \cdot S \cdot \frac{2{,}5}{\boldsymbol{q}} \cdot m$	d	$d \cdot \boldsymbol{q_d} = d_r$	$d_r \leq \dfrac{0{,}005 \div 0{,}010 \cdot h}{\nu}$
UBC	$\dfrac{C_v \cdot I}{\boldsymbol{R} \cdot T} \cdot W$	Δ_S	$\Delta_S \cdot \boldsymbol{R} \cdot 0{,}7 = \Delta_M$	$\Delta \leq 0{,}020 \div 0{,}025 \cdot h_{sx}$
ASCE, NEHRP	$\dfrac{S_{DS}}{\boldsymbol{R}} \cdot I \cdot W$	δ_{xe}	$\dfrac{\boldsymbol{C_d} \cdot \delta_{xe}}{I} = \delta_x$	$\Delta \leq 0{,}007 \div 0{,}025 \cdot h_{sx}$

where: γ_{I} (EC 8-1) or I (UBC, SCE, NEHRP) is the importance factor; q or R is the behaviour factors or response modification factor, respectively; q_{d} and C_{d} are the displacement behaviour factor and displacement modification factors, respectively; W is the seismic load; a_{g} is the design ground acceleration; m is the building weight; S and S_{DS} are the response spectrum parameters; C_{v} is the seismic coefficient (according to the ground type and the seismic zone); h, h_{sx} is the interstorey height.

To understand this difference, one may review the generalized classification of Performance Based Design (PBD) methodology.
In PBD, the building performance levels are in accordance with the expected damage. The performance levels typically applied are: *Immediate Occupancy (IO)*, *Life Safety (LS)*, *Collapse Prevention (CP)* and *Not Considered (NC)*. At *Basic Safety Objective* (BSO) that is similar to the Eurocode objective, two requirements have to be fulfilled: *LS* – an earthquake with 10% probability of exceedance in 50 years (475-year mean return period), *CP* – an earthquake with 2% probability of exceedance in 50 years (2475-year mean return period). *IO* performance level, which is analog to EC 8-1 damage limitation requirement only have to be satisfied at *Enhanced Rehabilitation Objective*. In case of *IO* the general requirements are: no permanent deformations and no stiffness and strength degradation shall be experienced. However, while EC8-1 requires the analysis for an earthquake with 10% probability of exceedance in 10 years (95-year mean return period), the *IO* level of PBD shall be usually checked with 50% probability of exceedance in 50 years (72-year mean return period). The wider design spectrum which is used in PBD can be taken into consideration with reliability differentiation implemented by classifying structures into different importance classes in EC 8-1. For every importance class EC8-1 adjusts the so-called importance factor γ_{I}. The different levels of reliability are obtained by multiplying the reference seismic action by this importance factor. The importance factor γ_{I} = 1,0 is assigned to an earthquake which has a reference return period T_{NCR}. This is equal with a reference probability of exceedance P_{NCR} in 50 years. The importance factor γ_{I} can be calculated as $\gamma_{\mathrm{I}} \sim (T_{\mathrm{LR}}/T_{\mathrm{L}})^{-1/k}$, where the same probability of exceedance in T_{L} years as in the reference T_{LR} years (for which the reference seismic action is defined); the value of the exponent k depending on seismicity, but being generally of the order of 3. Fig.. 3.1 illustrates that at importance class I, buildings of minor importance for public safety (e.g. agricultural buildings) the damage limitation requirement should be satisfied for 49-years mean return period (γ_I = 0,8), while at importance class II, ordinary buildings for 95-years mean return period belonging to γ_I = 1,0. The 72-years mean return period in PBD can be corresponded to importance factor γ_I = 0,91.

Further difference is that PBD does not categorize the non-structural elements as brittle or ductile; the drift limitation is determined according to the types of elements individually. It sets different limits for exterior veneers, heavy or light participations, interior veneers, suspended ceilings. The Uniform Building Code (UBC) against the EC 8-1 damage limitation requirement has only two categories which depends on the vibration period T and does not depends on the non-structural elements behaviour or the importance classes. The National Earthquake Hazards Reduction Program (NEHRP) has only three importance classes: minor importance class does not exist. The allowable storey drift in NEHRP depends on the building structures type (e.g. masonry cantilever shear walls, other masonry shear walls, structures other than masonry walls). The American Society of Civil Engineers (ASCE) analysis is similar to NEHRPs with little differences, but none of them depends on non-structural elements, or its behaviour.

