Bauforschung

Improved seismic design concepts for masonry buildings in Germany

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Report WP1: Objectives and basics of linear and nonlinear calculation methods

Report WP2: Selection and definition of representative masonry buildings

Report WP3: Parametric study

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DIBt-Project

Report WP1-RWTH

Objectives and basics of linear and nonlinear calculation methods

Improved seismic design concepts for masonry buildings in Germany

(Verbesserte seismische Nachweiskonzepte für Mauerwerksbauten in Deutschland)

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1 INTRODUCTION

The project aims to illustrate the available safety margins of modern unreinforced masonry buildings to make them accessible within the application of linear and nonlinear seismic design concepts consistent with the recent code requirements in Germany. The project results are generally useable and prepared for an application to all types of bricks (clay, calcium silicate, autoclaved aerated concrete, light-weight concrete) in German earthquake regions. The project is subdivided in two parts (Figure 1-1), which are described briefly in the following.

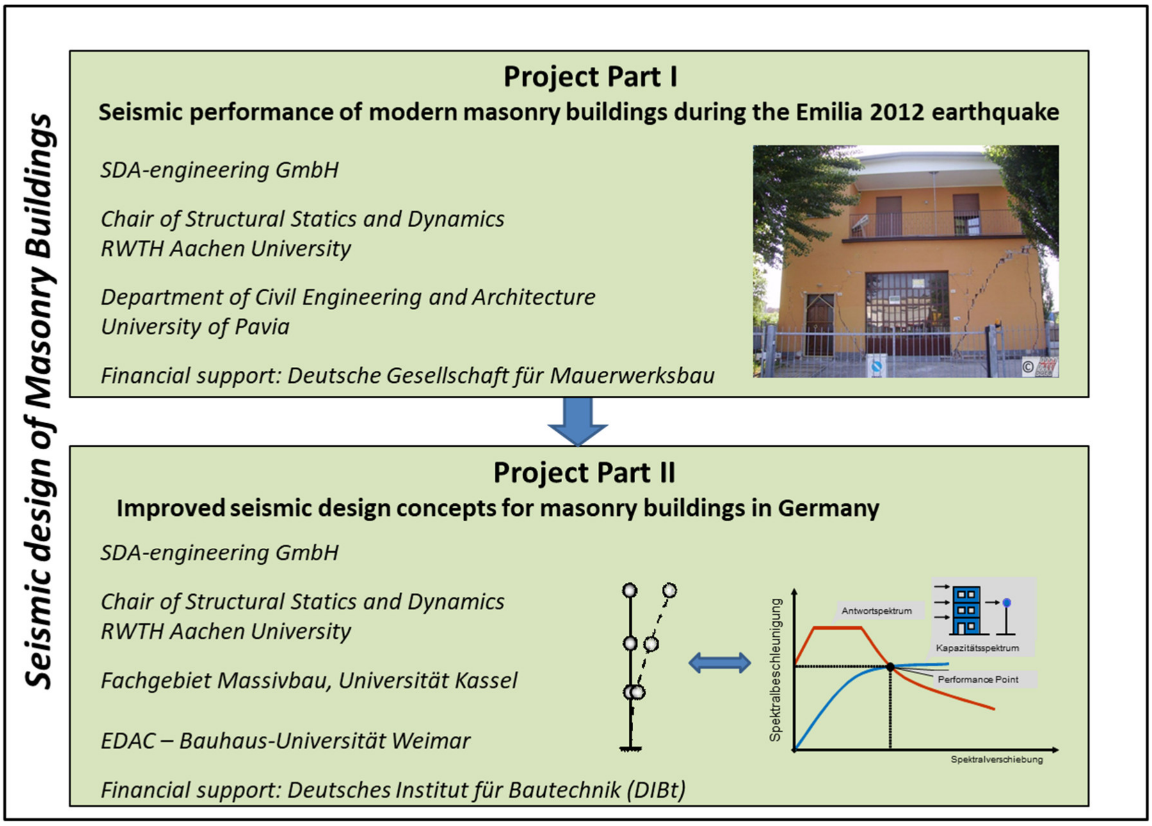


Figure 1-1 Research project: Seismic design of masonry buildings

1.1 Project part I (DGfM)

An earthquake with magnitude up to 5.9 hit the Emilia Romagna region of Italy on May 20, 2012, resulting in 7 deaths, significant damage to historic structures, churches and industrial buildings, and 7000 people needing shelter. This earthquake was followed by a second event on May 29, 2012 with a magnitude of 5.8 and further aftershock events. Although the maximum PGA values during the two earthquakes were quite high, the majority of modern ordinary residential and commercial masonry buildings have sustained either minor damage or structural damage that is deemed repairable.

A deeper investigation of this fact was the starting point of the first project part, financed by the DGfM (Deutsche Gesellschaft für Mauerwerks- und Wohnungsbau e.V.). Three modern masonry buildings with different damage grades are selected in the Emilia Romagna region. These buildings are investigated in detail, to evaluate the safety margins beyond the linear and nonlinear design concepts. For the three buildings complete building data, detailed damage descriptions and all required information on the installed materials and their statistical properties are collected and summarized in the report of WP1 [10]. In addition, the seismic actions at each building location are determined within the framework of detailed seismic hazard assessment and summarized in the report of WP2 [11]. The knowledge and quality of the seismic actions at the building sites are particularly excellent for the seismic events on May

29, 2012, as additional temporary seismic stations were installed after the first earthquake event close to the regarded buildings.

At first the three buildings are analysed by means of linear and nonlinear calculations and design procedures using the code spectra at each building location. The results are summarized in the reports WP3a-1 [30] and WP3a-2 [13]. The linear verification results show a remarkable gap between the design PGA associated to a return period of 475 years and the calculated maximum attainable PGA values. Overall, a successful linear verification is possible just for 1.1% to 6.5% of the design PGA according to the Italian code NTC2008 [9]. The results show clearly the deficiencies of the existing linear verification concept and the need of improvements (e.g. force redistribution) to get more realistic design results. The nonlinear calculation results are much more realistic, but depending on the chosen level of modelling (cantilever, equivalent frame) there is still a gap to the design PGA prescribed by the code.

The reports WP 3b-1 [12] and WP3b-2 [14] of the DGfM-project include the results of probabilistic analyses of the three investigated buildings and the verification results of linear and nonlinear calculations and design procedures using site specific hazards at each building location. As an average over all buildings just 4.6% of the estimated PGAs at the building sites can be applied to get a successful safety verification. If pushover analysis with a simplified beam model is applied, an average of 56% of the estimated PGA at the building sites can be verified. The results are in line with the verifications based on code spectra, summarized in the reports WP3a-1 [10] and WP3a-2 [14].

In the last part of the project, fragility curves are derived by means of probabilistic analyses for each of the three buildings. In report WP3b-1 [12] the fragility curves are derived based on nonlinear time-history analyses using an equivalent frame model. In report WP3b-2 [14] the fragility curves are calculated based on nonlinear pushover analyses using a simplified beam model and four different methods of verification (damped spectra; Inelastic spectra: N2-method, Modified N2-method [31], Guerrini [22]). The fragility curves of the two reports based on the different calculation approaches show a satisfactory agreement. In the next step, the fragility curves of each building are convolved with the seismic hazard curves at the building sites. The results show, that none of the three buildings reach the required target annual probability of exceedance and safety index according to the new draft of Eurocode 8-1 [3], if unscaled seismic hazard curves are applied. However, although the safety levels of the three buildings determined with consideration of uncertainties of seismic input, modelling parameters, material and strength characteristics, drift limits, vertical actions and verification method related parameters (e.g. damping) do not fulfil the code requirements, all buildings survived the earthquake series of the 20th and 29th May 2012.

Final investigations in report WB3b-1 [12] with scaled hazard curves and hazard curves at specific sites, selected to match the maximum attainable PGA for a return period of 475 years, clarify that the required target annual probability of exceedance and safety index according to the draft of Eurocode 8-1 [3] can only be achieved for reduced seismic hazard curves. This confirms that the hazard level at the building locations was too high for the building resistances, calculated based on the recent code concepts.

Finally, sensitivity studies show that the probabilistic results are significantly influenced by the drift limits, the modelling approach and the vertical load level. The strength characteristics are getting important, if the limits are stricter, as it is the case of the German National Annexes [2], [6]. Furthermore, it has to be pointed out, that the application of the N2-method and the modified N2-method according to Michel et al. [31] produces large scatters in the verification results. Therefore, it is recommended to use either damped spectra or the improved approach according to Guerrini [22]. However, it has to be considered that the approach according to Guerrini [22] is not implemented in any code concept so far. If damped spectra with the proposed damping functions introduced in report WP3a-2 [14] are applied, the results are more conservative in comparison to the approach according to Guerrini [22]. Therefore, it is recommended to use damped spectra as long as new inelastic spectra are not implemented in the codes.

The results of the first part of the project clarify the deficiencies of the linear approach and the need of code improvements to get more realistic design results. Nonlinear static analyses lead to much more realistic results, but depending on the verification procedure and modelling approach there is still a gap with respect to the observed resistances. The performed probabilistic analyses using nonlinear static analyses identified drift limits and the type of modelling as the most important influence factors on the verification results. Therefore, is of utmost importance to define detailed rules for a reliable application of nonlinear static analysis.

However, it has to be considered that linear analyses will be the standard for engineers for the coming years. Due to this fact it is essential to consider the existing load bearing reserves, if the traditional force-based design is applied. This requires further investigations in order to quantify the safety margin between the results of linear and more precise nonlinear calculation methods. These investigations are the part of the second project part with financial support by the DIBt (Deutsches Institut für Bautechnik) in Berlin.

1.2 Project part II (DIBt)

The objective of the second project part is to find reliable solutions to make a part of the identified safety margins accessible in the everyday design praxis of typical masonry construction types in Germany. The second project part is financially supported by the DIBt and carried out in the following six consecutive working packages.

WP 1: Basics for linear and nonlinear verification concepts

The verification using nonlinear static analyses allows a better utilization of the nonlinear load bearing reserves of masonry. However, their use is not yet approved in Germany because of the insufficient description of this method of analysis in the basic document DIN EN 1998-1 [6]. This shortcoming is also reported in the final report on the practical applicability of DIN EN 1998-1 [6] under supervision of the German Institute for Building Technique (DIBt). The aim of this working package is a summary of the linear and nonlinear verification concepts. At first, the traditional linear elastic design approach is described shortly as a basis for the modifications in WP 4. In the next step the application of nonlinear static analysis to masonry buildings is introduced considering the following aspects: drift limits, torsional effects, modelling approach, redistribution of shear and vertical forces, wall-slab interaction, verification approach, safety factors, material properties, damage states.

WP 2: Selection and definition of representative masonry buildings

This work package includes the definition and modelling of representative masonry buildings in Germany, which are further used in the computational parametric studies in WP3. A terraced, single-family and multi-family house with continuous shear walls and variable number of floors are considered. The buildings are selected with respect to the engineering practise and a previous parametric study conducted in 2009 [28]. Each building is described in detail.

WP 3: Parametric study

This working package contains a systematic parametric study for the selected representative masonry buildings based on verifications using linear elastic and nonlinear static analyses. The nonlinear verification results are regarded as reference solutions with a higher utilization of the calculative load bearing reserves. The comparison of the linear and nonlinear verification results is the basis for the development of a rational approach to improve the traditional force-based design concept. For selected building configurations, the calculations are carried out with two software packages to validate the verification results. RWTH executes the calculations with MINEA-Research [38] and UNIK with an alternative software package, e.g. 3muri [39].

WP 4: Modification of the traditional linear elastic design approach

The comparison of linear and nonlinear verification results serves as a basis for deriving a new approach for a better utilization of the load-bearing reserves when applying the traditional force-based design method. The basic idea is to activate the load bearing reserves by a redistribution of the seismic forces among the continuous shear walls. This procedure is justified since the linear elastic and stiffness-proportional load distribution differs strongly from the load distribution close to the ultimate limit state. The challenge of this working package is the development of a generalized solution for arbitrary floor plan configurations and its integration into the traditional force-based design concept.

WP 5: Reports and meetings

The first online project meeting took place at the beginning of May. The final meeting with presentation of the project results to the steering committee will be organized in Aachen in December 2018. In addition to the project meetings, continuous communication will take place via online meetings of the project partners. The basic calculation approach as well as the input values and boundary conditions of the parametric study and its results are explained and discussed in detail in several reports.

WP 6: Dissemination

The transfer of project results into the engineering practice is guaranteed through the activities of the applicants in the standard committees NA 005-06-37 AA (Earthquake Safety of Masonry) and NA 005-51-06 AA (Earthquakes - Special issues). In addition, the project results will be presented on conferences and training courses for practising engineers. A part of the results obtained in the first project part was already published on the European Conference of Earthquake Engineering (ECEE) [36] and in a journal article [37].

2 BASICS FOR LINEAR CALCULATION AND DESIGN

2.1 General design approach

According to both, the basic document of DIN EN 1998-1 [6] and the German National Annex DIN EN 1998-1/NA [7], the calculations of the selected masonry buildings are carried out for the seismic design situation using linear elastic calculation models. The stress resultants of the calculations are used to verify the safety of each single wall according to the basic document DIN EN 1996-1-1 [1] and the German National Annex DIN EN 1996-1-1/NA [2].

2.2 Modelling approach

The cantilever modelling approach is applied for regular masonry buildings with the continuous shear walls over the building height. Each wall contribution is simply considered by an equivalent beam with shear deformations. The system stiffness in the main building directions is calculated as the sum of the single wall stiffness'. To calculate the overall stiffness, all walls shall fulfil the requirement of minimum wall length in accordance with DIN EN 1998-1/NA, Table NA.10 [6]. The single wall stiffness can be calculated by taking into account a reduction factor for shear deformations according to Müller and Keintzel [32]:

$$I_E = \frac{I}{1+\frac{3{,}64EI}{h^2GA}}$$

Here, E is the elastic modulus, h is the building height, G is the shear modulus and A is the shear wall area. Similar to the building practice frame actions due to the restraint of slabs, the contribution of spandrels and interlocking effects of transverse walls are not considered. Although these assumptions lead to results on the safe side, they can be regarded as representative for the recent design philosophy in Germany. Wall areas with openings due to doors and windows are neglected and the single continuous walls are acting as decoupled cantilever arms. Figure 2-1 illustrates the equivalent beam method, in which the continuous shear walls are substituted by uncoupled cantilever beams. The basement is regarded as rigid and not considered.

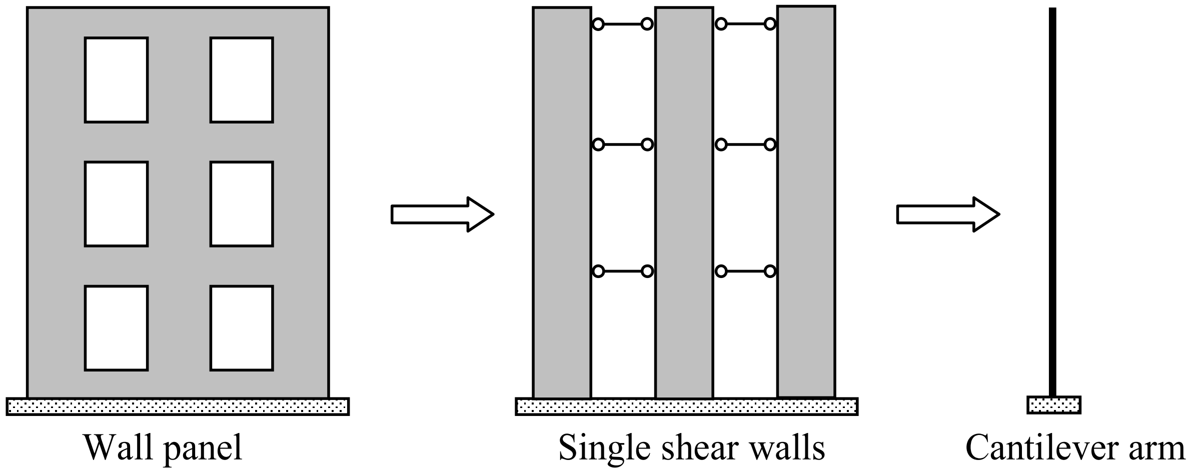


Figure 2-1 Wall panel, single shear walls and cantilever arm as equivalent static system

The interaction of perpendicular walls can only be considered if a sufficiently stiff interconnection of intersecting walls with shear transfer in vertical direction is guaranteed. Shear transfer between intersecting walls can be assumed if the walls are bound or tied together with suitable connectors. However, the interaction is neglected in the parametric study, as reliable connections are usually not executed in practise.

2.3 Lateral force method of analysis

2.3.1 Total base shear

Linear calculations are carried out using the simplified response spectrum method. The building is idealized as MDOF system with concentrated masses on each floor level. The resulting seismic base shear force F_b is the product of the ordinate of the design response spectrum $S_a(T_1)$ at fundamental period T_1 and the total mass m of the building:

$$F_b = S_a(T_1) \cdot m \cdot \lambda$$

In case of masonry buildings, the first natural period is usually quite low and yield to maximum spectral accelerations in the plateau range of the spectrum. The mass correction factor λ is set equal to 0.85 if the number of stories is greater than two and the fundamental period T_1 is smaller than $2T_c$ (corner period of the design spectrum). In all other cases λ is applied with 1.0. The behaviour factor q is applied according to DIN EN 1998-1/NA [7], Table NA.9, which results in most of the configurations to the minimum value of 1.5. The behaviour factor q is used to calculate the reduced design base shear F_b. Figure 2-2 illustrates the calculation steps of the total design base shear.

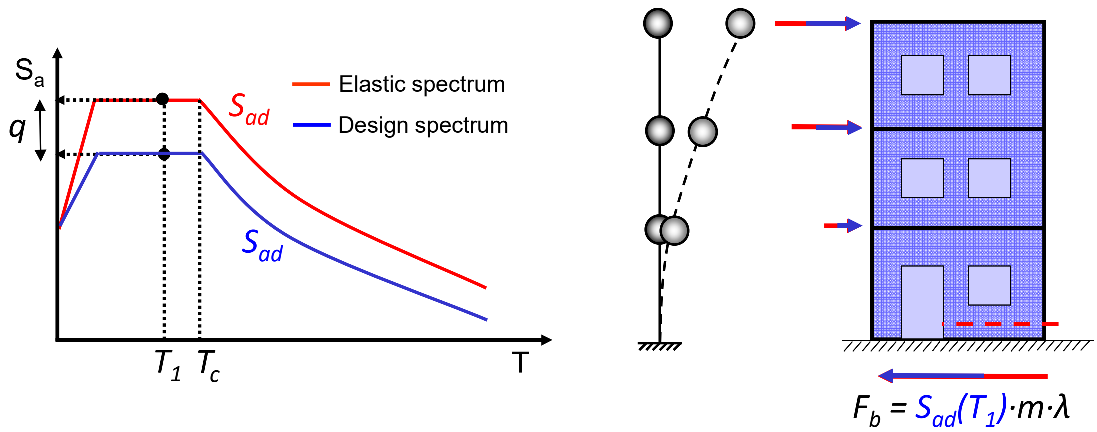


Figure 2-2: Calculation of the total design base shear

The distribution of the total base shear F_b to the storey levels of the building is executed height and mass proportional. The horizontal forces at each storey level are calculated to:

$$F_i = F_b \cdot \frac{z_j \cdot m_i}{\sum z_j \cdot m_j}$$

Here, z_i, z_j are the heights of the masses m_i and m_j at each floor level above the foundation level. The distribution to the storey levels is illustrated in Figure 2-3.

2.3.2 Combination of the effects of seismic actions

As the investigated buildings of the parametric study can be regarded as regular in plan and the walls are the only primary seismic elements in the two main horizontal directions, the seismic actions are assumed to act separately and without combinations along the two main orthogonal horizontal axes of the structure (DIN EN 1998-1 [6], Section 4.3.3.5.1(8)).

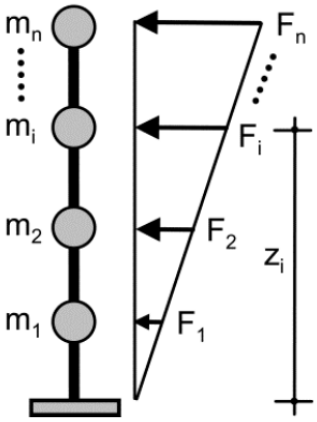


Figure 2-3: Distribution of the total seismic shear F_b over the building height

2.3.3 Torsional effects

The torsional effects are considered by a more accurate calculation approach according to DIN EN 1998-1/NA, NA.D.4 [7], which can be applied if the centres of stiffness and masses of the individual stories are approximately on a vertical line and if the conditions $r_x^2 > l_s^2 + e_{ox}^2$ and $r_y^2 > l_s^2 + e_{oy}^2$ are satisfied. Herein, l_s^2 is the square of the radius of gyration, which corresponds to the quotient of the mass moment of inertia of the regarded storey for rotations around the vertical axis through its centre of gravity and the storey mass. The square of the radius of gyration l_s^2 for a rectangular plan with dimensions L and B and uniformly distributed mass is equal to:

$$l_s^2 = (L^2 + B^2)/12$$

The approach considers torsional effects in each direction of the seismic action by means of the structural eccentricity e_o, the additional eccentricity e_2 (consideration of dynamic effects due to simultaneous translational and torsional vibrations) and the accidental eccentricity e_1. The additional eccentricity e_2 results as a minimum from the following expressions:

$$\text{Min}\begin{cases} e_2 = 0.1 \cdot (L+B) \cdot \sqrt{\dfrac{10 \cdot e_0}{L}} \leq 0.1 \cdot (L+B) \\ e_2 = \dfrac{1}{2e_o}\left[l_s^2 - e_0^2 - r^2 + \sqrt{\left(l_s^2 + e_0^2 - r^2\right)^2 + 4 \cdot e_0^2 \cdot r^2}\right] \end{cases}$$

The maximum and minimum eccentricities for each storey are calculated from the eccentricities e_0, e_1 and e_2:

$$e_{max} = e_0 + e_1 + e_2\,, \quad e_{min} = 0.5\, e_0 - e_1$$

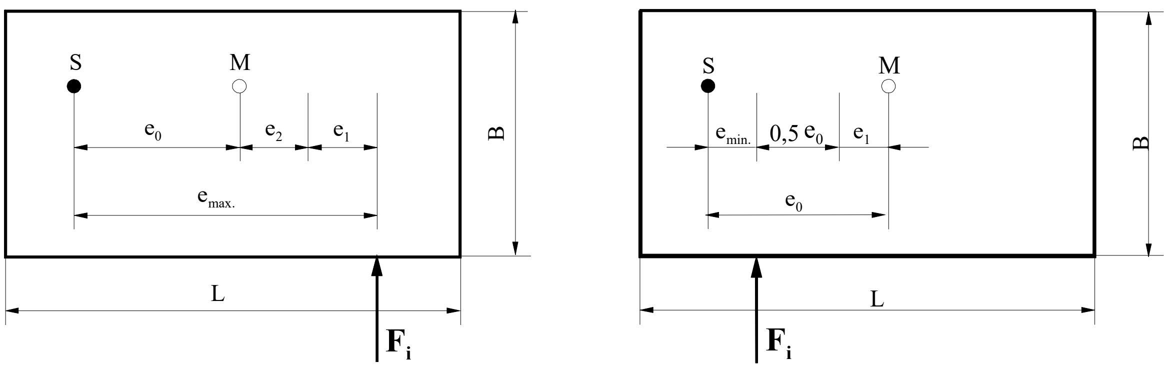


Figure 2-4 Maximum and minimum eccentricities for planar models

Thereafter, the resulting seismic forces in the stiffening elements parallel (index *i*) and perpendicular (index *j*) to the direction of the seismic action are calculated with the maximum and minimum eccentricities e_{min} and e_{max}. For this purpose, load distribution factors s_i, s_j for the stiffening elements in each storey are calculated parallel and perpendicular to the direction of the seismic action. The distribution factors correspond to a percentage of the horizontal seismic forces acting on the floor levels and can be calculated as follows:

$$s_i = \frac{k_i}{K_i}\left(1 \pm \frac{K \cdot r_{di} \cdot e}{K_T}\right), \quad s_j = \frac{k_j \cdot r_{dj} \cdot e}{K_T}$$

2.4 Seismic design situation

The design value E_{dAE} of the effects of actions in the seismic design situation is determined in accordance with DIN EN 1990 [8]:

$$E_{dAE} = E\left\{\sum_{j\geq 1} G_{k,j} \oplus P_k \oplus \gamma_1 \cdot A_{Ed} \oplus \sum_{i\geq 1} \psi_{2,i} \cdot Q_{k,i}\right\}$$

where:

E	Effect of actions
$\oplus$	„in combination with“
$G_{k,j}$	Characteristic value of a permanent action j
A_{Ed}	Design value of seismic action
$Q_{k,i}$	Characteristic value of a single variable action i
$\psi_{2,i}$	Combination coefficient for quasi-permanent value of a variable action i
P_k	Characteristic value of prestressing actions
γ_1	Weighting factor for seismic actions, ($\gamma_1 = 1.0$)

The design value of the seismic action A_{Ed} is determined taking into account permanently acting vertical loads:

$$A_{Ed} = A\left\{\sum G_{k,j} \oplus \sum \psi_{E,i}\, Q_{k,i}\right\} = A\left\{\sum G_{k,j} \oplus \sum \varphi\, \psi_{2,i}\, Q_{k,i}\right\}$$

The combination coefficients $\psi_{E,i} = \varphi \cdot \psi_{2,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. The factor φ is defined in Table NA.5 of DIN EN 1998-1/NA [7], and the combination coefficients $\psi_{2,i}$ are given in DIN EN 1990 [8], Table A1.1. The safety verification is fulfilled, if the design value E_{dAE} of the effects of actions in the seismic design situation does not exceed the corresponding design resistance R_d for all structural elements including connections and relevant non-structural elements:

$$E_{dAE} < R_d = R\left\{\frac{f_k}{\gamma_m}\right\}$$

The design resistance is calculated in accordance with the material-specific rules in terms of the characteristic value of material properties f_k and partial factors γ_M.