After all, due to the intensity of the seismic event normally considered and with respect that the drift limitation criteria differ, it can be stated that EC 8-1 is found stringent in comparison to international specifications.

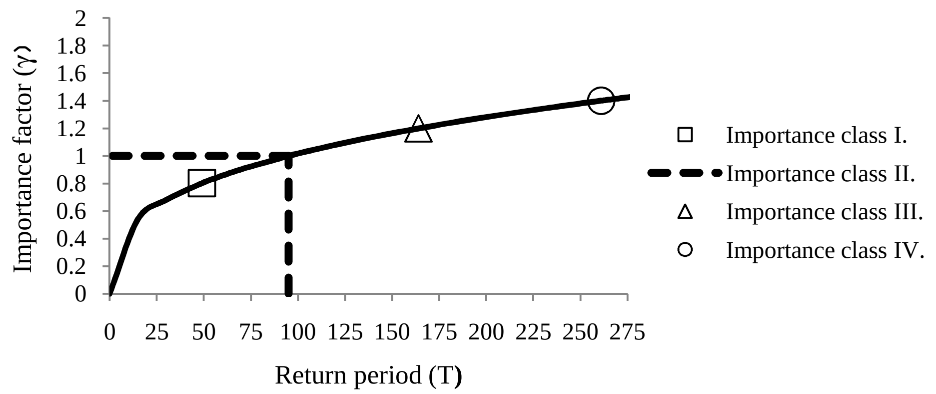

Figure 3.1. Return periods of different importance factors

4. PERFORMANCE EVALUATION OF NON-STRUCTURAL ELEMENTS

4.1. Non-structural elements and the evaluation procedure

In the current phase of the research, test results for masonry infills, drywall participations and anchored brick veneers are collected from available literature. On a common basis, the results are systematized and used for the seismic performance evaluation as well as damage level qualification. New limits can be defined according to the actual behaviour of the elements on the basis of the rate of the damage and the corresponding drift ratio.

Numerous studies and tests were explored with monotonic and cyclic loading to evaluate the behaviour of these elements. During the experiments they recorded the seismic response of the drywall participations (NAHB 1997, SEAOSC 2001, Fülöp and Dubina 2004, Arnold *et al.* 2002, Landolfo *et al.* 2006, Bersofsky 2004, Lee *et al.* 2006, Moghimi and Ronagh 2009), masonry infills (Alcocer and Zepeda, Calvi *et al.* 2004, Braz-César *et al.* 2008, Baran and Sevil 2010, Yuksel *et al.* 2010) and anchored brick veneers (Klingner *et al.* 2010, Thurston and Beattie 2008, Jo 2010) for different structural constructions.

The recorded force vs. deformation curves – hysteresis diagrams – are appropriate to calculate the ductility of the tested specimens and thus the different non-structural elements can be categorized as brittle or ductile elements. The classification is completed in accordance with specifications of the FEMA-356 document, Fig. 4.1. The behaviour of the element under investigation is considered ductile if its hysteresis diagram is similar to curve of Type 1 or 2 in Fig. 4.1 and the strain hardening range (1-2) is longer than the elastic range (0-1) $e > 2g$. In all other cases, the behaviour is considered brittle.

The evaluation accordingly requires the multi-linear idelaization of the nonlinear response. For the present discussion, the authors applied bilinear idelaization (elastic-perfectly plastic model, based on Frumento et al., 2009) as illustrated in Fig.4.2. The elastic stiffness k_{el} is obtained from the 70 % load value of the maximum strength of the experimental envelope (i.e. 70% of F_{max}) and the corresponding actual deformation. The ultimate displacement d_{u} can be evaluated as the displacement corresponding to strength degradation equal to 20% of F_{max}. The ultimate limit strength is determined so that the “dissipated” energy in the actual and in the idealized system is equal. The ultimate ductility is defined as the quotient of the ultimate displacement d_u and the yield displacement d_y.