2.5 Verification procedure according to DIN EN 1996-1-1/NA

The verifications are performed at the ultimate limit state for the seismic design situation according to DIN EN 1996-1-1/NA [2] in combination with DIN EN 1998-1/NA [6]. The relevant verifications are described in the following sections.

2.5.1 Combined vertical and horizontal loading at the base and top of the wall (BL)

The check under combined vertical and horizontal loading is performed by comparison of the applied normal force N_{Ed} and the resistance N_{Rd}. The resistance is obtained according to Eurocode 6, Section 6.1.2 [1]:

$$N_{Rd} = \Phi \cdot l \cdot t \cdot f_d$$

with:

Φ Capacity reduction factor at the top or bottom of the wall with respect to the wall slenderness and eccentricity

f_d Design compressive strength of masonry, $f_d = \frac{f_k}{\gamma_M}$

l Length of the wall

t Thickness of the wall

The eccentricity has to be determined for both, in-plane and out-of-plane eccentricity and at the base and top of the wall. The capacity reduction factor due to an in-plane eccentricity at the base or top of the wall is calculated by using the eccentricity e_W:

$$\Phi_{IP} = 1 - 2 \cdot \frac{e_W}{l} = 1 - 2 \cdot \frac{M_{Ed}}{N_{Ed} \cdot l}$$

with:

M_{Ed} design value of bending moment at the location of verification (base or top of the wall)

N_{Ed} design value of vertical load at the location of verification (base or top of the wall)

e_W Eccentricity of the wall: $e_W = \frac{M_{Ed}}{N_{Ed}}$ (base or top of the wall)

The reduction factor due to an out-of-plane eccentricity is determined as follows:

$$\Phi_{OOP} = 1 - 2 \cdot \frac{e_i}{t}$$

where e_i is the eccentricity at the base or top of the wall, as appropriate, calculated using the following equation:

$$e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05 \cdot t$$

with:

M_{id} Design value of bending moment at the base or top of the wall, resulting from the eccentricity of the slab at the support.

N_{id} Design value of vertical load at the base or top of the wall

e_{he} Eccentricity at the base or top of the wall due to horizontal loads (e.g. wind)

e_{init} Initial eccentricity at the base or top of the wall

It has to be pointed out, that DIN EN 1996-1-1/NA [2] prescribes to combine the reduction factors for in-plane and out-of-plane eccentricities, while the basic document DIN EN 1996-1-1 [1] does not include any combination of these factors and verifies both directions separately. The total reduction factor Φ_{TOT} according to DIN EN 1996-1-1/NA [2] is equal to:

$$\Phi_{TOT} = \Phi_{\mathrm{IP}} \cdot \Phi_{\mathrm{OOP}}$$

2.5.2 Combined vertical and horizontal loading in the middle of the wall (BL)

The verification in the middle of the wall corresponds to the verification at the base and top of the wall. The resistance can be calculated as:

$$N_{Rd} = \Phi_{\mathrm{m}} \cdot L \cdot t \cdot f_d$$

The resulting reduction factor Φ_{m} is determined considering the following eccentricities:

$$\mathrm{e_{mk}} = \mathrm{e_m} + \mathrm{e_k} \geq 0.05 \cdot \mathrm{t}$$

$$\mathrm{e_m} = \frac{\mathrm{M_{md}}}{\mathrm{N_{md}}} + \mathrm{e_{hm}} + \mathrm{e_{init}}$$

with:

e_m Eccentricity due to loading

e_k Eccentricity due to creep: $e_k = 0.002 \cdot \frac{\Phi_\infty h_{ef}}{t_{ef}} \sqrt{t \cdot e_m}$

M_{md} Design moment at the middle of the height of the wall resulting from the moments at the top and bottom of the wall, including any load applied eccentrically to the face of the wall

N_{md} Design value of vertical load at the middle height of the wall, including any load applied eccentrically to the face of the wall

e_{hm} Eccentricity at mid-height resulting from horizontal loads

e_{init} Initial eccentricity with a sign that increases the absolute value of e_m

h_{ef} Effective wall height according to DIN EN 1996-1-1 [1], Section 5.5.1.2

t_{ef} Effective wall thickness according to DIN EN 1996-1-1 [1], Section 5.5.1.3

Φ_∞ Final creep value coefficient according to DIN EN 1996-1-1 [1], Section 3.7.4 (2)

2.5.3 Shear loading (SS, SZ)

The design value of shear resistance V_{Rd} is determined according to DIN EN 1996-1-1/NA [2], NCI 6.2:

$$V_{Rd} = f_{vd} \cdot l_{cal} \cdot t \cdot \frac{1}{c}$$

with:

f_{vd} Design value of shear strength of masonry, with $f_{vd} = f_{vk} / \gamma_M$

γ_M Partial safety factor according to DIN EN 1996-1/NA [2], Table NA.1

l_{cal} Length of the compressed part of the wall. The verification of shear walls under wind loading must be carried out with: MIN (l_{cal} = 1.125 l; l_{cal} = 1.333 $l_{c,lin}$). In all other cases is $l_{cal} = l$, respectively $l_{c,lin}$.

c Shear stress distribution factor (intermediate values can be interpolated linearly)

$c = 1.0$ for $h/l \leq 1$

$c = 1.5$ for $h/l \geq 2$

h Height of the wall

l Total length of the wall

$l_{c,lin}$ Compressed length of the wall: $l_{c,lin} = \frac{3}{2}\left(1 - 2\frac{e_W}{l}\right) l \leq l$

t Thickness of the wall

e_W Eccentricity of the wall: $e_W = \frac{M_{Ed}}{N_{Ed}}$

M_{Ed} Design moment due to loading

N_{Ed} Design value of the vertical load

The characteristic shear strength f_{vk} is considered as the minimal value of:

$$f_{vk} = \min(f_{vlt1}; f_{vlt2})$$

Shear failure due to bed-joint sliding (SS)

The first limit value of shear strength f_{vlt1} is calculated for filled head joints to:

$$f_{vlt1} = f_{vk0} + 0.4 \cdot \sigma_{Dd}$$

For perpend joints unfilled, but with adjacent faces of the masonry units closely abutted together:

$$\mathrm{f_{vlt1}} = 0.5\ \mathrm{f_{vk0}} + 0.4 \cdot \sigma_{\mathrm{Dd}}$$

with:

f_{vk0} Characteristic initial shear strength under zero compression stress

σ_{Dd} Design compressive stress corresponding to the maximum shear stress

Shear failure due to diagonal tension (SZ)

The second limit value of shear strength f_{vlt2} depends on the tension failure of the brick (SZ):

$$\mathrm{f_{vlt2}} = 0.45 \cdot \mathrm{f_{bt,cal}} \cdot \sqrt{1 + \frac{\sigma_{\mathrm{Dd}}}{\mathrm{f_{bt,cal}}}}$$

with:

$f_{bt,cal}$ Calculative tensile strength according to DIN EN 1996-1-1/NA [2], NPD 3.6.2 (3)

2.5.4 Decisive wall sections for verifications (SS, SZ, BL)

Wall failures due to diagonal tension (SZ) usually take part in the middle of the wall and wall failures due to bed-joint sliding (SS) take place at the base of the wall. Therefore, the shear capacity for diagonal tension failure (SZ) is calculated in the middle of the wall, while the shear capacity for bed-joint sliding failure (SS) is calculated at the base of the wall. The bending capacity (BL) is checked at the base, middle and top of the wall.

3 VERIFICATION BASED ON NONLINEAR STATIC ANALYSIS

The safety verification using nonlinear static analysis is based on the comparison of the displacement demand due to seismic loading and the displacement capability of the building. The displacement capability can be described by means of an inelastic static load-deformation curve calculated under monotonously increasing horizontal loads. Such an investigation is also referred as "pushover" analysis and the load-deformation relationship as "pushover curve". The seismic demand is either described by damped spectra [17], [18], [23], [24] or inelastic spectra [22], [31]. In the following sections, the procedure and the theoretical background for the calculation of the pushover curve is explained first, followed by description of the safety verification using damped and inelastic spectra.

3.1.1 Modelling assumptions and input data

Again, the structural system of all three buildings is simply represented by continuous shear walls over the building height, neglecting spandrels and interlocking effects with transverse walls. This simplified modelling allows the assumption, that the failure will take place in the ground floor while the upper stories will mainly remain in the elastic range. For this reason, the control point for the nonlinear pushover curves is chosen on the floor level of the first storey. The basement is regarded as rigid and not considered.

In contrast to the linear elastic models, the interactions of walls and slabs are considered by the moment distribution factor α. The moment distribution factor is detailed described in Section 3.1.6. As nonlinear calculations are carried out, mean values of the material and strength parameters are applied. The seismic input is defined by linear elastic design spectra according to DIN EN 1998-1-1/NA [7].

The vertical loads for the pushover analysis are applied as calculated for the linear elastic models (Section 2.3.1) and kept constant during the nonlinear calculation. Due to the conservative assumption of a failure in the ground floor, investigations of further load patterns are unnecessary. All nonlinear calculations are executed with MINEA-R [38].

3.1.2 Pushover curve

The overall building capacity is described by means of an inelastic static load-deformation curve under monotonously increasing horizontal loads. It is assumed that the failure mechanism take part in the ground floor, characterized by a large inelastic drift while the upper stories remain linear elastic. Therefore, the overall capacity of the building is represented by the pushover curve of the ground floor. The pushover curve is calculated iteratively by imposing a displacement increment Δx in the direction of the seismic action. Afterwards the resulting forces of all single shear walls are calculated using shear wall capacity curves, whose calculation is presented in Section 3.1.3. The typically non-symmetric wall configuration leads to torsional effects, which produces a rotation of the system around the centre of mass. In order to find the equilibrium, the system is rotated by $\Delta\varphi$ and translated by Δy in the perpendicular direction. The resulting pair of imposed displacement and reaction force in the direction of seismic action is a single point of the pushover curve. The overall pushover curve is calculated by repeating the calculation for different displacements Δ_{GF} (Figure 3-1). Similar to the building practice the contributions of spandrels and the interlocking effects of perpendicular walls are not considered. Therefore, wall sections with openings due to doors and windows are completely neglected. Accidental torsional effects are simply considered within the iteration scheme by imposing an eccentricity between the centre of mass and stiffness. The redistribution of vertical forces is not considered during the calculation. As experimental tests show, this assumption is conservative.

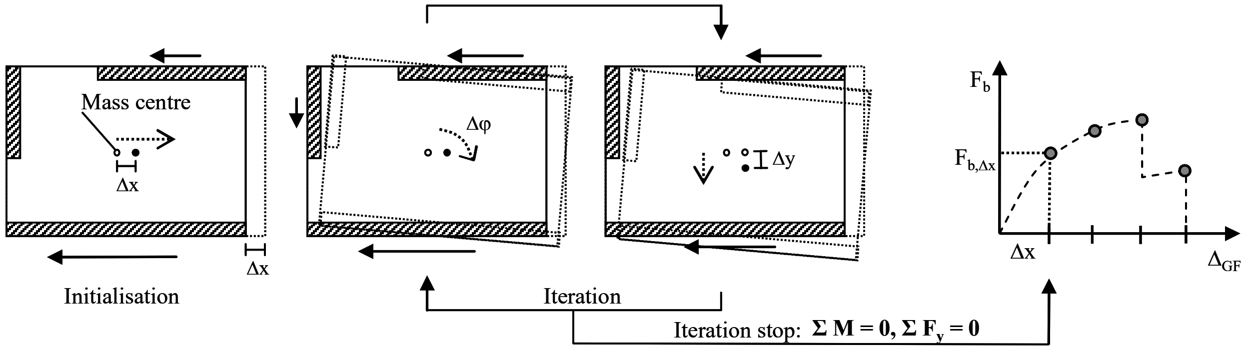


Figure 3-1 Procedure for the calculation of the pushover curve for the ground floor

3.1.3 Wall capacity curves

The nonlinear wall capacity curves are calculated based on the resistances for bending and shear according DIN EN 1996-1-1 [2]. Furthermore, the deformation limits (Section 3.1.5) and the moment distribution factor (Section 3.1.6) are required as input parameters. The nonlinear calculation model idealizes the masonry wall as a Timoshenko beam element considering bending and shear deformations (Figure 3-2).

In the elastic range, the stiffness of the wall is defined by the superposition of shear and bending stiffness of the equivalent beam. In the nonlinear range cracked stiffness values for shear and bending are calculated with respect to the compressed length of the wall. The capacity of each wall is determined within a deformation-controlled calculation using a tangent stiffness formulation. For each applied displacement step the corresponding shear forces are calculated as the minimum of the actual bending and shear resistances. The calculation procedure results in a nonlinear load-displacement curve for the wall. As common in the context of nonlinear analyses, mean values of the material properties and strengths are used for the nonlinear calculation. The overall iteration procedure is shown in Figure 3-3.

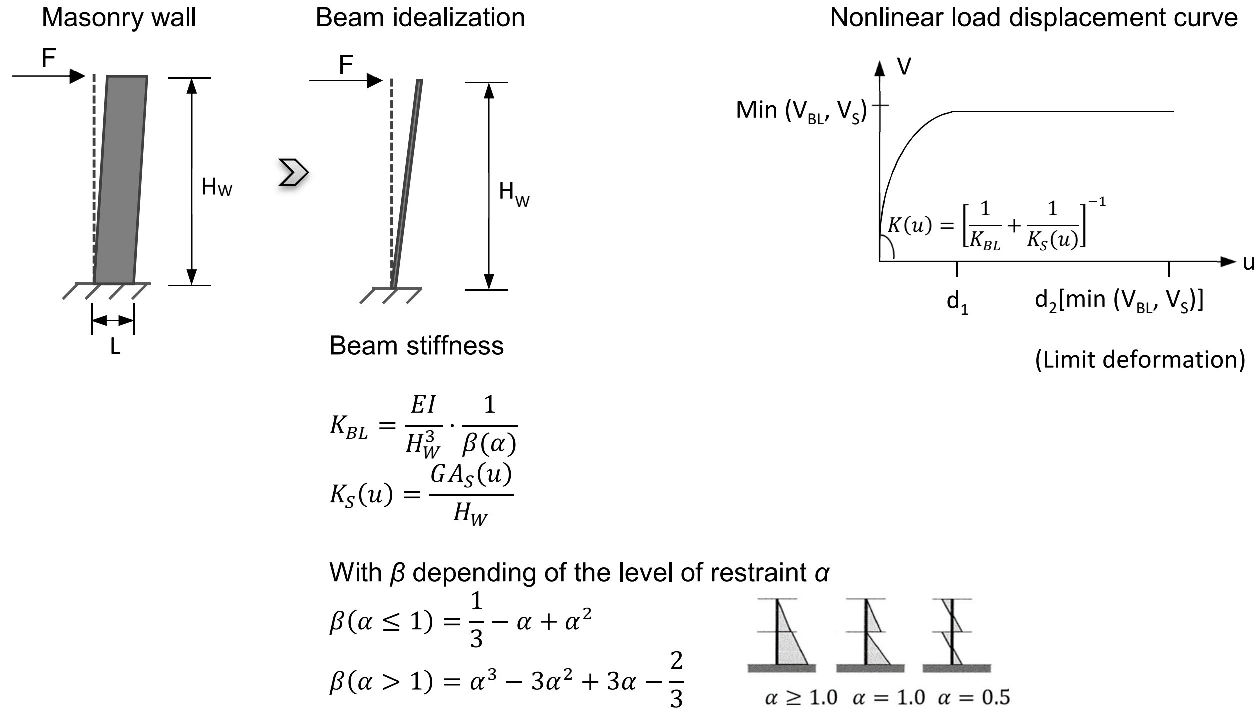


Figure 3-2 Nonlinear calculation procedure of wall capacity curves

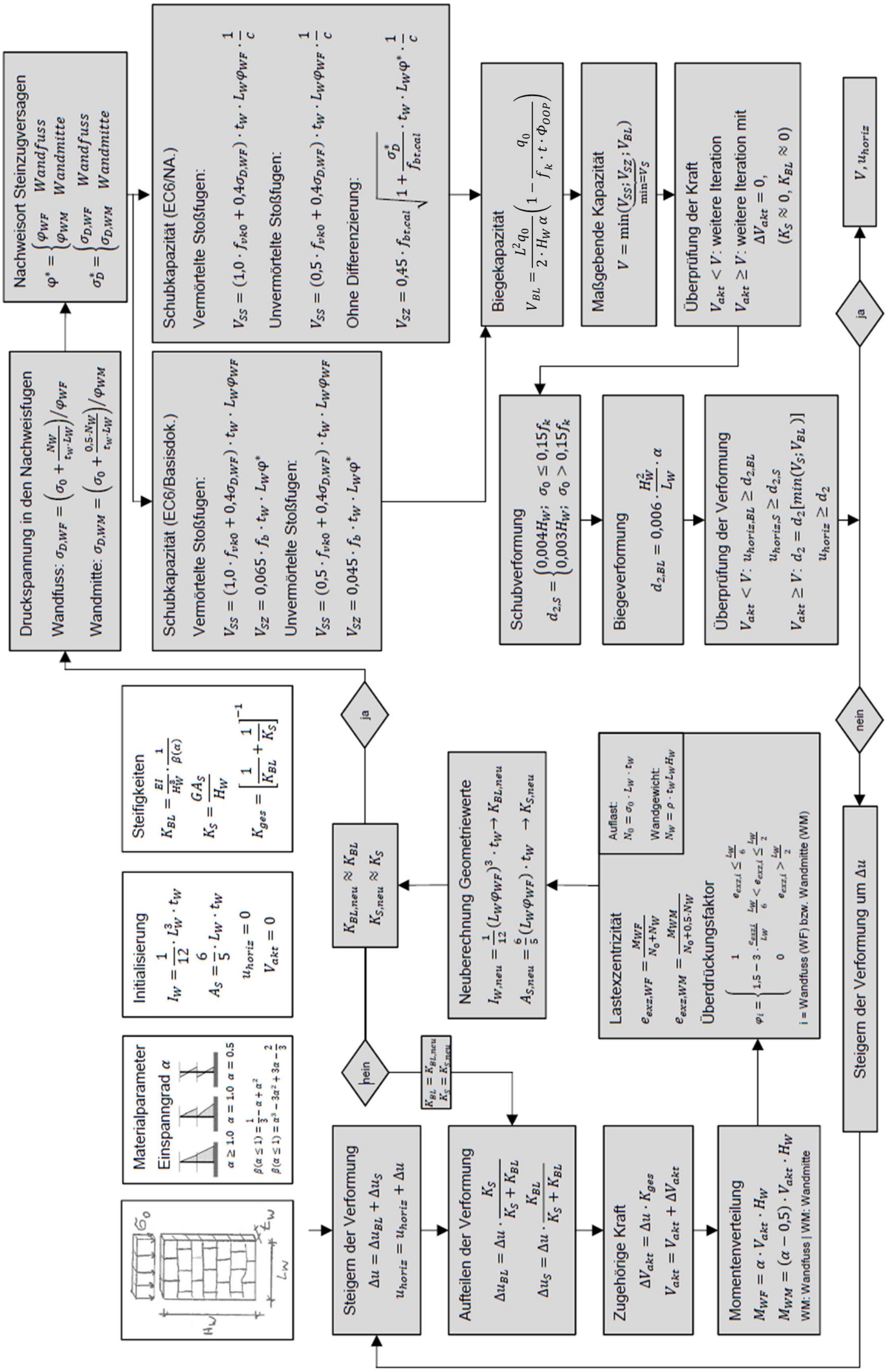


Figure 3-3 Overall nonlinear iteration procedure for the calculation of wall capacity curves.

3.1.4 Failure modes

Three possible failure modes according to DIN EN 1996-1-1/NA [2] are considered:

- Shear failure due to bed-joint sliding (SS)
- Shear failure due to diagonal tension (SZ)
- Bending failure (Combined vertical and horizontal loading) (BL)

The shear resistances (SS, SZ) are calculated according to the formulas already introduced in Section 2.5.3. The shear resistance V_{Rd} for a masonry wall subjected to flexural loading can be calculated from the equilibrium of a masonry wall subjected to vertical and horizontal loading (Figure 3-4).