The recorded damage history allows the damage level quantification and can be evaluated with respect to economy of possible rehabilitation methods.

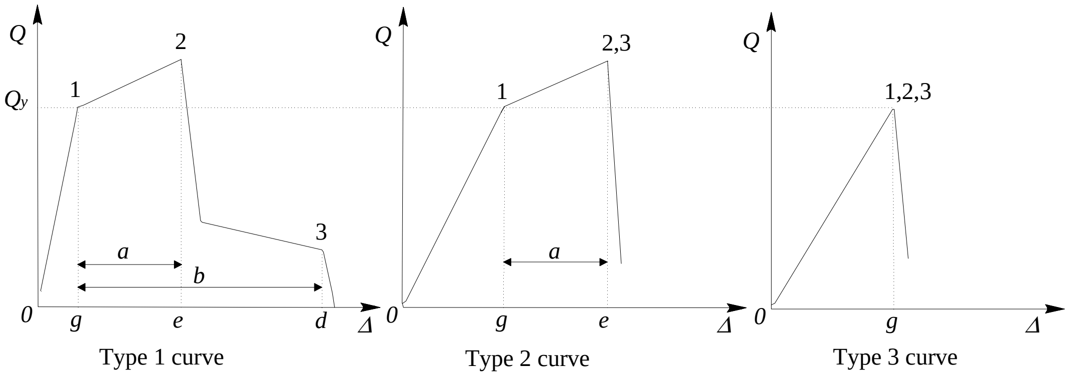

Figure 4.1. Component force versus deformation curves, FEMA-356

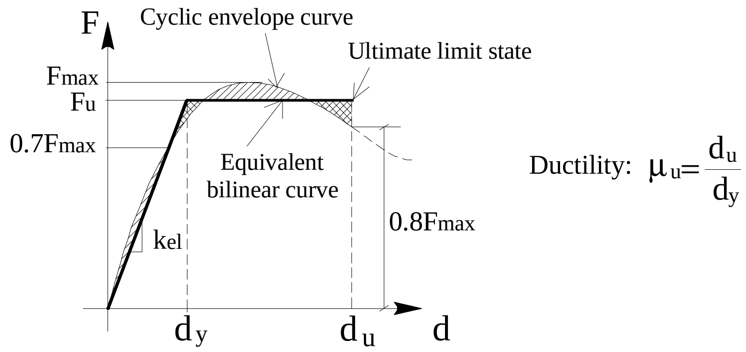

Figure 4.2. Hysteresis envelope and its bilinear idealization, Frumento *et al.* (2009)

4.2. Analysis of Gypsum Board Walls

Bersofsky et al. (2004), Lee *et al.* (2006), Restrepo et al. (2010), Moghimi and Ronagh (2009) and the *Structural Engineers Association of Southern California* (SEAOSC) (2001) completed test programme for drywall participations as non-structural elements. The NAHB (*National Association of Home Builders)* (1997), Fülöp and Dubina (2004), Arnold *et al*. (2002), Landolfo *et al.* (2006) tested the gypsum board walls as load-bearing elements, not as non-structural elements. Bersofsky analysed eight different specimens, with different srew spacings, gypsum board thicknesses, metal stud gages and stud spacings. The SEAOSC tested thirty six kind of wall constructions, most of them were made with OSB covering. Moghimi and Ronagh tests focused on drywalls with strap bracing. In the experiments of Lee *et al.* two specimens were tested with and without an opening.

As representative examples, for the same gypsum thickness covering, Fig. 4.3 a) and b) illustrate results from SEAOSC (2001) and Bersofsky (2004), respectively. In the hysteretic curve, linear elastic range is followed by plastic strain hardening. Reaching the capping point, significant in-cycle strength degradation can be observed. Stiffness degradation appears in both loading and unloading part. Severe stiffness degradation can be experienced during reloading after unloading, along with stiffness recovery when displacement is imposed in the opposite direction (pinching behaviour). Idealized model for SEAOSC specimen is shown in Fig. 4.4; the calculated ductility is μ_u= 3,08; 3,33. Typical response of the drywall participations can be described as Type 1 behaviour, thus it can be considered as ductile elements.