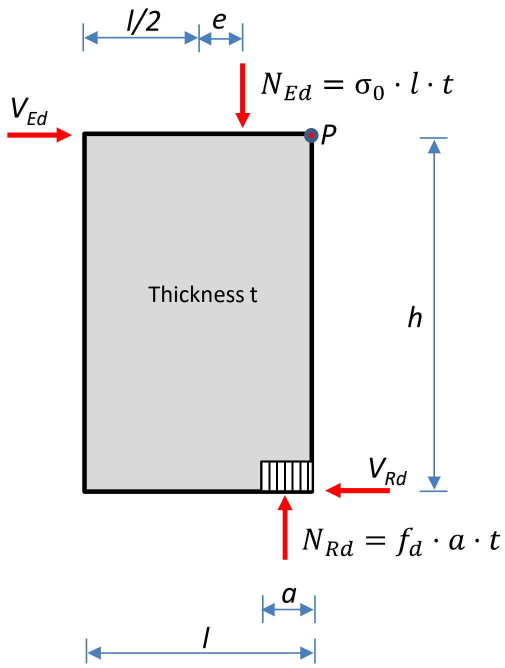


Figure 3-4 Equilibrium of a masonry wall subjected to vertical and horizontal loading

The maximum flexural strength at the ultimate state can be calculated by means of equilibrium conditions with the average vertical stress at the top σ_0, the design compressive strength of masonry f_d and the length a of the compression block. The equilibrium in vertical direction leads to:

$$\left.\begin{aligned} N_{Rd} &= f_d \cdot a \cdot t \\ N_{Ed} &= \sigma_0 \cdot l \cdot t \end{aligned}\right\} \Rightarrow \sigma_0 \cdot l \cdot t = f_d \cdot a \cdot t \Rightarrow a = \frac{\sigma_0\, l}{f_d}$$

The maximum eccentricity e_{max} is determined by the moment equilibrium with respect to point P:

$$N_{Rd}\,\frac{a}{2} - N_{Ed}\left(\frac{l}{2} - e_{max}\right) = 0$$

$$\Rightarrow \quad N_{Rd}\,\frac{a}{2} - N_{Ed}\,\frac{l}{2} + N_{Ed}\, e_{max} = 0$$

$$\Rightarrow \quad \sigma_0^2\,\frac{l^2 t}{2\, f_d} - \frac{\sigma_0 l^2 t}{2} + \sigma_0\, l\, t\, e_{max} = 0$$

$$\Rightarrow \quad e_{max} = \frac{l}{2} - \frac{\sigma_0\, l}{2\, f_d} = \frac{l}{2}\left(1 - \frac{\sigma_0}{f_d}\right)$$

The bending moment at the ultimate limit state M_{Rd} is obtained by multiplying N_{Ed} by e_{max}:

$$M_{Rd} = \frac{\sigma_0 \, t \, l^2}{2}\left(1 - \frac{\sigma_0}{f_d}\right)$$

Finally, the maximum shear resistance V_{Rd} can be calculated as follows:

$$V_{Rd} = \frac{M_{Rd}}{\alpha \, h} = \frac{\frac{\sigma_0 \, t \, l^2}{2}\left(1 - \frac{\sigma_0}{f_d}\right)}{\alpha \, h} = \frac{q_0 \cdot l^2}{2 \cdot \alpha \cdot h}\left(1 - \frac{q_0}{f_d \cdot t}\right)$$

where h is the height of the masonry wall, α is the moment distribution factor which defines the position of the inflection point (0.5 for fully fixed and 1.0 for a cantilever wall) and $q_0 = \sigma_0 \cdot t$ is the vertical load per meter at the top of the wall. Alternatively, the shear resistance V_{Rd} can also be recalculated from DIN EN 1996-1-1, Section 6.1.2 [1]. Here, the congruent resistance N_{Rd} is equal to:

$$N_{Rd} = \Phi_{TOT} \cdot l \cdot t \cdot f_d = \Phi_{IP} \cdot \Phi_{OOP} \cdot l \cdot t \cdot f_d$$

The reduction factor Φ_{IP} for bending in wall direction is calculated according to DIN EN 1996-1-1/NA [2], Section NA.K.2:

$$\Phi_{IP} = 1 - 2\frac{V_{Ed}}{N_{Ed}} \cdot \alpha \cdot \frac{h}{l}$$

$$\Phi_{TOT} = \Phi_{\mathrm{IP}} \cdot \Phi_{\mathrm{OOP}}$$

$$\Rightarrow \; N_{Rd} = \left(1 - 2\frac{V_{Ed}}{N_{Ed}} \cdot \alpha \cdot \frac{h}{l}\right) \cdot \Phi_{\mathrm{OOP}} \cdot l \cdot t \cdot f_d$$

$$\Rightarrow \; V_{Ed} = \frac{l \cdot t \cdot f_d N_{Ed}}{2 \cdot \alpha \cdot h \cdot t \cdot f_d} - \frac{N_{Rd} \, N_{Ed}}{2 \cdot \alpha \cdot h \cdot t \cdot f_d \cdot \Phi_{OOP}}$$

$$\Rightarrow \; V_{Ed} = \frac{N_{Ed} \cdot l}{2 \cdot \alpha \cdot h} - \frac{N_{Rd} \, N_{Ed}}{2 \cdot \alpha \cdot h \cdot t \cdot f_d \cdot \Phi_{OOP}}$$

with: $N_{Ed} = N_{Rd} = \sigma_0 \cdot l \cdot t$ (at the ultimate limit state)

$$\Rightarrow \; V_{Rd} = \frac{\sigma_0 \cdot l^2 \cdot t}{2 \cdot \alpha \cdot h \cdot}\left(1 - \frac{\sigma_0}{f_d \, \Phi_{OOP}}\right)$$

with: $q_0 = \frac{\sigma_0}{t}$

$$\Rightarrow \; V_{Rd} = \frac{q_0 \cdot l^2}{2 \cdot \alpha \cdot h}\left(1 - \frac{q_0}{f_d \cdot t \cdot \Phi_{OOP}}\right)$$

Both derivations lead to the same result based on the principles of structural mechanics. However, it has to be considered, that the code application requires the consideration of an additional reduction factor Φ_{OOP} accounting for out-of-plane eccentricities.

3.1.5 Deformation limits

The calculation of the wall capacity curves requires the definition of drift limits with respect to the introduced failure modes for shear and bending. After reaching these drift limits, the wall capacity drops down to zero. The drift limits are defined according to DIN EN 1998-1/NA [7] and summarized in Table 3-1.

Table 3-1: Drift limits for shear and bending

Bending	Shear
$0.006 \cdot \alpha \cdot H_W^2/l$	$0.004 \cdot H_W$, if $\sigma_0 \leq 0.15\ f_k$
	$0.003 \cdot H_W$, if $\sigma_0 > 0.15\ f_k$
H_W: floor height, l: wall length, α: level of restraint	

3.1.6 Moment distribution factor α

Masonry shear walls continuous over the building height or several stories are not acting as simple cantilever arms like reinforced concrete shear walls. In case of masonry shear walls the tension transfer is missing and the walls start to rotate and to interact with the slabs. Figure 3-5 illustrates that the rotation of the walls leads to an uplift on one side and to a formation of a diagonal compression strut between two corners of the wall. The interaction activates a restoring effect, which is increasing in the higher deformation range. The activation of the frame action with contribution of the slab influences the moment distribution within the shear wall. As illustrated in Figure 3-6 the moment distribution can be described by the factor α as the quotient of the height of the zero moment h' to the wall height h. Usually the moment at base M_u is greater than the moment M_o at the top and the moment distribution factor α is calculated to:

$$\alpha = \frac{h'}{h} = \frac{M_u}{M_u - M_o}$$

In case of eccentrically acting normal forces the moment at the base can be less than the moment at the top and the level of restraint is defined as follows:

$$\alpha = \frac{M_u}{M_o - M_u}$$

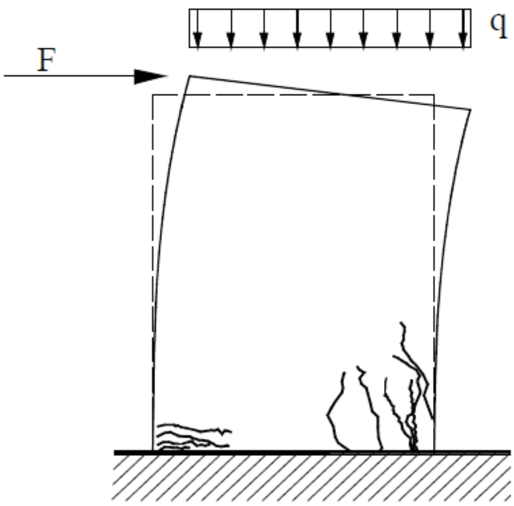
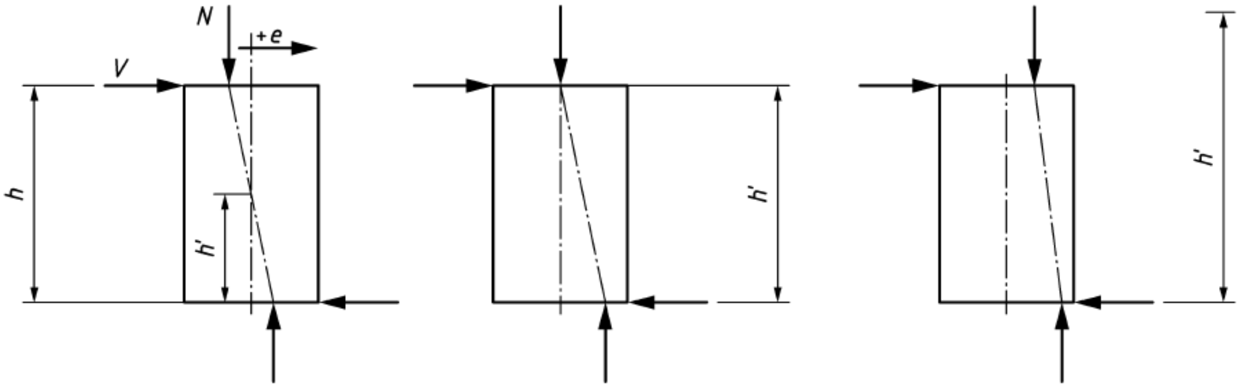


Figure 3-5: In-plane flexural behaviour of a masonry panel [34]

The moment distribution factor is an input parameter for the calculations and kept constant during the nonlinear calculation. As the incorporation of interaction effects on structural level is too complex, the rational and conservative choice of moment distribution factors is really important.



α = 0.5 α = 1.0 α > 1.0

Figure 3-6: Moment distribution and corresponding distribution factor α according to DIN EN 1996-1-1/NA [2]

3.2 Transformation of the pushover curve

The capacity curve of the overall building (Section 3.1.2) refers to a multi-storey building and has to be transformed into the capacity curve of an equivalent single-degree-of-freedom oscillator. The transformation is performed for each deformation state *i* with a corresponding deformation Δi and an associated force $F_{b,i}$:

$$S_{a,i} = \frac{F_{b,i}}{M_{1,eff}}$$

and

$$S_{d,i} = \frac{\Delta_i}{\beta_1 \cdot \Phi_1}$$

Here, $M_{1,eff}$ is the effective mass, β_1 represents the participation factor and Φ_1 is the ordinate at the control point of the first natural Eigenmode. The transformation can be carried out using the initial stiffness and corresponding modal parameters. Alternatively, the modal parameters can be recalculated for each deformation state taking into account an updated secant stiffness formulation.

3.3 Safety verification using damped spectra

In 1975, Freeman et al. [24] developed a displacement-based design approach, called capacity spectrum method (CSM). The CSM is a nonlinear static method which compares the seismic action with the load bearing capacity of the building. The seismic action is represented by a reduced response spectrum due to damping and the building capacity is described by an inelastic cyclic pushover curve. As illustrated in Figure 3-7 both curves are converted into acceleration-displacement response spectral ordinates (ADRS format). The intersection point of both curves is called “Performance Point” and delivers the maximum spectral displacement of the equivalent single degree of freedom oscillator from which the maximum displacement of the chosen control point can be calculated by inverse transformation.

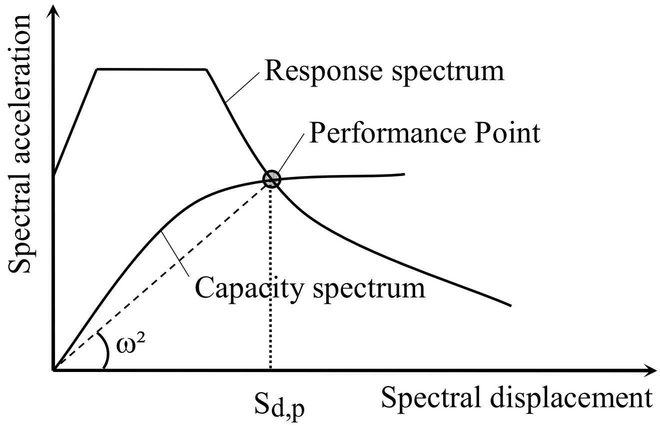


Figure 3-7: Performance Point: Superposition of the pushover curve and the damped response spectrum.

The energy dissipation is taken into account by means of an effective damping ξ_{eff}, calculated as the sum of the equivalent viscous damping ξ_0 and the hysteretic damping ξ_{hyst}. The effective damping is calculated for each point of the capacity curve and used to scale the elastic response spectra by the reduction factor η proposed by Priestley and Grant [35]

$$\eta = \sqrt{\frac{7}{2+\xi_{eff}}} \text{ with } \xi_{eff} = \xi_0 + \xi_{hyst}$$

As recommended in DIN EN 1998-1 [6] the equivalent viscous damping ξ_0 is considered with 5%. The calculation of the hysteretic damping is based on a comprehensive evaluation of cyclic shear wall tests by Norda [33]. The damping is described in terms of the ductility μ depending on the dominant failure modes for bending and shear:

- Bending failure (BL)
- Shear failure due to bed joint sliding (SS)
- Shear failure due to diagonal tension (SZ)

The effective damping of each individual wall is calculated according to a parametrized approach proposed by Dwairi et al. [15] and Priestley et al. [20]:

$$\xi_{eff} = \xi_0 + c \cdot \left(\frac{\mu-1}{\pi \cdot \mu}\right)$$

Norda [33] proposed different constants *c* for each of the dominant failure types, as their amount of damping is quite different. For the calculations of the three buildings median values of the statistical evaluation described by the following functions are applied:

$$\xi_{\text{eff,BL,mean}} = \xi_0 + 0.20 \cdot \left(\frac{\mu-1}{\pi \cdot \mu}\right)$$

$$\xi_{\text{eff,SS,mean}} = \xi_0 + 0.90 \cdot \left(\frac{\mu-1}{\pi \cdot \mu}\right)$$

$$\xi_{\text{eff,SZ,mean}} = \xi_0 + 0.44 \cdot \left(\frac{\mu-1}{\pi \cdot \mu}\right)$$

As the hysteretic damping occurs only in the nonlinear range, hysteretic damping is not considered for ductility values less than 1. The resulting damping functions (median and 5% quantile) and the test data are shown in Figure 3-8, Figure 3-9 and Figure 3-10.

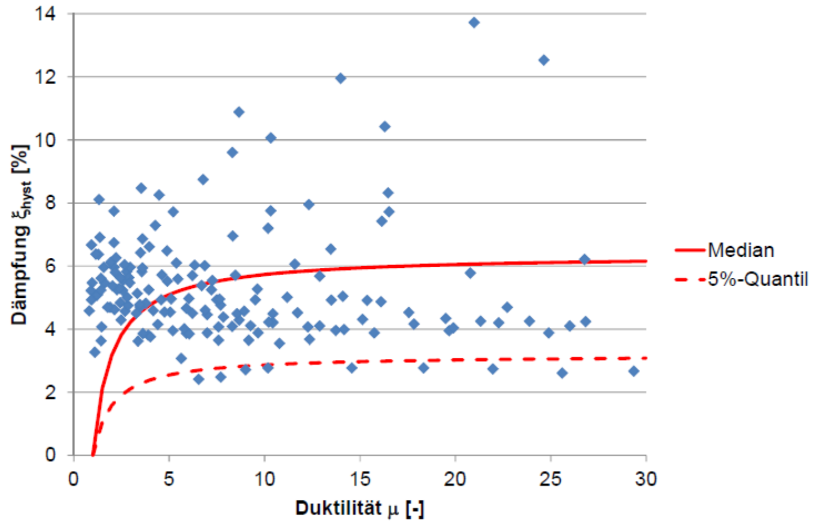


Figure 3-8: Damping functions for the failure mode bending and axial force [33]

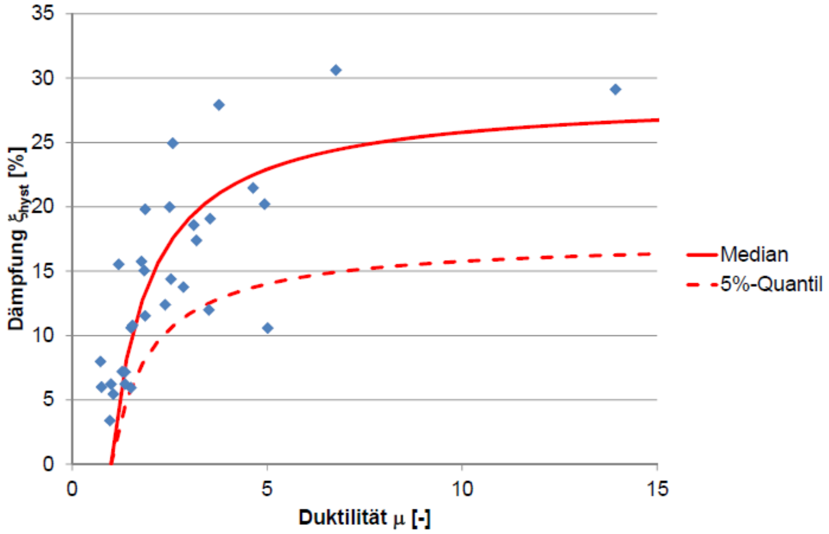


Figure 3-9: Damping functions for shear failure due to bed-joint sliding [33]

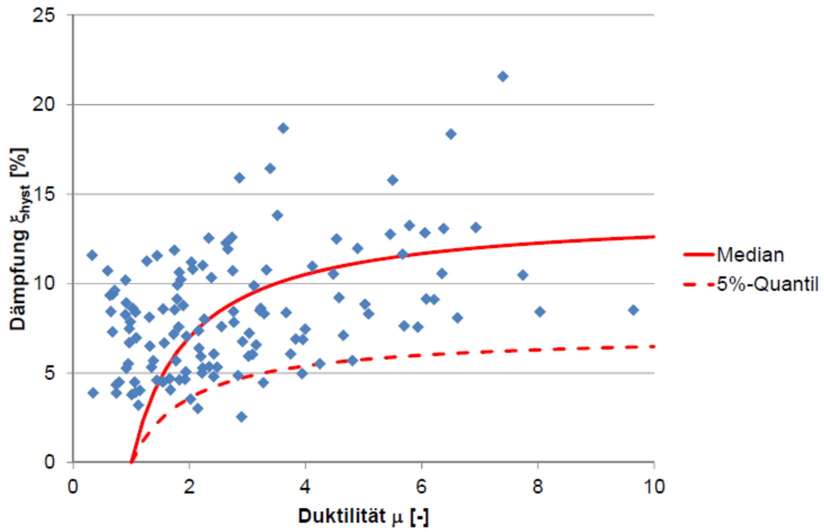


Figure 3-10: Damping functions for shear failure due to diagonal tension [33]

According to Magenes and Calvi [29] an additional damping of 5% is considered for bending failure due to the rocking of the wall. The additional 5% are considered linearly for ductility values μ greater than 1.

The effective damping on structural level is determined taking the damping contribution of each wall with respect to specific failure mode into account. The overall damping of the building is calculated as the ratio of the weighted sum of the single wall damping contributions:

$$\xi_{building} = \frac{\sum V_j \cdot x_j \cdot \xi_j}{\sum V_j \cdot x_j}$$

with:

V_j horizontal load of the wall j

x_j horizontal displacement of the wall j

ξ_j damping ratio of the wall j

The overall damping of the building is used to calculate the reduction factor η along the load-displacement path. This leads to a resulting damped spectrum with increasing damping values in the higher deformation range as shown in Figure 3-11.

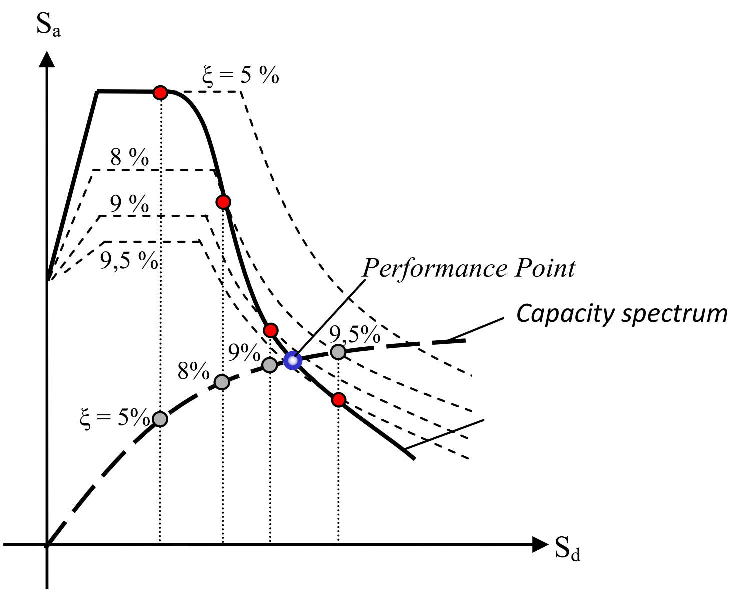


Figure 3-11: Calculation of the damped spectrum [33]

3.4 Safety verification using inelastic spectra

In the literature inelastic spectra for different building types (e.g. reinforced concrete frame, steel frames) are available. Well known approaches have been developed by Chopra and Goel [16], Fajfar [17], [18], Vidic et al.[19] and Priestley et al. [20] in form of ductility functions. Several research activities clarified that the derivation of inelastic spectra is quite sensitive to the subsoil conditions, and thus the spectral shape is important. Furthermore, the applied nonlinear calculation models must be able to mirror the nonlinear behaviour of the regarded structural type. The quality and reliability of the nonlinear models is always a crucial point. Especially for masonry structures with different wall failure modes, complex nonlinear cyclic behaviour and interaction effects with the floor slabs “correct” and realistic modelling and simulation is a challenge. So far, the majority of the inelastic spectra in the literature are not applicable to masonry buildings, since they were derived for reinforced concrete or steel structures.

The problems due to the lack of inelastic spectra for masonry structures are also reflected in the current codes. DIN EN 1998-1 [6] proposes in Annex B the application of inelastic spectra according to Vidic et al. [19], which were derived for reinforced concrete frames. The application of these spectra to masonry structures with short periods completely overestimates the building capacity, which was already recognized in several publications. In the Italian code NTC2008 [9], the N2 method is modified by limiting the reduction factor to $q^* = 3$. However, the reduction is only a reasonable "fit" based on experience and parametric studies, without a strong theoretical background.

More recent Michel et al. [31] proposed a modification of the N2 method for the short period range. Also Graziotti [21] developed new relationships to calculate more realistic inelastic displacements. A continuation of these works is the publication of Guerrini et al. [22]. This publication contains new expressions for the calculation of the inelastic displacements with respect to the specific structural behaviour of masonry structures. However, the current status of the codes is the N2 method and the results of the recent publications are not introduced so far. The results of the application of inelastic spectra with the N2 method in the first project part can be summarised as follows:

- The results of the N2 method are too optimistic and lead to quite different results in comparison to all other proposals. This confirms that the N2 method, as proposed in DIN EN 1998-1 [6], is not directly applicable to short period masonry structures as already recognized and published by several researchers.
- The probabilistic analyses based on the N2 method produce large scatters and is quite sensitive with respect to the initial stiffness and drift limits. Again, it is not recommended to apply the N2 method to short period masonry structures.

Based on the results of the first project part, inelastic spectra are not considered for the parametric study of the representative masonry buildings.

4 DIN EN 1998-1/NA: NONLINEAR STATIC PUSHOVER ANALYSIS

The first project part clarified, that the nonlinear pushover analysis is incompletely described in DIN 1998-1 [6]. Especially the application to masonry buildings needs further descriptions and clear rules for a reliable application in practise. For this reason, a detailed description of the procedure and their application to masonry buildings is part of DIN EN 1998-1/NA [7]. The general description of the nonlinear pushover analysis is given in NCI 4.3.3.4.2.1 to 4.3.3.4.2.7, while special rules for masonry buildings are given in NCI 9.4. All further descriptions are added as non-contradictory complementary information. The additional description enables the practising engineer a reliable application nonlinear pushover analysis to masonry buildings. The parametric study of the representative masonry buildings considers both, the requirements of the basic document DIN EN 1998-1 [6] and the additional descriptions in DIN EN 1998-1/NA [7]. In the following the relevant NCI, implemented in DIN EN 1998-1/NA [7], are summarized.

NCI zu 4.3.3.4.2.1(1) Nichtlineare statische (pushover) Berechnung

*Die Pushover-Berechnung dient zu Ermittlung der inelastischen Last-Verformungskurve (Kapazitätskurve) eines Bauwerks unter konstanten Gewichtslasten und monoton wachsenden Horizontalkräften (**4.3.3.4.2.2**). Die Kapazitätskurve (**4.3.3.4.2.3**) gibt die Beziehung zwischen der Relativverschiebung eines maßgebenden Bezugspunktes (Kontrollknoten nach Anhang B.1) und der Oberkante des Fundaments oder eines starren Kellergeschosses sowie der einwirkenden Gesamterdbebenkraft an. Für die Ermittlung der Kapazitätskurve und den Ansatz der Festigkeiten sind **4.3.3.4.1(1)** bis **(4)** zu beachten. Der Schnittpunkt der Kapazitätskurve mit dem elastischen Antwortspektrum (**3.2.2.2**) im (Spektralbeschleunigungs-Spektralverschiebungsdiagramm (Sa-Sd-Diagramm) liefert die Zielverschiebung (**4.3.3.4.2.6**), die Grundlage des Nachweises ist. Der Nachweis ist erfüllt, wenn die 1,5-fache Zielverschiebung kleiner oder gleich der auf den Bezugspunkt (Kontrollknoten) bezogenen Verformungskapazität ist. Die Verformungskapazität ist diejenige Verformung, bei der gemäß der Kapazitätskurve nach Erreichen der maximalen Tragfähigkeit noch mindestens 80 % des maximalen Tragwerkswiderstands vorhanden ist.*

<u>Hinweis:</u> Falls die kinematische Boden-Bauwerk-Wechselwirkung (DIN EN 1998-5) von Bedeutung ist, sind deren Auswirkungen zu beachten.

NCI zu 4.3.3.4.2.1(2) und (3) Nichtlineare statische (pushover) Berechnung

Folgende Bedingungen sollten bei der Berechnung und der Wahl der Rechenmodelle berücksichtigt werden:

- *Das gewählte Rechenmodell muss in der Lage sein das Schwingungsverhalten und die nichtlinearen Lastumverteilungseffekte im Bauwerk zu erfassen.*
- *Die Decken verfügen über eine ausreichende Steifigkeit in ihrer Ebene und sind in der Lage die Horizontallasten über ausreichend dimensionierte Verbindungen auf die Aussteifungselemente zu verteilen.*
- *Wenn das dynamische Verhalten im Wesentlichen durch die Grundeigenform bestimmt wird, ist es ausreichend nur die modale Verteilung der Horizontalkräfte zu berücksichtigen. Höhere Schwingformen können vernachlässigt werden, wenn die Bedingungen in **4.3.3.2.1(2)** erfüllt sind.*
- *Wird das dynamische Verhalten auch durch höhere Eigenformen beeinflusst, so sind die modalen Horizontalkraftverteilungen der wesentlichen Eigenformen mit wechselnden Vorzeichen zu kombinieren. Für jede Kombination der Eigenformen ist eine eigene Pushover-Analyse durchzuführen.*
- *Dem Nachweis der Zielverschiebung ist die ungünstigste Erdbebenrichtung zugrunde zu legen.*

- *Die Horizontalkomponenten der Erdbebeneinwirkung sind als gleichzeitig anzunehmen. Die Kombination der Richtungen kann nach* ***4.3.3.5.1(6)*** *erfolgen.*

NCI zu 4.3.3.4.2.3(2) Kapazitätskurve

Bei der Ermittlung der Kapazitätskurve sind insbesondere ***4.3.3.4.1(2)*** *bis* ***(6)*** *von Bedeutung. Die Aufstellung der Kapazitätskurve setzt voraus, dass alle am Lastabtrag beteiligten primären seismischen Bauteile in der Lage sind, die auftretenden Beanspruchungen entlang des gesamten Belastungspfads aufnehmen zu können (z. B. zusätzliche Deckenmomente infolge Rahmentragwirkung).*

Für unbewehrte Mauerwerksbauten können zur Ermittlung der Kapazitätskurve des Bauwerks für die Einzelwände die Verformungskapazitäten nach ***NCI zu 9.4(6)*** *unter Ansatz der Biege- und Schubkapazitäten nach DIN EN 1996 angesetzt werden. Zur realitätsnahen Modellierung sollten die Wandsteifigkeiten für Schub und Biegung für gerissene Querschnitte angesetzt werden. Sind keine genaueren Werte verfügbar, können diese aus den um 50 % abgeminderten Werten der Wandsteifigkeiten im ungerissenen Zustand erhalten werden.*

NCI zu 4.3.3.4.2.3(3) Kapazitätskurve

Der Bezugspunkt darf am Massenmittelpunkt des Gebäudedaches angenommen werden, sofern es sich beim obersten Geschoss um ein Vollgeschoss handelt. Ansonsten ist der Bezugspunkt am Massenmittelpunkt der Decke des obersten Vollgeschosses geeignet. Es gilt die Definition des Vollgeschosses nach ***NA.D.8(2)****.*

NCI zu 4.3.3.4.2.6 Zielverschiebung

Das in Anhang B angegebene N2-Verfahren zur Ermittlung der Zielverschiebung ist in der Anwendung auf Stahlbetonrahmentragwerke begrenzt. Alternativ können allgemein die Kapazitätsspektrum-Methode nach ATC 40 oder die Verschiebungs-Koeffizienten-Methode nach FEMA 356 angewendet werden.

NCI zu 4.3.3.4.2.7 Verfahren zur Abschätzung der Torsionswirkungen

Zufällige Torsionswirkungen sind nach ***4.3.2*** *beim Ansatz der Horizontalkräfte in den Massenpunkten des Rechenmodells durch Ansatz von Exzentrizitäten oder Zusatzmomenten zu berücksichtigen.*

NCI zu 9.4(6) Tragwerksberechnung

Alternativ zu den Regeln in 9.4(6) kann eine Umverteilung der durch lineare Berechnung nach Abschnitt 4 ermittelten Gesamterdbebenlast auf die einzelnen Wände auch vorgenommen werden, wenn:

d) die Lastumlagerung unter Verwendung elastisch-ideal plastischer Lastverformungskurven der Einzelwände unter Einhaltung des globalen Gleichgewichts erfolgt. Hierbei darf hinsichtlich der Tragwerksmodellierung nach DIN EN 1996-1-1 verfahren werden.

Dabei dürfen die maximalen Verformungen zwischen Wandfuß und Wandkopf für alle Mörtel und Steinarten von in Deutschland üblichen Bauweisen für unbewehrtes Mauerwerk wie folgt angesetzt werden:

- *Biegeversagen:* $0{,}006 \cdot H_0/L \cdot H$.

- *Schubversagen: 0,004 · H, wenn bei der Bemessungssituation infolge Erdbeben die mittlere Normalspannung 15 % der charakteristischen Mauerwerkdruckfestigkeit f_k nach DIN EN 1996-1-1 oder allgemeiner bauaufsichtlicher Zulassung oder Bauartgenehmigung nicht überschreitet. In allen anderen Fällen ist die Verformung für Schubversagen mit höchstens 0,003 · H anzusetzen.*

Hierbei sind H die Stockwerkshöhe, L die Wandlänge und H_0 der Abstand zwischen dem Querschnitt, in dem die Biegekapazität erreicht wird, und dem Wendepunkt, jeweils in m.

Für eingefasstes Mauerwerk dürfen die für unbewehrtes Mauerwerk angegebenen Verformungswerte um den Faktor 2 vergrößert werden.

5 DAMAGE STATES

Five grades of damage are considered as a function of the interstory drift and the load drop-down after exceeding the maximum displacement along the pushover curve of the building. Figure 5-1 shows the damage grades along an idealized pushover curve, defined in dependency on the yield displacement Δ_y, the maximum displacement Δ_u and the maximum base shear $F_{b,max}$.

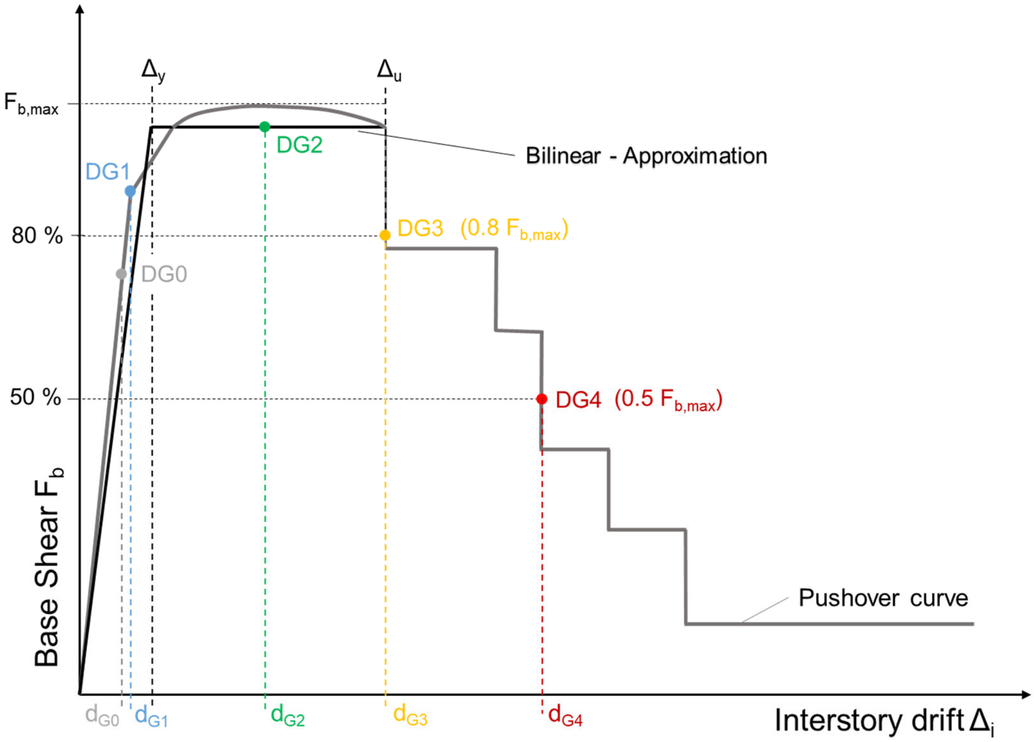


Figure 5-1: Definition of the damage grades along the pushover curve

The characteristics and definitions of the damage grades can be described as follows:

DG0: First cracks – d_{G0}: 0.7 · d_{G1}
Damage grade DG0 characterizes first cracks and is defined by an interstory drift of 70% of damage grade DG1. The building behaves linear and only slight damage in the form of first hairline cracks occurs.

DG1: Slight damage – d_{G1}: Transition from linear to nonlinear
If damage grade (DG 1) is reached, most of the walls show minor cracks and the building starts to behave more and more nonlinear. The load displacement curve is characterized by a first stiffness reduction and the walls start to enter the plastic range, but the maximum displacement Δ_u is still not reached.

DG2: Damage Limitation (DL) – d_{g2}: 0.5 · (Δ_y + Δ_u)
At damage grade DG2 the building exhibit visible wide cracks and the displacements state lies in the middle of the plateau range with the maximum shear capacity $F_{b,max}$. As the stiffness tends to zero, the building reacts with increasing displacements to further seismic actions.

DG3: Significant damage/Life safety (SD) – d_{g3} at 80% of F_{bmax}
At the occurrence of damage grade DG3, one or more walls fail and the building shows significant amount of damage. The DG3 is defined at 80% of the maximum shear force $F_{b,max}$ on the capacity curve. It is assumed, that the building is still safe, if the drop-down of the maximum shear force $F_{b,max}$ does not exceed 20%.

DG4: Near collapse/Collapse prevention (NC) – d_{g4} at 50 % of F_{bmax}

Damage grade DG4 is reached, if the remaining building capacity is reduced to 50% of the maximum shear force $F_{b,max}$. A critical number of walls are heavily damaged or failed and a significant redistribution of horizontal and vertical forces is necessary to avoid partial building collapses. DG4 is a quite unstable building condition and close to a total building collapse.

The background of the definition of the five damage grades is discussed and detailed explained in report 3b-1 [12]. This description is completed by further references and comparison with DIN EN 1998-3 [5]. The introduction of five damage grades with quite similar definitions is also suggested by several other authors as Lang and Bachmann [26], Lagomarsino and Giovinazzi [27] and Kappos and Papanikolaou [25]. Furthermore, the concept is in line with the latest revised draft version of DIN EN 1998-3 [4], in which the ultimate displacement capacity is defined as the displacement at which total lateral resistance drops down below 80% of the peak resistance of the structure, caused by progressive damage and failure of lateral load resisting elements. Therefore, the proposed damage grades can be regarded as a reasonable and up-to-date proposal to classify the structural damage on building level.

The relevant damage grade for the parametric study is DG3 associated with life safety. The definition of DG3 corresponds to the recommendation given in DIN EN 1998-1/NA [7], NCI 4.3.3.4.2.1(1). For this reason, the nonlinear static verifications of the parametric studies are carried out taking into account load bearing reserves as long as the first wall fails. However, to be on the safe side, the descending branch after the first wall failure was not considered for the derivation of the new design concept in WP3.

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DIBt-Project

Report WP2-RWTH

Selection and definition of representative masonry buildings

Improved seismic design concepts for masonry buildings in Germany

(Verbesserte seismische Nachweiskonzepte für Mauerwerksbauten in Deutschland)

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1 OVERVIEW

This report includes the definition and modelling of representative masonry buildings in Germany, which are further used as a basis for the computational parametric studies in WP3. A terraced, single-family and multi-family house with continuous sliding shear walls and variable number of floors are considered for the brick types made of clay, calcium silicate, autoclaved aerated concrete and lightweight concrete. The buildings are selected with respect to the building practise and a previous parametric study conducted in 2009 [9]. Annex 1 contains a collection of further typical ground plan configurations from the building practise, which are comparable to the chosen building configurations. Hence, the chosen building configurations can be regarded as representative and their calculation and verification results can be further used to derive modifications of the linear elastic design concept.

1.1 Overview: parametric study

The following table summarizes the building configurations used in the parametric study. For each building type different number of stories are considered.

Table 1-1 Overview of the parametric study

Building	Building type	Stories	Material
TH-CB-1	Terraced house	1	Clay brick
TH-CB-2	Terraced house	2	Clay brick
FH-CB-2	Single-family house	2	Clay brick
FH-CB-3	Single-family house	3	Clay brick
MFH-CB-2	Multi-family house	2	Clay brick
MFH-CB-3	Multi-family house	3	Clay brick
MFH-CB-4	Multi-family house	4	Clay brick
TH-CS-1	Terraced house	1	Calcium silicate
TH-CS-2	Terraced house	2	Calcium silicate
FH-CS-2	Single-family house	2	Calcium silicate
FH-CS-3	Single-family house	3	Calcium silicate
MFH-CS-2	Multi-family house	2	Calcium silicate
MFH-CS-3	Multi-family house	3	Calcium silicate
MFH-CS-4	Multi-family house	4	Calcium silicate
TH-AC-1	Terraced house	1	Autoclaved aerated concrete
TH-AC-2	Terraced house	2	Autoclaved aerated concrete
FH-AC-2	Single-family house	2	Autoclaved aerated concrete
FH-AC-3	Single-family house	3	Autoclaved aerated concrete
MFH-AC-2	Multi-family house	2	Autoclaved aerated concrete
MFH-AC-3	Multi-family house	3	Autoclaved aerated concrete
MFH-AC-4	Multi-family house	4	Autoclaved aerated concrete
TH-LWC-1	Terraced house	1	Lightweight concrete
TH-LWC-2	Terraced house	2	Lightweight concrete
FH-LWC-2	Single-family house	2	Lightweight concrete
FH-LWC-3	Single-family house	3	Lightweight concrete
MFH-LWC-2	Multi-family house	2	Lightweight concrete
MFH-LWC-3	Multi-family house	3	Lightweight concrete
MFH-LWC-4	Multi-family house	4	Lightweight concrete

1.2 Overview: material properties

The required material and strength properties for the linear and nonlinear calculations are collected in Table 1-2. The nomenclature corresponds to the material and strength properties according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3]. The characteristic material parameters were checked and the relationship between characteristic and mean values were estimated based on statistical by using the following relationships for most of the materials: $f = 1.2 \cdot f_k$ and $f_{vo} = 1.25 \cdot f_{vk0}$. All calculations are carried out either with mean values or characteristic values. The scatters of the material parameters were not taken into account.

Table 1-2 Material properties and corresponding symbols

Material and strength properties	Symbol	Unit
Mean compressive strength of a masonry unit	f_{st}	[N/mm^2]
Compressive strength of masonry mortar	f_m	[N/mm^2]
Calculated tensile strength of a masonry unit	$f_{bt,cal}$	[N/mm^2]
Mean compressive strength of masonry	f	[N/mm^2]
Characteristic compressive strength of masonry	f_k	[N/mm^2]
Mean initial shear strength of masonry	f_{v0}	[N/mm^2]
Characteristic initial shear strength of masonry	f_{vk0}	[N/mm^2]
Density of masonry / concrete	ρ	[kg/m^3]
Poisson's ratio	μ	[-]
Young's modulus of masonry	E_M	[N/mm^2]
Thermal expansion coefficient	α_T	[-]
Cross section of concrete	A	[mm^2]
Secant modulus of concrete	E_{cm}	[N/mm^2]
Characteristic compressive strength of concrete	f_{ck}	[N/mm^2]

2 GENERAL ASSUMPTIONS

2.1 Seismic masses

The seismic storey masses of the buildings are calculated from the floor dead and live loads and the weight of the masonry walls. DIN EN 1998-1 [4], [5] differentiates between independently occupied storeys and storeys with correlated occupancies reflected by a factor φ varying with respect to the load categories of DIN EN 1991-1-1 [6]. Corresponding to the use as residential building, each storey is independent and the factor φ is applied with 1.0 for the roof and with 0.7 for all other storeys according to DIN EN 1998-1/NA [5].

The combination factor ψ_2 is defined in accordance with DIN EN 1990 [5] except for snow loads. In contrast to DIN EN 1990 [5], the snow loads are combined with a combination factor of ψ_2 = 0.5 according to DIN EN 1998-1/NA [5]. For the parametric study, the snow load is calculated for a roof pitch of 40° in snow load zone II and at an altitude of 245 m determined according to DIN EN 1991-1-3/NA [1].

Table 2-1 provides the floor masses of dead and live loads with the corresponding combination coefficients. The mass of the roof for all investigated buildings is assumed to be smaller than 50 % of the mass of the subjacent storey. Therefore, the roofs are simply considered as additional masses on the upper storeys instead of additional independent storeys.

Table 2-1 Floor masses: Dead and live loads and combination coefficients

		Floors above ground floor	**Top floor**	**Roof structure**
Floor loads	Floor Area	A_{GFi}	A_{TF}	A_R
	Permanent loads	Reinforced concrete floor incl. floor cover	Reinforced concrete floor incl. floor cover	Span roof construction
		$g_k = 6$ KN/m²	$g_k = 7$ KN/m²	$g_k = 1.2$ KN/m²
	Variable loads	Including loads of partitions walls	Including loads of partitions walls	Snow loads
		$q_k = 2.7$ KN/m²	$q_k = 2.7$ KN/m²	$q_k = 0.45$ KN/m²
	φ - coefficient	0.7 [-]	1.0 [-]	1.0 [-]
	ψ_2 - coefficient	0.3 [-]	0.3 [-]	0.5 [-]
Wall loads	Wall heights	h (full storey height)	h /2 (half storey height)	-
	Material density masonry	ρ_{MW}	ρ_{MW}	-
	Wall weight	$G_{k,MW} = \sum_i \rho_{MW} \cdot l_i \cdot h_i \cdot t_i$	$G_{k,MW} = \sum_i \rho_{MW} \cdot \frac{l_i}{2} \cdot h_i \cdot t_i$	-
Total floor load	$\sum G_{ki}$	$A_{GF,i} \cdot 6\,kN/m^2 + G_{k,MW}$	$A_{TF} \cdot 7\,kN/m^2 + G_{k,MW}$	$A_R \cdot g_k$
	$\sum \varphi \cdot \psi_{2i} \cdot Q_{ki}$	$A_{GF,i} \cdot (2.7 \cdot 0.7 \cdot 0.3)$	$A_{TF} \cdot (2.7 \cdot 1.0 \cdot 0.3)$	$A_R \cdot (0.45 \cdot 1.0 \cdot 0.5)$
	$\sum G_{ki} + \sum \varphi \cdot \psi_{2i} \cdot Q_{ki}$	$A_{GF,i} \cdot 6\,kN/m^2 + G_{k,MW} +$ $A_{GF,i} \cdot (2.7 \cdot 0.7 \cdot 0.3)$	$A_{TF} \cdot 7\,kN/m^2 + G_{k,MW} +$ $A_{TF} \cdot (2.7 \cdot 1.0 \cdot 0.3)$	$A_R \cdot g_k +$ $A_R \cdot (0.45 \cdot 1.0 \cdot 0.5)$

2.2 Seismic design situation and safety verifications

The design value E_{dAE} of the effects of actions in the seismic design situation is determined in accordance with DIN EN 1990 [6] as presented in Report 1-RWTH [8], Section 2.4:

$$E_{dAE} = E\left\{\sum_{j\geq 1} G_{k,j} \oplus A_{Ed} \oplus \sum_{i\geq 1} \psi_{2,i} \cdot Q_{k,i}\right\}$$

The combination coefficient $\psi_{2,i}$ for quasi-permanent values of a variable action *i* is 0.3 for variable loads and 0.5 for snow loads. The safety verifications for the single walls are carried out with the software MINEA-Research [19] according to DIN EN 1998-1/NA [5] and DIN EN 1996-1/NA [3]. The axial (vertical) loads on the walls are determined according to their tributary areas, shown for each of the investigated buildings in the following sections.

Based on the calculated design internal force resultants, safety verifications at the ultimate limit state for axial, bending and shear loading at the top, centre and bottom of the walls in the ground floor are processed. The verification procedures include reduction factors for the wall slenderness and for in-plane eccentricities and they also take the risk of buckling into account. The out-of-plane eccentricity is considered with the minimum value of 0.05·t for both, linear and nonlinear analysis according to DIN EN 1996-1/NA [3].

2.3 Design response spectrum

The design response spectrum is applied according to the recent draft of DIN EN 1998-1/NA [5] for the subsoil conditions C-R. The combination C-R is the most unfavourable subsoil condition with respect to the spectral shape and the soil amplification factor. For each calculation, the PGA-value is scaled up and down as long as a successful verification is achieved. Figure 2-1 shows the shape of the design response spectrum for subsoil conditions C-R for an exemplary reference ground acceleration of a_g = 1 m/s². The calculations neglect the first rising branch and the plateau is continued to the period T = 0s. As the initial stiffness of masonry structures is a complex issue and exhibit always a large scatter, it seems to be more reliable to elongate the plateau range.