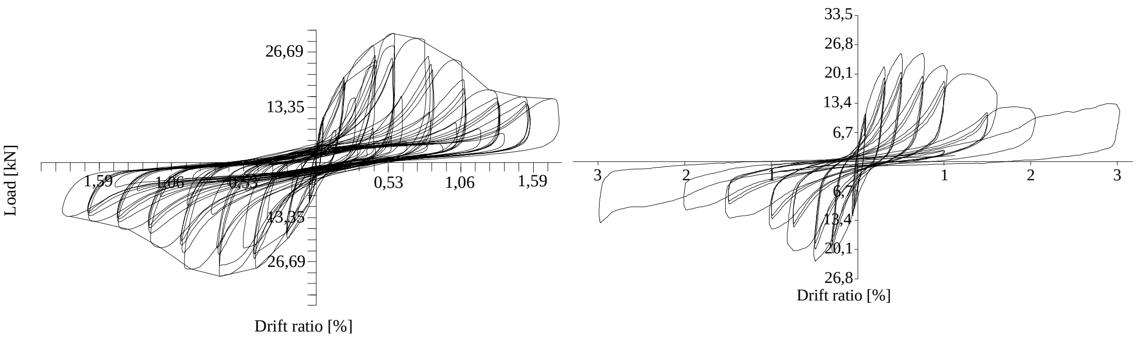

Figure 4.3. Lateral force–drift ratio hysteretic response: a) SEAOSC (2001); b) Bersofsky (2004)

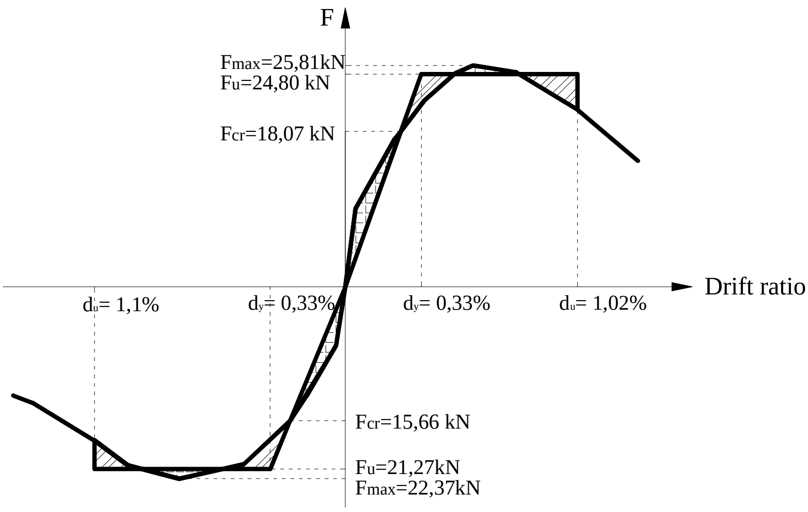

Figure 4.4. Hysteresis envelope of SEAOSC and its bilinear idealization

For the assessment of damage of drywall participations Taghavi and Miranda (2003) defined three damage levels reflecting the feasibility of the required repair works: slight, moderate and serious damage levels. At the first level (Damage State 1), minor damages, crackings and screw head poppings can be repaired with tape, paste and paint. At moderate damages (Damage State 2), it may be necessary to change the gypsum boards because of the crushed panels, but no damage can be detected on steel studs. At serious damages (Damage State 3), partial or total replacement may be needed because of the drywall spalling, steel stud buckling. Occurrence of the third level can be difficult to determine because of the gypsum board covering. Bersofsky defined the Damage State 1 with tape wrinkling at 0,3% drift ratio. The Damage State 3 was reached at 1,5% drift ratio with drywall spalling. However in the test of SEAOSC wood studs were used, the response were very similar.

Based on the gathered test data, drywall participations can be categorized as ductile and deformation sensitive elements. The concluded Damage Levels and the corresponding drift ratio limitations are summarized in Table 4.1. based on the observed damages and the strength degradation on the force – drift ratio hysteresis curves. It is suggested that damages beyond Damage Level 1 are not allowable neither on the level of damage limitation in EC 8-1 nor on Immediate Occupancy performance level in PBD. Significant strength and stiffness cyclic degradation can occur when Damage Level 2 is reached, and major injuries on the gypsum coverings which can prevent the immediate occupancy and the economic and rapid repairmen can be experienced.