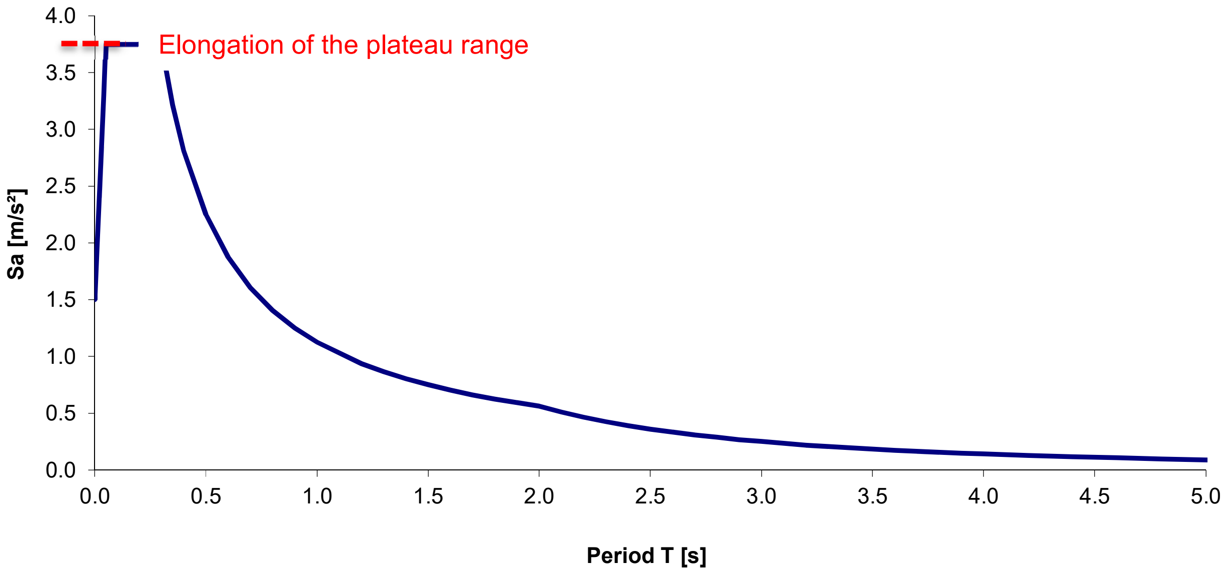

Figure 2-1 Design response spectrum for subsoil condition C-R, q = 1, a_g = 1.0

3 BUILDING 1: TH-CB

3.1 General information

The first building type TH-CB is a terraced house build of clay brick masonry. The ground plan is considered as single story as well as two-story building where the walls are expected to be continuous on all floors.

3.2 Structural layout

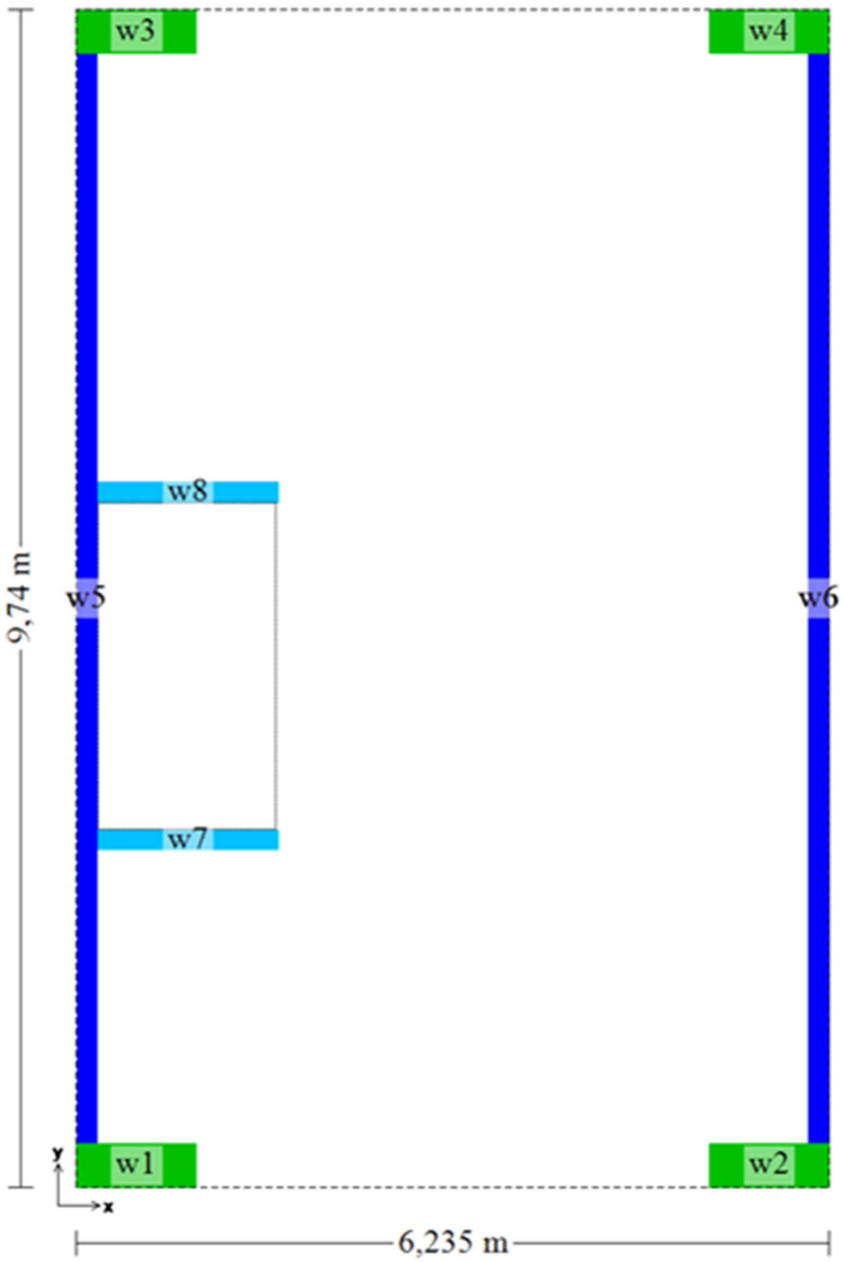

Figure 3-1 TH-CB: Structural walls – ground floor

3.3 Mechanical properties

The basic mechanical properties of the clay bricks, the mortar and the masonry have been provided by the Arbeitsgemeinschaft Mauerziegel im Bundesverband der Deutschen Ziegelindustrie e.V.

(ARGE-MZ). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type TH-CB is given in Table 3-1 and Table 3-2.

Table 3-1 TH-CB: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
CB-365-10-0,9-Nb.1		Clay brick, thin bed mortar	365	4840,00	0,10	900,00	1E-05
CB-175-12-0,9-Nb.2		Clay brick, thin bed mortar	175	4950,00	0,10	900,00	1E-05
CB-175-12-0,9-Nb.3		Clay brick, thin bed mortar	175	4950,00	0,10	900,00	1E-05

Table 3-2 TH-CB: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
CB-365-10-0,9-Nb.1		12,80	10,00	0,33	5,28	4,40	0,275	0,22
CB-175-12-0,9-Nb.2		15,00	10,00	0,39	5,00	4,50	0,275	0,22
CB-175-12-0,9-Nb.3		15,00	10,00	0,39	5,00	4,50	0,275	0,22

3.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

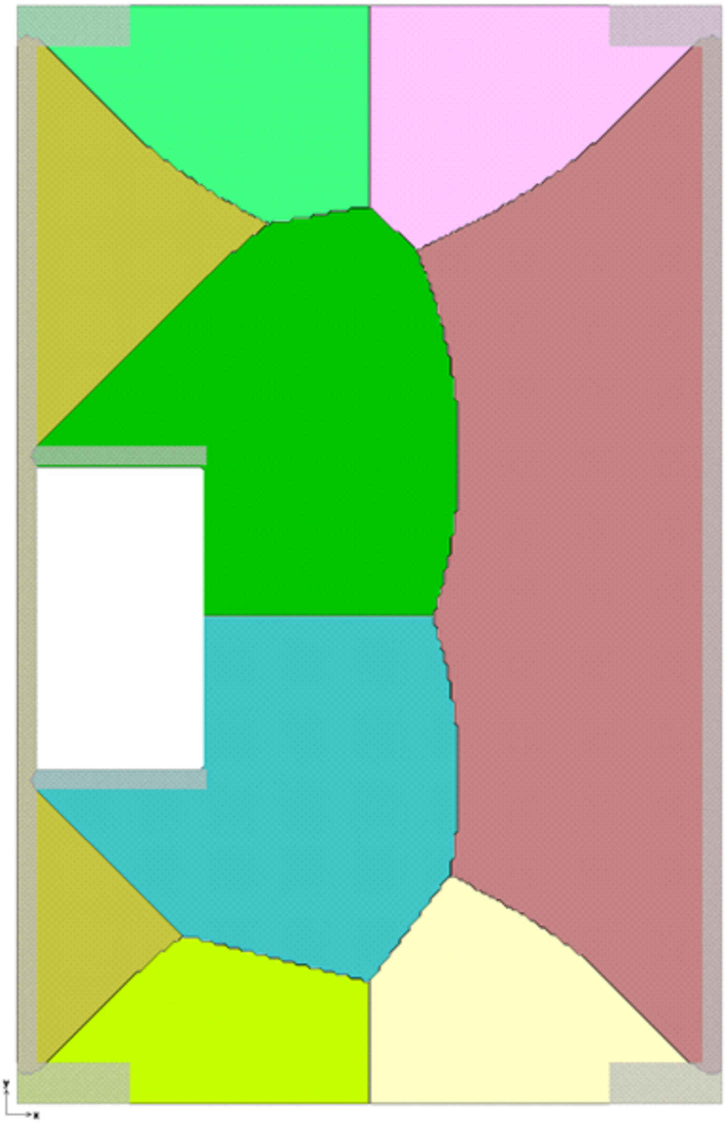

Figure 3-2 TH-CB: Tributary areas – ground floor

4 BUILDING 2: FH-CB

4.1 General information

The second building type FH-CB is a one family house build of clay brick masonry. The ground plan is considered as two- and three-story building where the walls are expected to be continuous on all floors.

4.2 Structural layout

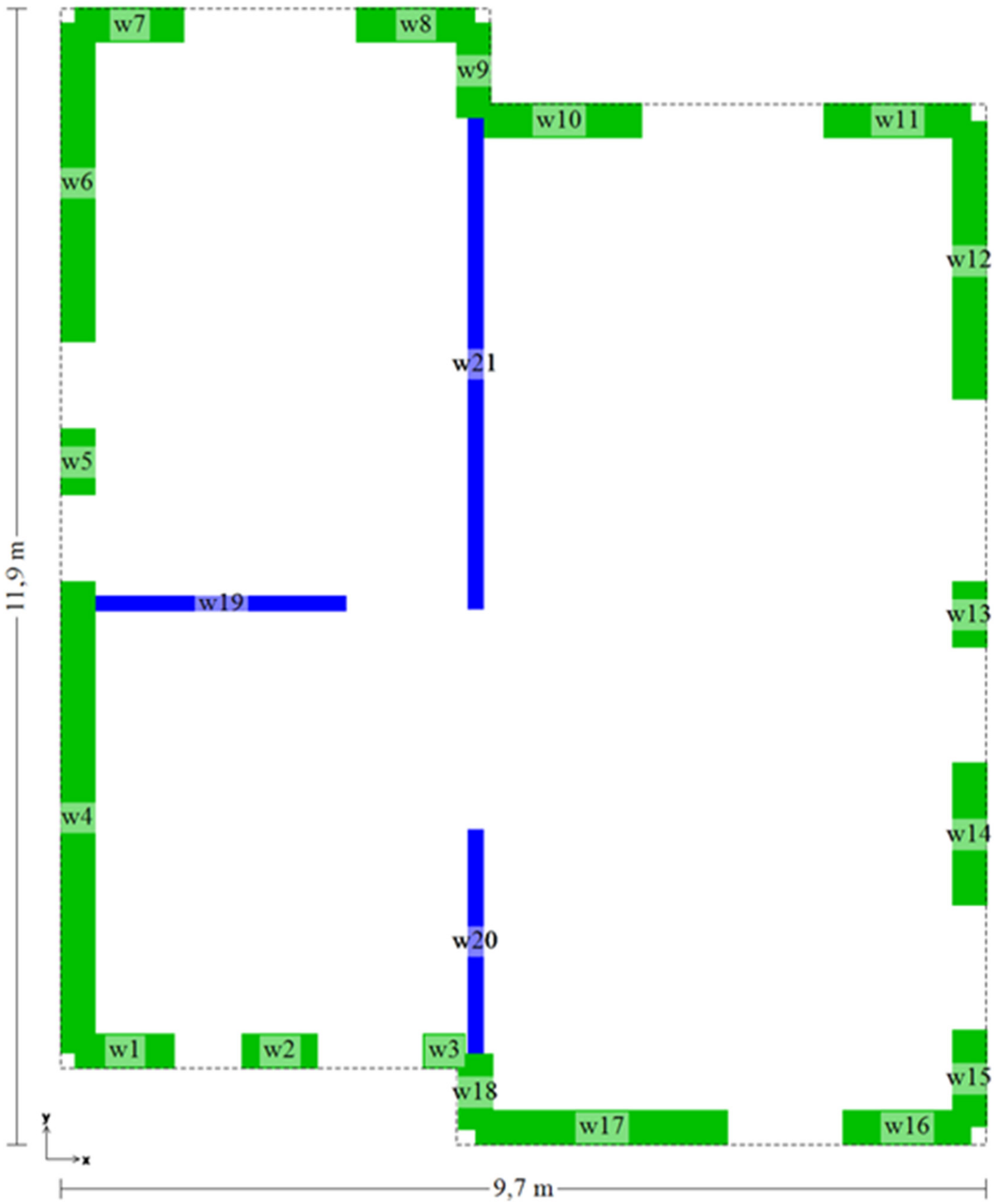

Figure 4-1 FH-CB: Structural walls – ground floor

4.3 Mechanical properties

The basic mechanical properties of the clay bricks, the mortar and the masonry have been provided by the Arbeitsgemeinschaft Mauerziegel im Bundesverband der Deutschen Ziegelindustrie e.V. (ARGE-MZ). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type FH-CB is given in Table 4-1 and Table 4-2.

Table 4-1 FH-CB: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	E_M	**μ**	**ρ**	α_T
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
CB-365-10-0,9-Nb.1		Clay brick, thin bed mortar	365	4840,00	0,10	900,00	1E-05
CB-175-12-0,9-Nb.3		Clay brick, thin bed mortar	175	4950,00	0,10	900,00	1E-05

Table 4-2 FH-CB: Strength parameters of different wall types (2 / 2)

Wall type		f_{st}	f_m	$f_{bt,cal}$	f	f_k	f_{v0}	f_{vk0}
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
CB-365-10-0,9-Nb.1		12,80	10,00	0,33	5,28	4,40	0,275	0,22
CB-175-12-0,9-Nb.3		15,00	10,00	0,39	5,00	4,50	0,275	0,22

4.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

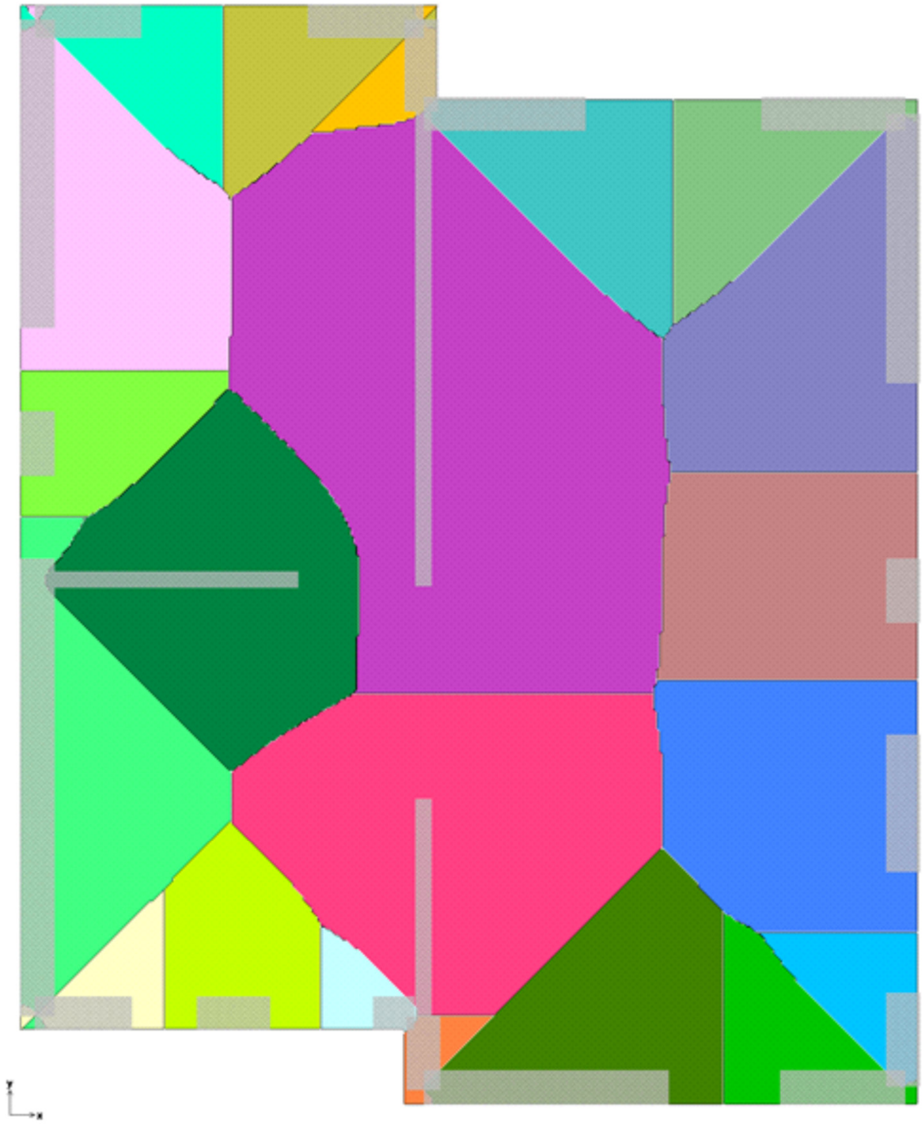

Figure 4-2 FH-CB: Tributary areas – ground floor

5 BUILDING 3: MFH-CB

5.1 General information

The third building type MFH-CB is a multi-family house build of clay brick masonry. The ground plan is considered as two-, three- and four-story building where the walls are expected to be continuous on all floors.

5.2 Structural layout

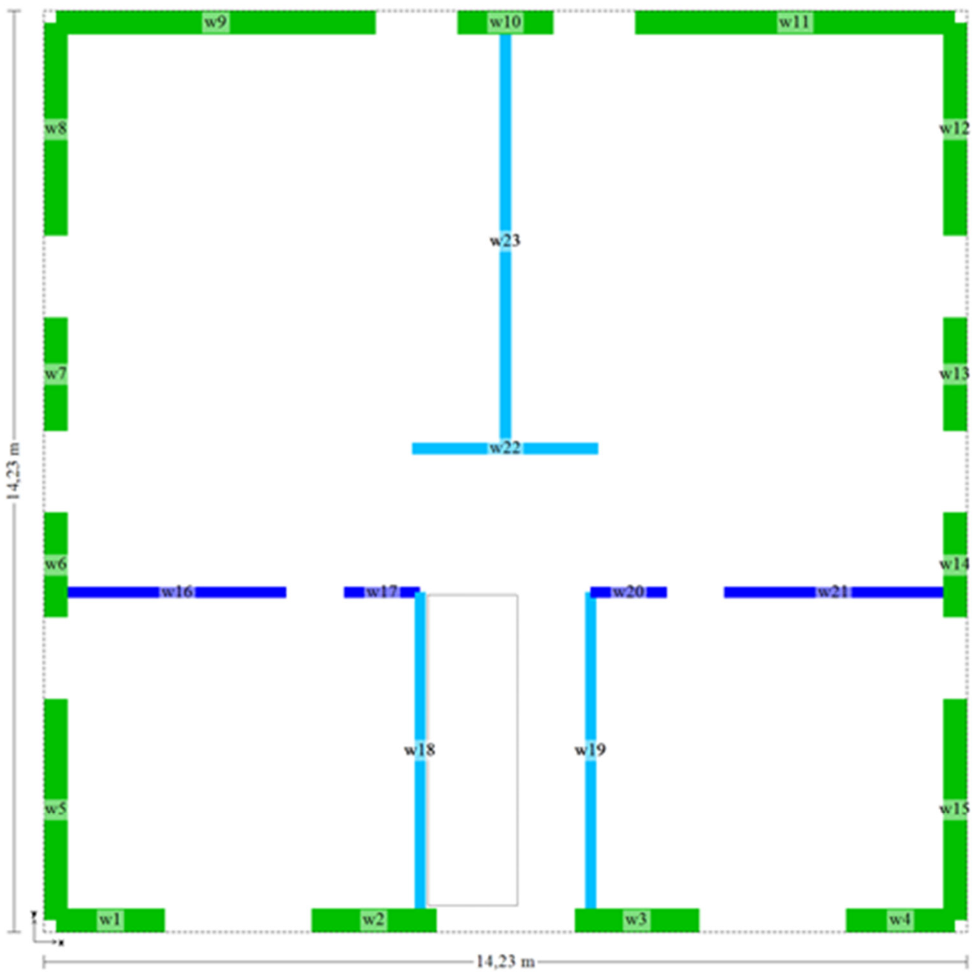

Figure 5-1 MFH-CB: Structural walls – ground floor

5.3 Mechanical properties

The basic mechanical properties of the clay bricks, the mortar and the masonry have been provided by the Arbeitsgemeinschaft Mauerziegel im Bundesverband der Deutschen Ziegelindustrie e.V. (ARGE-MZ). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type MFH-CB is given in Table 5-1 and Table 5-2.

Table 5-1 MFH-CB: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
CB-365-10-0,9-Nb.1		Clay brick, thin bed mortar	365	4840,00	0,10	900,00	1E-05
CB-175-12-0,9-Nb.3		Clay brick, thin bed mortar	175	4950,00	0,10	900,00	1E-05
CB-175-12-0,9-Nb.2		Clay brick, thin bed mortar	175	4950,00	0,10	900,00	1E-05

Table 5-2 MFH-CB: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
CB-365-10-0,9-Nb.1		12,80	10,00	0,33	5,28	4,40	0,275	0,22
CB-175-12-0,9-Nb.3		15,00	10,00	0,39	5,00	4,50	0,275	0,22
CB-175-12-0,9-Nb.2		15,00	10,00	0,39	5,00	4,50	0,275	0,22

5.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

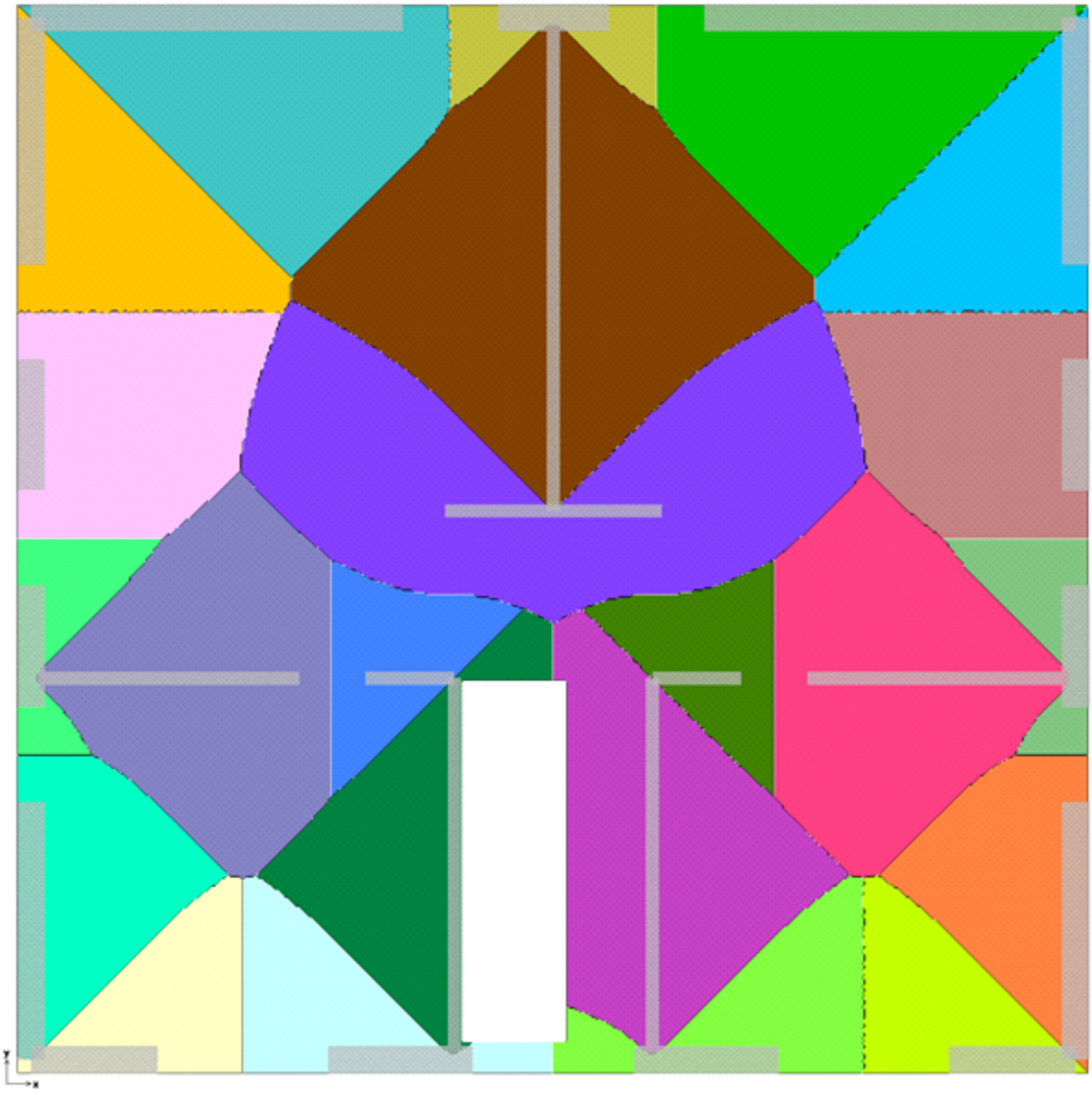

Figure 5-2 MFH-CB: Tributary areas – ground floor

6 BUILDING 4: TH-CS

6.1 General information

The fourth building type TH-CS is a terraced house build of calcium silicate masonry. The ground plan is considered as single story as well as two-story building where the walls are expected to be continuous on all floors.

6.2 Structural layout

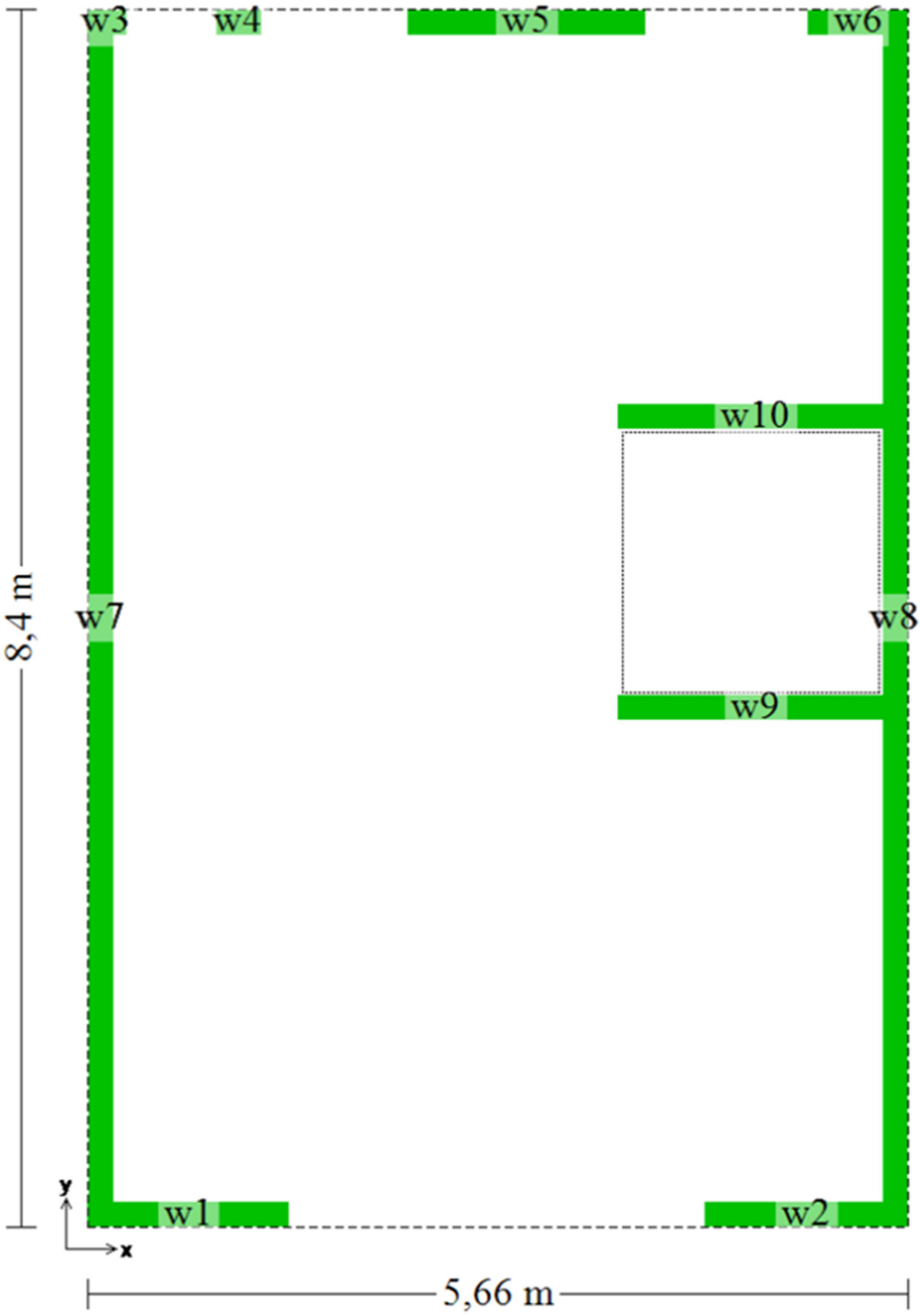

Figure 6-1 TH-CS: Structural walls – ground floor

6.3 Mechanical properties

The basic mechanical properties of the calcium silicate bricks, the mortar and the masonry have been provided by the Bundesverband Kalksandsteinindustrie e.V. (BV-KS). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type TH-CS is given in Table 6-1 and Table 6-2.

Table 6-1 TH-CS: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	E_M	**μ**	**ρ**	α_T
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
CS-175-12-1,5-Nb.4		Calcium silicate brick, thin bed mortar	175	5320,00	0,10	1500,0	1E-05

Table 6-2 TH-CS: Strength parameters of different wall types (2 / 2)

Wall type		f_{st}	f_m	$f_{bt,cal}$	**f**	f_k	f_{v0}	f_{vk0}
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
CS-175-12-1,5-Nb.4		15,00	10,00	0,30	7,00	5,60	0,31	0,22

6.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

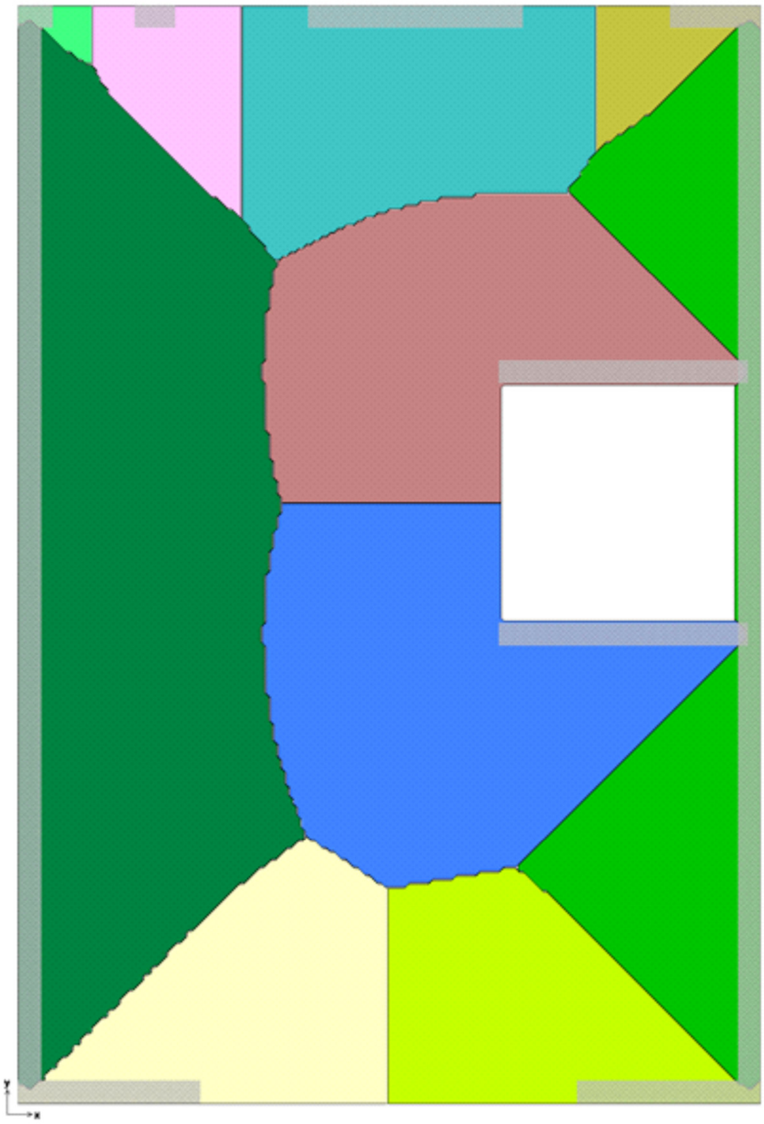

Figure 6-2 TH-CS: Tributary areas – ground floor

7 BUILDING 5: FH-CS

7.1 General information

The fifth building type FH-CS is a one family house build of calcium silicate masonry. The ground plan is considered as two- and three-story building where the walls are expected to be continuous on all floors.

7.2 Structural layout

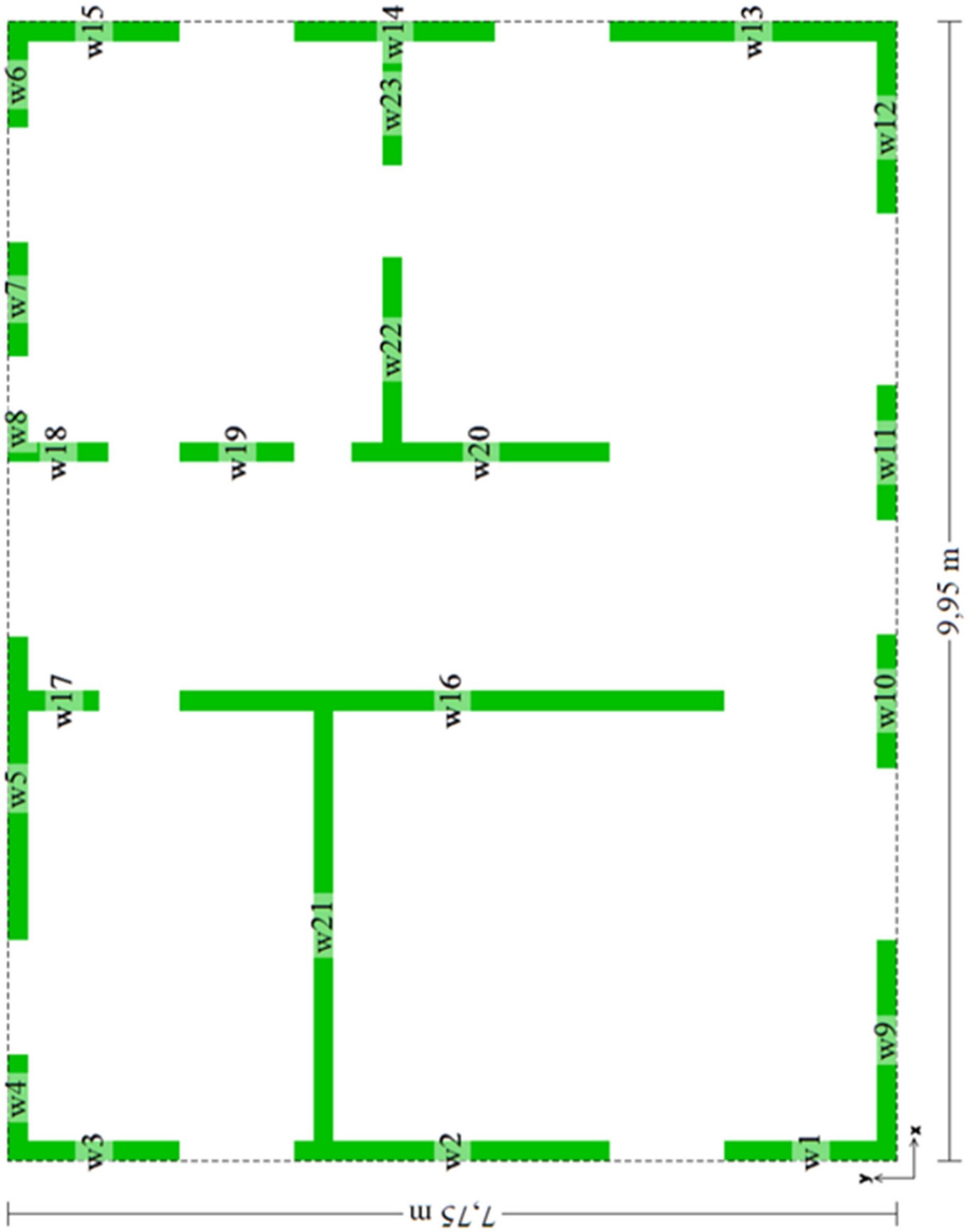

Figure 7-1 FH-CS: Structural walls – ground floor

7.3 Mechanical properties

The basic mechanical properties of the calcium silicate bricks, the mortar and the masonry have been provided by the Bundesverband Kalksandsteinindustrie e.V. (BV-KS). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type FH-CS is given in Table 7-1 and Table 7-2.

Table 7-1 FH-CS: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm^2]	[-]	[kg/m^3]	[-]
CS-175-12-1,5-Nb.4		Calcium silicate brick, thin bed mortar	175	5320,00	0,10	1500,0	1E-05

Table 7-2 FH-CS: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]
CS-175-12-1,5-Nb.4		15,00	10,00	0,30	7,00	5,60	0,31	0,22

7.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

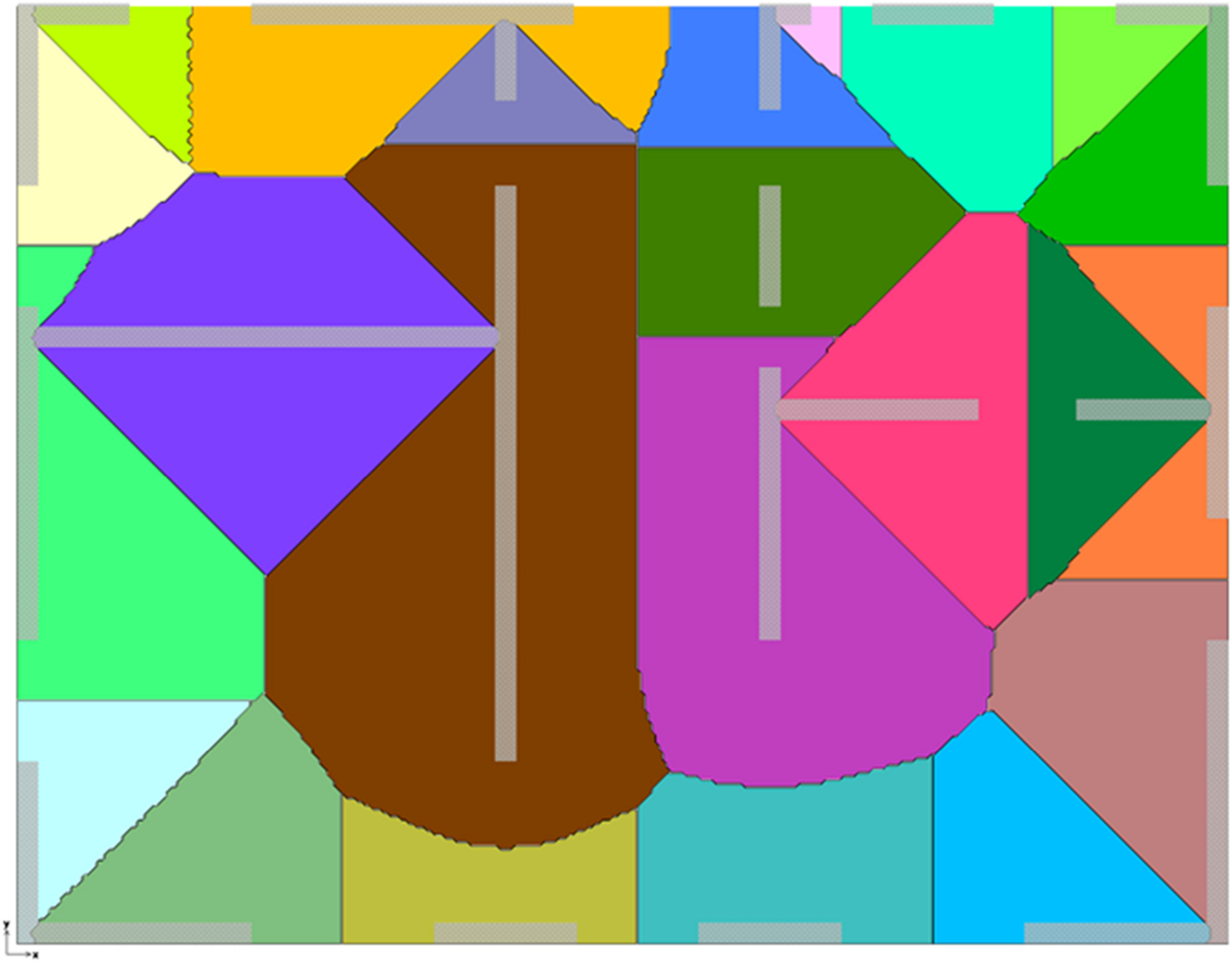

Figure 7-2 FH-CS: Tributary areas – ground floor

8 BUILDING 6: MFH-CS

8.1 General information

The sixth building type MFH-CS is a multi-family house build of calcium silicate masonry. The ground plan is considered as two-, three- and four-story building where the walls are expected to be continuous on all floors.

8.2 Structural layout

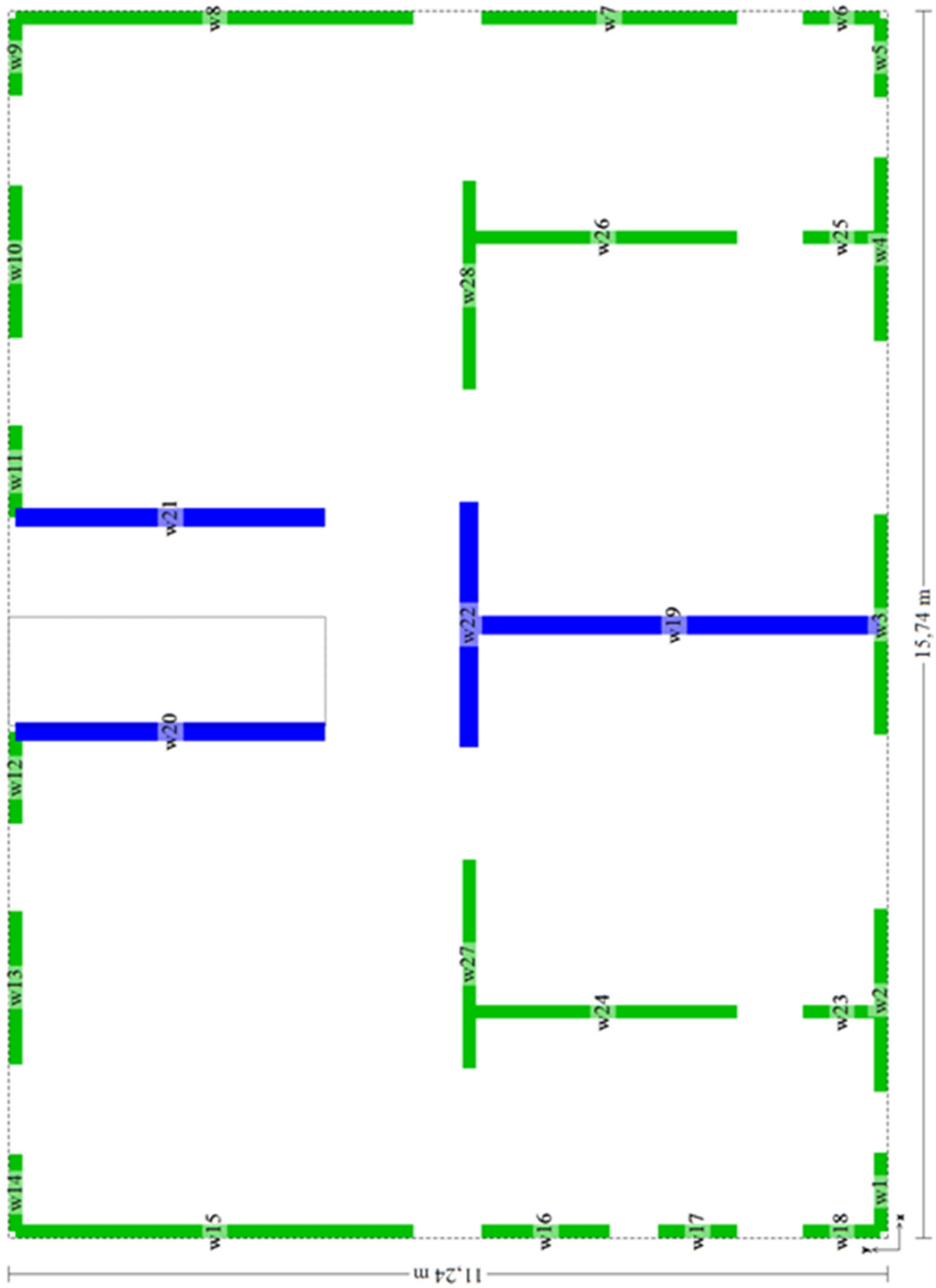

Figure 8-1 MFH-CS: Structural walls – ground floor

8.3 Mechanical properties

The basic mechanical properties of the calcium silicate bricks, the mortar and the masonry have been provided by the Bundesverband Kalksandsteinindustrie e.V. (BV-KS). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type FH-CS is given in Table 8-1 and Table 8-2.