Table 4.1. Suggested damage levels and corresponding drift limits – Drywall participations

Level	Drift ratio	Expectable damages
Level 0	0-0,5%	Tape uplifting, drywall cracking, screw head popping
Level 1	0,5-1%	Drywall buckling and crushing, increasing of damages reached at Level 0
Level 2	1%-	Drywall spalling, steel stud buckling

4.3. Evaluation of masonry infills

Calvi *et al.* (2004) completed experiments for analyzing the benefits of slight reinforcements of masonry infills. One goal of their research is to evaluate the positive effects of the mortar layer reinforcements to serviceability and of the repair costs. Braz – César *et al.* (2008) analysed frames with or without masonry infills. Their scope was to compare the results of the experiments and the numerical model. In the program of Baran and Sevil (2010) they examined the behaviour of one- and two-storey masonry infilled frames. Altogether they had nine versions where they varied strength of the concrete, axial load of the column, load spectrums etc. Yuksel *et al.* (2010) focused on walls with carbon fiber reinforced polymers (CFRP).

The lateral capacity of masonry infills depends on the frame wall interaction. When horizontal loads are relatively low and the connection between the wall and the frame is rigid, the infills significantly increase the global stiffness of the frame. When increasing the load, due to the lack of tensile strength of the wall, the frame and the infill may separate. The further failure mechanism is governed by the relative strength and stiffness of the frame and the wall. The frame as a primary structural element needs to transfer the vertical loads, and because of its damage can be excluded with complying requirements it is sufficient to observe only the masonry infills failures. The main difference in the hysteretic behaviour between the drywall participations and the masonry infills is the shortness or the lack of strain-hardening range whereas the masonry infills have great stiffness and strength before reaching the first crackings. After the maximum load capacity the cyclic strength and stiffness degradation can be the consequence of the diagonal crackings.

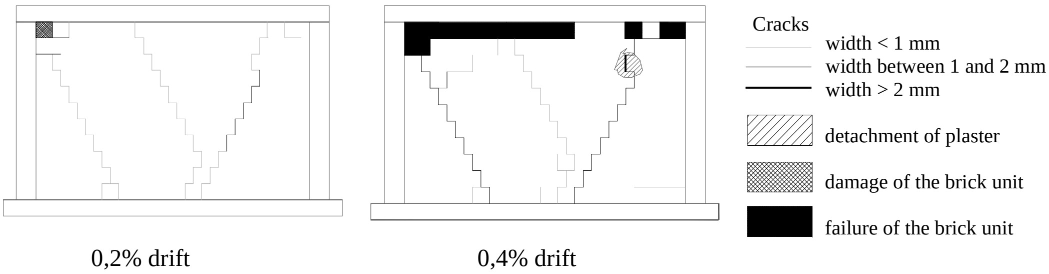

Figure 4.5. State of damage at 0.2 % and 0.4 % drift, Calvi *et al.* (2004)

In the experiments of Calvi *et al.*, as shown in Fig. 4.5, the diagonal crackings appeared at a drift ratio of 0,2%, followed by the damage and failure of the brick units at 0,4%; at this deformation level mortar at the top of the wall remained undamaged. Because of these failures, the restoration cannot be simply carried out with injection of the gaps and crackings; approximately 10% of the whole wall would be required to change.

As confirmed in Fig. 4.6, in the experiment of Braz – César *et al.* (2008) the highly stiff hybrid frame could withstand a fairly high load until the first cracks appeared in the wall at a drift ratio of 0,2-0,3%. Following this only a minor strength and stiffness reduction could be observed up to 0,81%. drift. Baran and Sevil (2010) used a lower concrete class. At their test, the ultimate capacity was reached at a drift ratio of 0,36%. In the case of Yuksel *et al.* (2010) the damages were only dominant in the frame and the infill cracked only at a high drift ratio. The first damages were tensional cracks which appeared at 0,22% in the frame column, the ultimate failure was the damage of the frame, too.