Table 8-1 MFH-CS: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm^2]	[-]	[kg/m^3]	[-]
CS-175-20-2,0-Nb.6		Calcium silicate brick, thin bed mortar	175	12255,00	0,10	2000,0	1E-05
CS-240-20-2,0-Nb.5		Calcium silicate brick, thin bed mortar	240	12255,00	0,10	2000,0	1E-05

Table 8-2 MFH-CS: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]
CS-175-20-2,0-Nb.6		25,00	10,00	0,80	16,10	12,90	0,31	0,22
CS-240-20-2,0-Nb.5		25,00	10,00	0,80	16,10	12,90	0,31	0,22

8.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

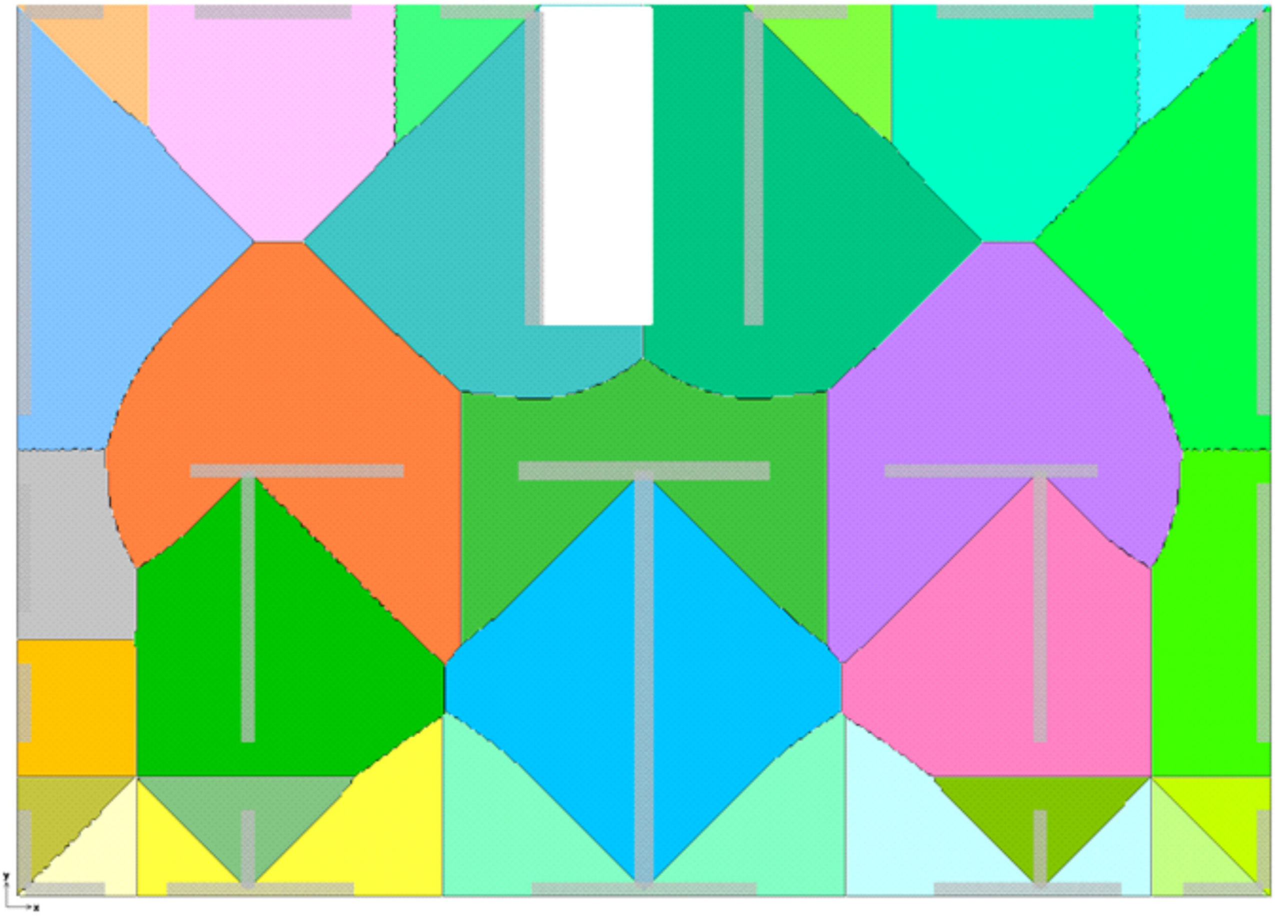

Figure 8-2 MFH-CS: Tributary areas – ground floor

9 BUILDING 7: TH-AC

9.1 General information

The seventh building type TH-AC is a terraced house build of autoclaved aerated concrete masonry. The ground plan is considered as single story as well as two-story building where the walls are expected to be continuous on all floors.

9.2 Structural layout

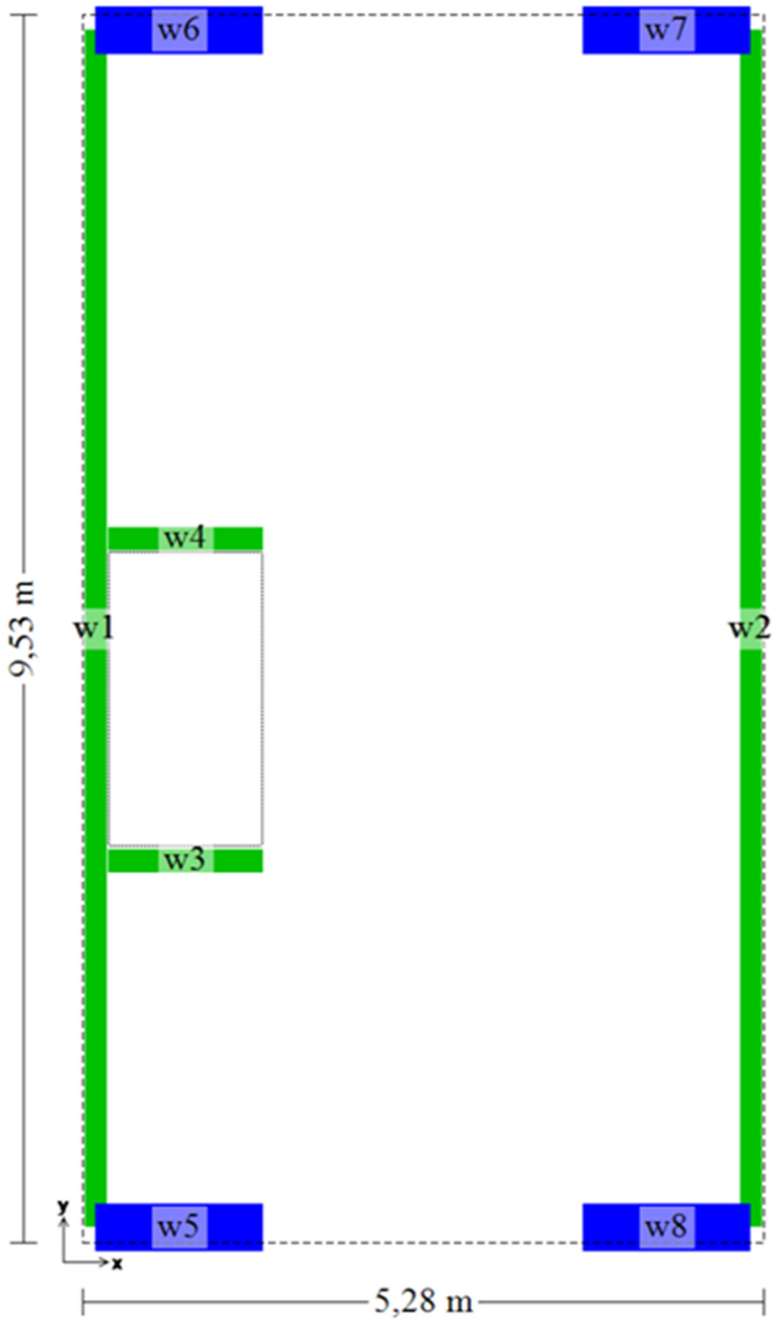

Figure 9-1 TH-AC: Structural walls – ground floor

9.3 Mechanical properties

The basic mechanical properties of the autoclaved aerated concrete bricks, the mortar and the masonry have been provided by the Bundesverband Porenbetonindustrie e.V. (BV-PB). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type TH-AC is given in Table 9-1 and Table 9-2.

Table 9-1 TH-AC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
AC-175-4-0,6-Nb.8		Aerated concrete brick, thin bed mortar	175	1320,00	0,10	600,00	1E-05
AC-365-2-0,4-Nb.7		Aerated concrete brick, thin bed mortar	365	792,00	0,10	400,00	1E-05

Table 9-2 TH-AC: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
AC-175-4-0,6-Nb.8		5,00	10,00	0,16	3,00	2,40	0,22	0,18
AC-365-2-0,4-Nb.7		2,50	10,00	0,08	1,80	1,44	0,22	0,18

9.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

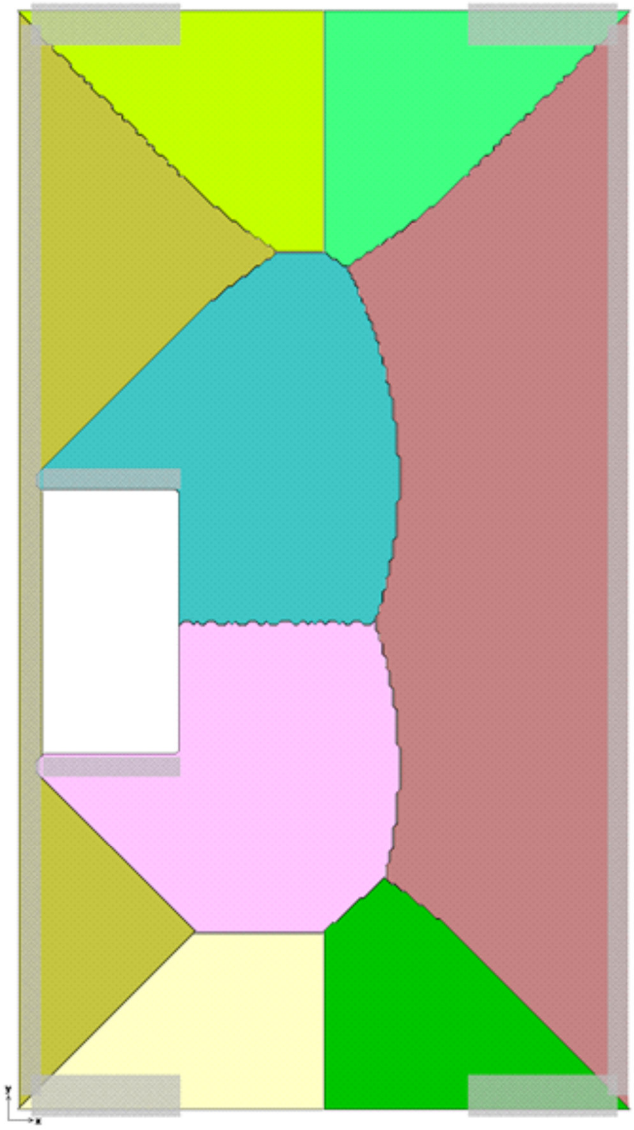

Figure 9-2 TH-AC: Tributary areas – ground floor

10 BUILDING 8: FH-AC

10.1 General information

The eighth building type FH-AC is a one family house build of autoclaved aerated concrete masonry. The ground plan is considered as two- and three-story building where the walls are expected to be continuous on all floors.

10.2 Structural layout

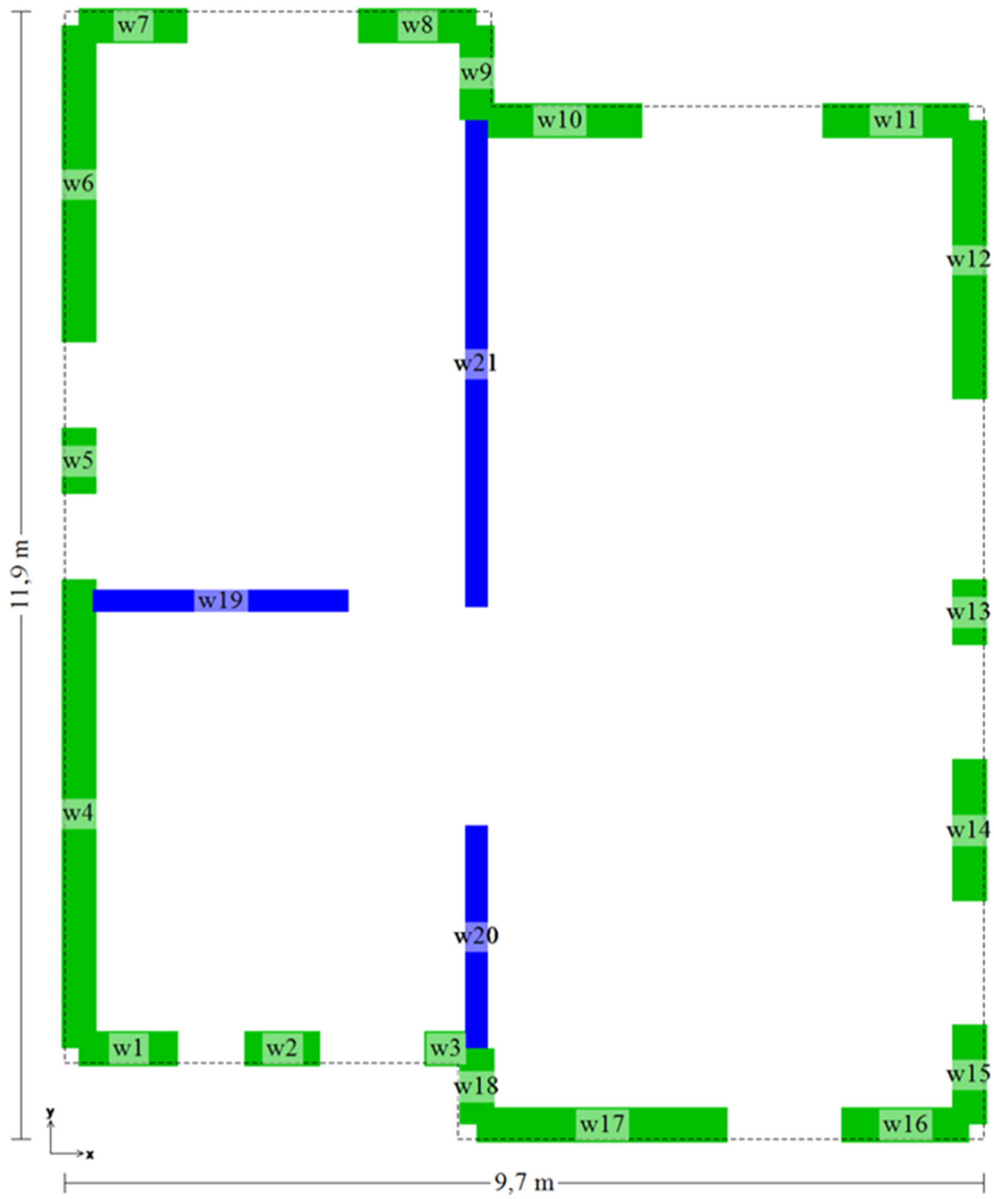

Figure 10-1 FH-AC: Structural walls – ground floor

10.3 Mechanical properties

The basic mechanical properties of the autoclaved aerated concrete bricks, the mortar and the masonry have been provided by the Bundesverband Porenbetonindustrie e.V. (BV-PB). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type FH-AC is given in Table 10-1 and Table 10-2.

Table 10-1 FH-AC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**T**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm^2]	[-]	[kg/m^3]	[-]
AC-365-2-0,4-Nb.7		Aerated concrete brick, thin bed mortar	365	792,00	0,10	400,00	1E-05
AC-240-4-0,6-Nb.8		Aerated concrete brick, thin bed mortar	240	1320,00	0,10	600,00	1E-05

Table 10-2 FH-AC: Material parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]
AC-365-2-0,4-Nb.7		2,50	10,00	0,08	1,80	1,44	0,22	0,18
AC-240-4-0,6-Nb.8		5,00	10,00	0,16	3,00	2,40	0,22	0,18

10.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

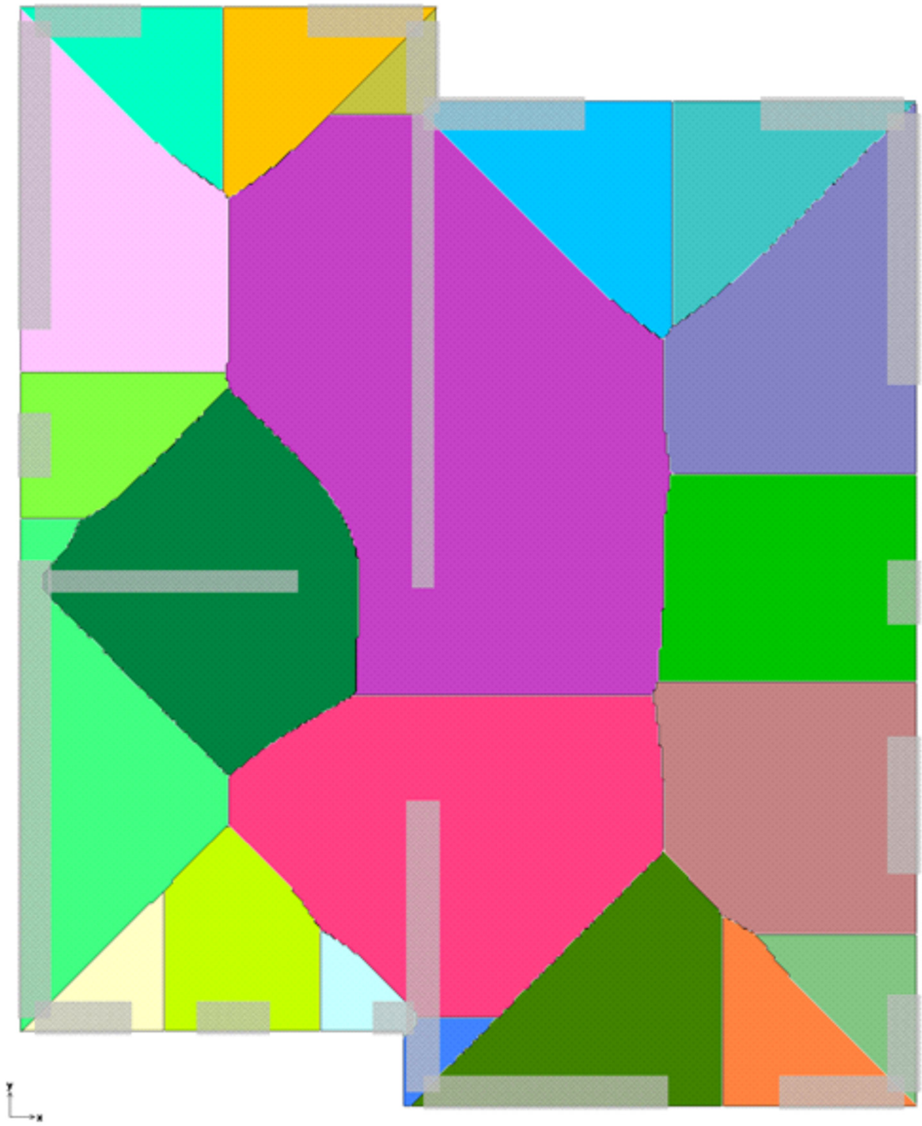

Figure 10-2 FH-AC: Tributary areas – ground floor

11 BUILDING 9: MFH-AC

11.1 General information

The ninth building type MFH-AC is a multi-family house build of autoclaved aerated concrete masonry. The ground plan is considered as two-, three- and four-story building where the walls are expected to be continuous on all floors.

11.2 Structural layout

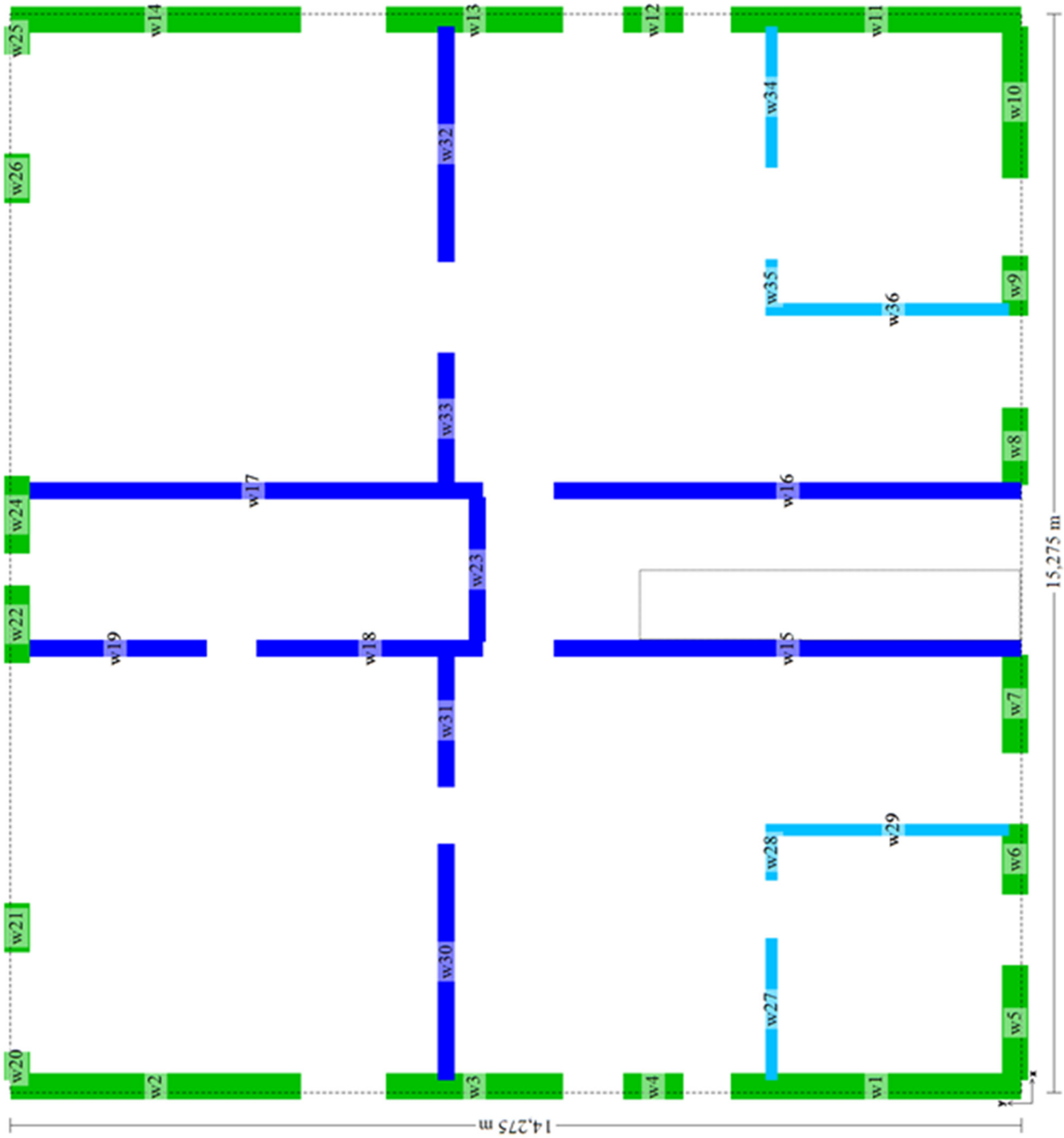

Figure 11-1 MFH-AC: Structural walls – ground floor

11.3 Mechanical properties

The basic mechanical properties of the autoclaved aerated concrete bricks, the mortar and the masonry have been provided by the Bundesverband Porenbetonindustrie e.V. (BV-PB). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type MFH-AC is given in Table 11-1 and Table 11-2.