Concludingly, it can be stated that if masonry infill failure is predominates, the full depth diagonal cracking will occur at reaching the maximum load capacity and by the opening of the cracks the damage limitation requirement cannot be fulfilled.

Based on the bilinear idealization, it can be stated that the masonry infill walls are displacement-sensitive and ductile, just like the drywall participations, but with greater degree of damage. 80% strength degradation was reached at d_u =1,64%; 1,51% drift ratio, meaning $\mu_u =$ 4,21; 3,02 displacement ductility. On the basis of the experienced damages and the hysteretic responses, the damage levels listed in Table 4.2 are defined. It is emphasized that although the behaviour is classified as ductile, the expected damage is tolerable only on a relatively small deformation level. This observation is in line with the statement that element classification shall be completed individually for each element type and not necessarily related to the ductility of the element.

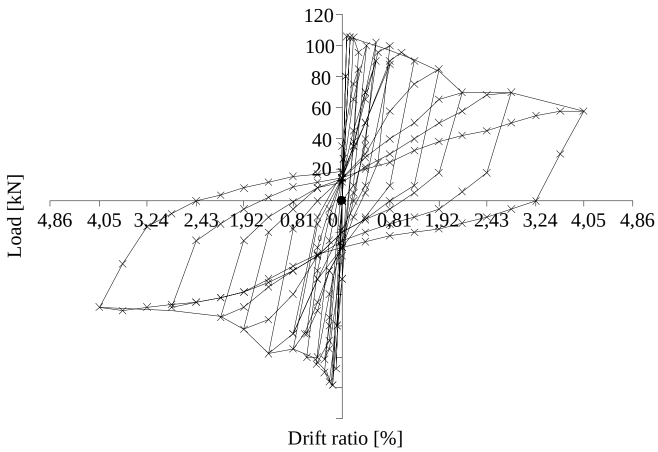

Figure 4.6. Lateral force – drift ratio hysteretic response, Braz – César *et al.* (2008)

Table 4.2. Suggested damage levels and corresponding drift limits – Masonry infills

Level	Drift ratio	Expectable damage
Level 0	0-0,3%	shearing of the mortar between wall and infill, first diagonal cracks
Level 1	0,3-0,55%	brick failures in the corners, which is less than 10% of the wall, more diagonal cracks appear,
Level 2	0,55%-	the increase of the damages occurred in Level 1

4.4. Summary of Results

The determined element classes and allowable drift limits are summarized in Table 4.3. As a result of the experiments both the gypsum board walls and the masonry infill walls are ductile and displacement-sensitive. The damages that can be experienced at the damage limitation requirement of EC 8-1 can be corresponded to PBD's IO performance level and to Damage State 1 of the experimental data. The evaluated limitation for gypsum board wall is in line with the current regulations of EC8-1 and PBD. In case of masonry infill walls, PBD provides acceptable values, but EC8-1 ductile classification would mislead in the damage limitation analysis, since the ductile behaviour comes with damages that cannot be economically repaired. It is expedient to set limitations for the types of non-structural elements individually according to PBD, rather than to the brittle-ductile behaviour class in damage limitation requirement set in EC 8-1.

Table 4.3. Non-structural element classification and recommended drift limitations

	Behaviour	Limit of PBD, IO	Limit of EC 8-1	Limit based on the amount of damage
Gypsum board walls	ductile	1%	0,75%	0,5-1%
Masonry infills	ductile	0,5%	0,75%	0,3-0,55%

5. COMPARISON OF ANALYSIS METHODS

Analysis and damage evaluation approaches are finally illustrated through numerical example of a multi-storey hotel building. The lateral load resisting system is dissipative eccentrical bracing. The study investigates the following aspects: a) global analysis type (modal analysis vs. pushover analysis); b) consideration of actual behaviour (influence of non-structural elements on the global structural performance); c) damage evaluation methods; d) cost calculation methods. The global numerical model is shown in Fig. 5.1. As an example, the actual gypsum wall behaviour is modelled with the ideal characteristics indicated in Fig. 5.1, on the basis of the processed experiment results. For further details refer to Soós (2009).

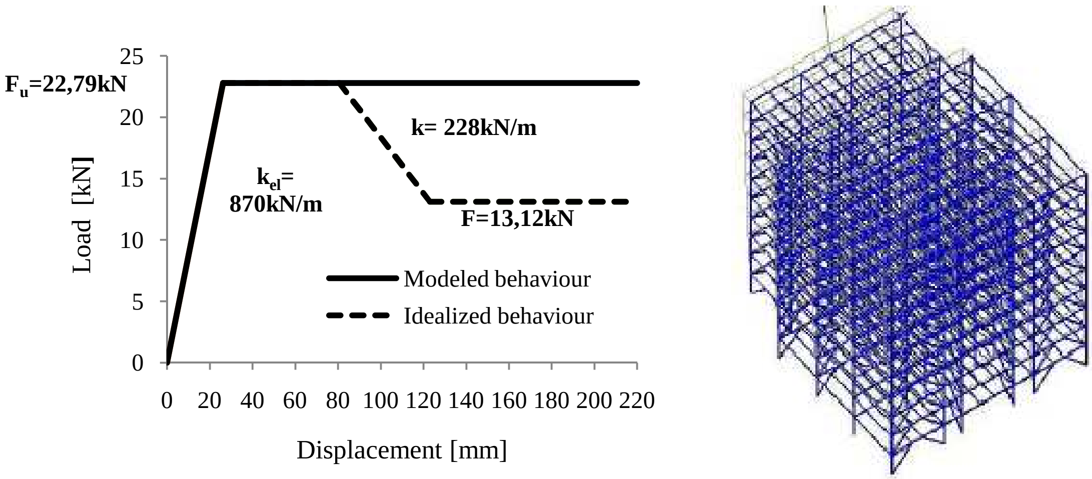

Figure 5.1. Modeled and idealized behaviour of gypsum board walls and the global modell, Soós (2009)

While existence of springs representing gypsum board walls slightly modified the overall behaviour, the effect of masonry infill is considerable in strength (20-25% increase) and stiffness. As expected, the largest interstorey drifts were induced by the modal response spectral analisys, followed by the push over analisys without walls, finally with modelled walls. From the storey heights and interstorey displacements the drift ratios can be calculated and the expected damages can be defined. Associating repair costs to the failure modes on the basis of the walls state the financial losses are estimated and summarized in Table 5.1.

Table 5.1. Financial costs from different analysis methods

	Gypsum board walls		Masonry infills	
	Maximmum displacement	Average storey displacement	Maximmum displacement	Average storey displacement
Modalanalysis	1053 USD	845 USD	12529 USD	7064 USD
Pushover analysis	757 USD	620 USD	4253 USD	2850 USD

The results confirm that advanced displacement analysis and damage evaluation leads to smaller expected damage costs: the evaluations based on the maximum storey drifts can overestimate the expected damages; pushover analysis results in much lower damages than the modal analisys. In the current example, the repair cost of drywall is 28,7% of the cost for masonry infill walls based on their actual behaviour, meaning that a much more favorable criterium can be set for gypsum board walls to have an expected repair cost similar to infill walls. The example detailed in Soós (2009) gives guidance for simplified as well as advanced procedure that allows designer to achieve more economical design.

6. CONCLUDING REMARKS

On the basis of experimental researches, the non-structural systems and their hysteresis behaviour can be analysed and the main behaviour parameters can be identified. The results of the evaluation showed that the categorization by ductility does not properly reflect the character and extent of the damages, therefore in verifying of the damage limitation in the Eurocode is misleading. The damage levels can be defined for the different non-structural elements based on the experimental results, these can be expressed in the rate of the relative displacement, what can be used directly to the examination of the damage limitation requirements. It is also illustrated that damage and the repair cost evaluation highly depends on the analysis type. Advanced analysis invoking performance based design methodology directly supports the proper decision between different alternatives.

ACKNOWLEDGEMENTS

The authors would like to thank civil engineers Imre Orbán and Tamás Nagy of Wienerberger, Inc., Hungary for technical information about masonry infills, and Imre Birkás of BB Immo Bulid, Inc., Hungary for technical information about gypsum board walls. The authors would also like to acknowledge the support and contributions from Anita Fódi and Endre Dulácska of Budapest University of Technology and Economics.

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