Table 11-1 MFH-AC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
AC-365-2-0,4-Nb.7		Aerated concrete brick, thin bed mortar	365	792,00	0,10	400,00	1E-05
AC-240-4-0,6-Nb.9		Aerated concrete brick, thin bed mortar	240	1320,00	0,10	600,00	1E-05
AC-175-4-0,6-Nb.8		Aerated concrete brick, thin bed mortar	175	1320,00	0,10	600,00	1E-05

Table 11-2 MFH-AC: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
AC-365-2-0,4-Nb.7		2,50	10,00	0,08	1,80	1,44	0,22	0,18
AC-240-4-0,6-Nb.9		5,00	10,00	0,16	3,00	2,40	0,22	0,18
AC-175-4-0,6-Nb.8		5,00	10,00	0,16	3,00	2,40	0,22	0,18

11.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

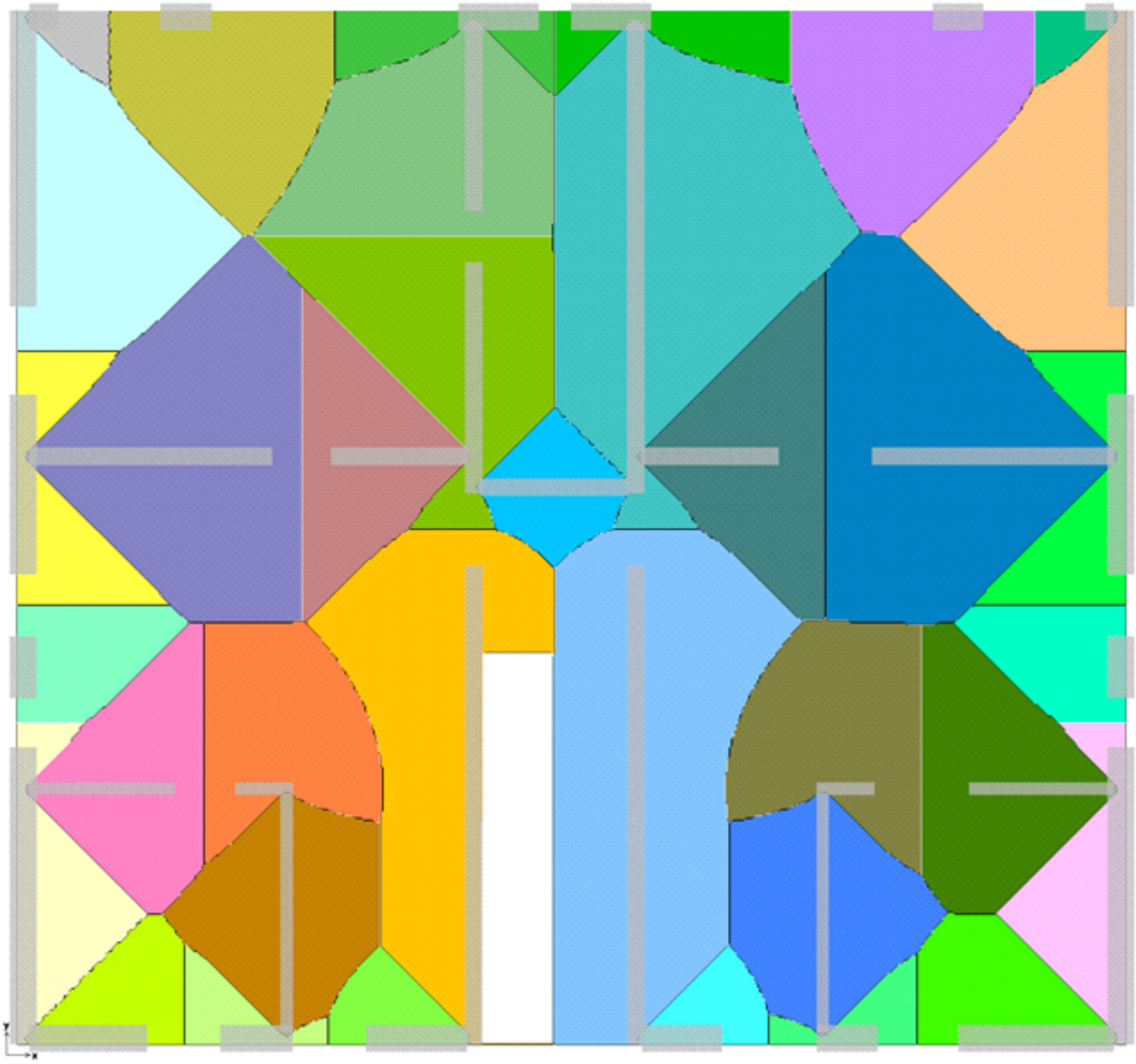

Figure 11-2 MFH-AC: Tributary areas – ground floor

12 BUILDING 10: TH-LWC

12.1 General information

The tenth building type TH-LWC is a terraced house build of light weight concrete masonry. The ground plan is considered as single story as well as two-story building where the walls are expected to be continuous on all floors.

12.2 Structural layout

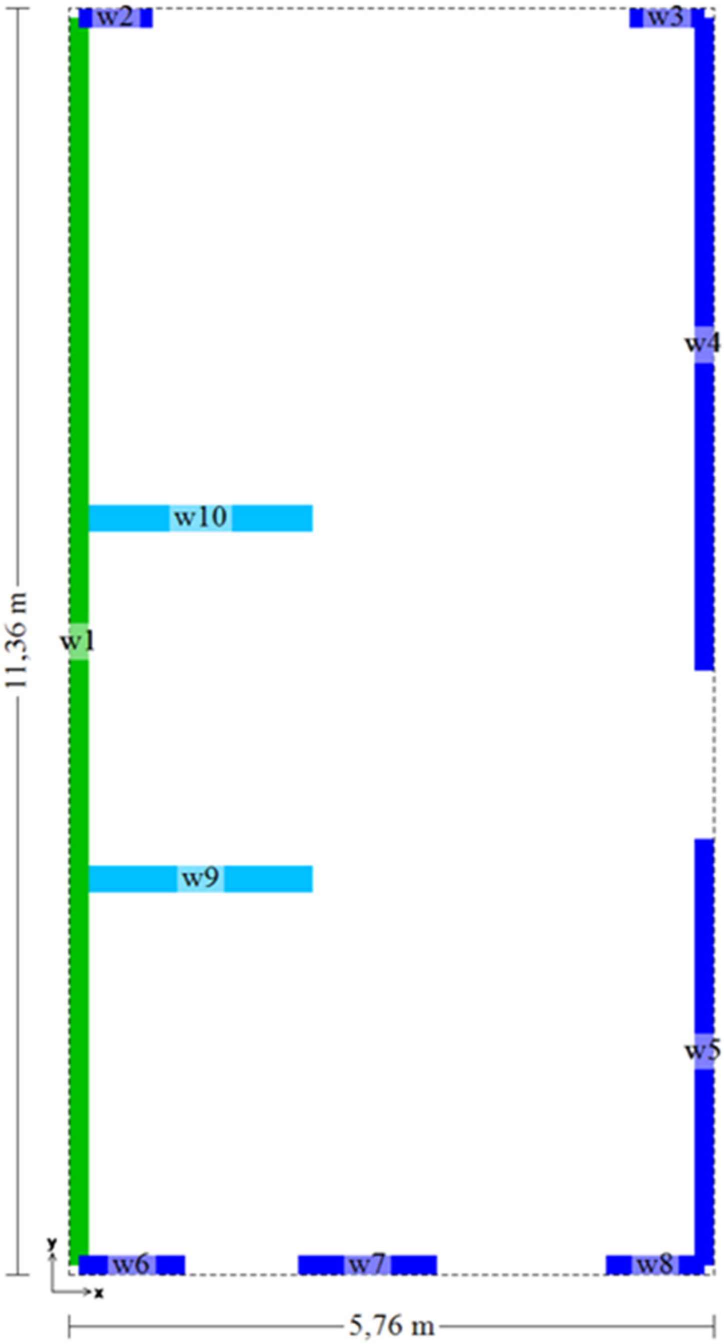

Figure 12-1 TH-LWC: Structural walls – ground floor

12.3 Mechanical properties

The basic mechanical properties of the lightweight concrete bricks, the mortar and the masonry have been provided by the Bundesverband Leichtbeton e.V. (BV-LB). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type TH-LWC is given in Table 12-1 and Table 12-2.

Table 12-1 TH-LWC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
LWC-175-20-2,0-Nb.13		Lightweight concrete brick, thin bed mortar	175	9500,00	0,10	2000,0	1E-05
LWC-175-2-0,45-Nb.12		Lightweight concrete brick, thin bed mortar	175	1425,00	0,10	450,00	1E-05
LWC-240-20-2,0-Nb.14		Lightweight concrete brick, thin bed mortar	240	9500,00	0,10	2000,0	1E-05

Table 12-2 TH-LWC: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
LWC-175-20-2,0-Nb.13		25,00	10,00	0,80	12,50	10,00	0,314	0,22
LWC-175-2-0,45-Nb.12		2,50	10,00	0,08	1,875	1,50	0,314	0,22
LWC-240-20-2,0-Nb.14		25,00	10,00	0,80	12,50	10,00	0,314	0,22

12.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

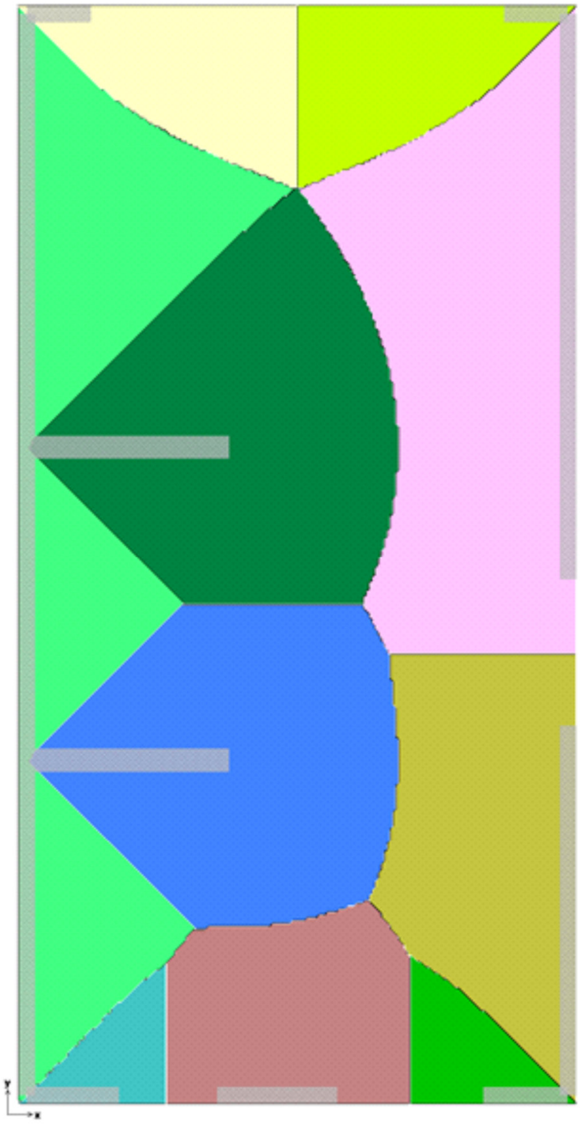

Figure 12-2 TH-LWC: Tributary areas – ground floor

13 BUILDING 11: FH-LWC

13.1 General information

The eleventh building type FH-LWC is a one family house build of light weight concrete masonry. The ground plan is considered as two- and three-story building where the walls as well as the columns are expected to be continuous on all floors.

13.2 Structural layout

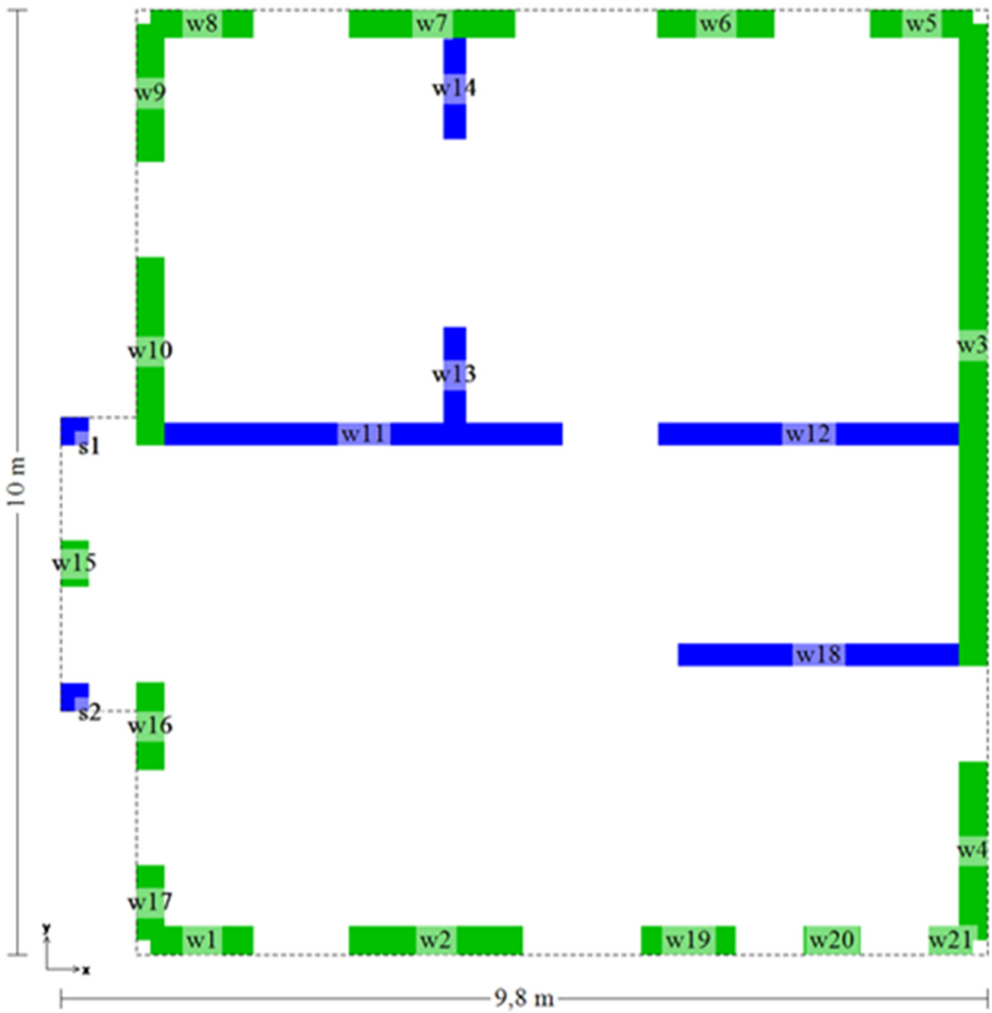

Figure 13-1 FH-LWC: Structural walls – ground floor

13.3 Mechanical properties

The basic mechanical properties of the lightweight concrete bricks, the mortar and the masonry have been provided by the Bundesverband Leichtbeton e.V. (BV-LB). Furthermore, the relationships according to DIN EN 1996-1-1 [2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type FH-LWC is given in Table 13-1 and Table 13-2.

Table 13-1 FH-LWC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**$α_T$**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
LWC-300-2-0,45-Nb.15		Light weight concrete brick, thin bed mortar	300	1425,00	0,10	450,0	1E-05
LWC-240-20-2,0-Nb.14		Light weight concrete brick, thin bed mortar	240	9500,00	0,10	2000,0	1E-05

Table 13-2 FH-LWC: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
LWC-300-2-0,45-Nb.15		2,50	10,00	0,08	1,875	1,50	0,314	0,22
LWC-240-20-2,0-Nb.14		25,00	10,00	0,80	12,50	10,00	0,314	0,22

Table 13-3 FH-LWC: Parameters of columns

Column type		**Material**	**A**	**E_{cm}**	**μ**	**ρ**	**$α_T$**	**f_{ck}**
			[mm²]	[N/mm²]	[-]	[kg/m³]	[-]	[N/mm²]
LWC		C25/30	90000,0	31000,0	0,17	2500,00	1E-05	25,00

13.4 Tributary areas

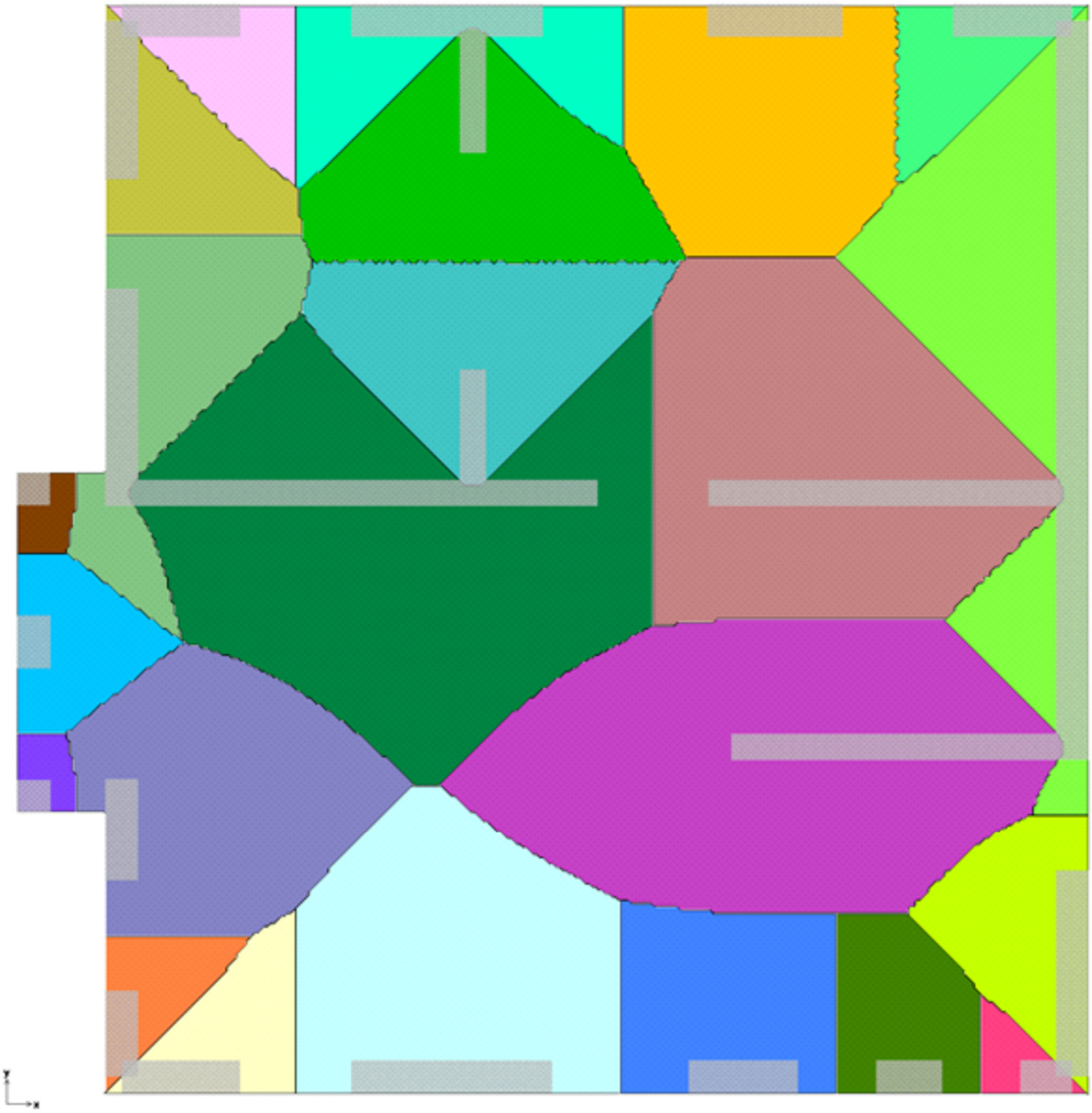

Figure 13-2 FH-LWC: Tributary areas – ground floor

14 BUILDING 12: MFH-LWC

14.1 General information

The twelfth building type MFH-LWC is a multi-family house build of light weight concrete masonry. The ground plan is considered as two-, three- and four-story building where the walls are expected to be continuous on all floors.

14.2 Structural layout

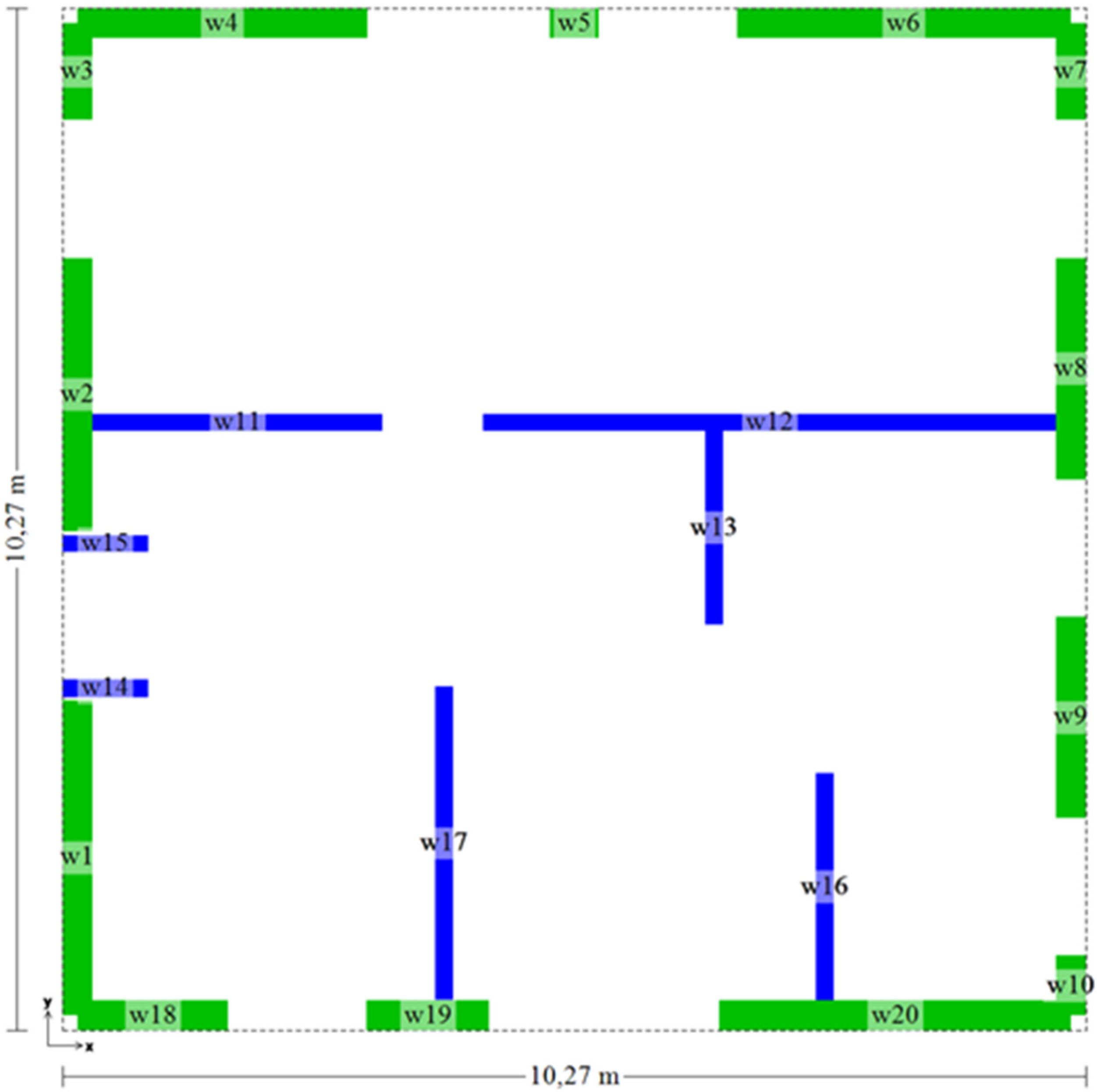

Figure 14-1 MFH-LWC: Structural walls – ground floor

14.3 Mechanical properties

The basic mechanical properties of the lightweight concrete bricks, the mortar and the masonry have been provided by the Bundesverband Leichtbeton e.V. (BV-LB). Furthermore, the relationships according to DIN EN 1996-11[2] and DIN EN 1996-1-1/NA [3] are considered. A summary of the material and strength parameters of the different wall types of the building type MFH-LWC is given in Table 14-1 and Table 14-2.

Table 14-1 MFH-LWC: Material parameters of different wall types (1 / 2)

Wall type		**Material**	**t**	**E_M**	**μ**	**ρ**	**α_T**
			[mm]	[N/mm²]	[-]	[kg/m³]	[-]
LWC-300-4-0,5-Nb.16		Light weight concrete brick, thin bed mortar	300	1425,00	0,10	500,00	1E-05
LWC-175-20-2,0-Nb.17		Light weight concrete brick, thin bed mortar	175	9500,00	0,10	2000,0	1E-05

Table 14-2 MFH-LWC: Strength parameters of different wall types (2 / 2)

Wall type		**f_{st}**	**f_m**	**$f_{bt,cal}$**	**f**	**f_k**	**f_{v0}**	**f_{vk0}**
		[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
LWC-300-4-0,5-Nb.16		5,00	10,00	0,10	1,875	1,50	0,314	0,22
LWC-175-20-2,0-Nb.17		25,00	10,00	0,80	12,50	10,00	0,314	0,22

14.4 Tributary areas

The tributary areas for determination of the vertical load level are depicted in the following figure.

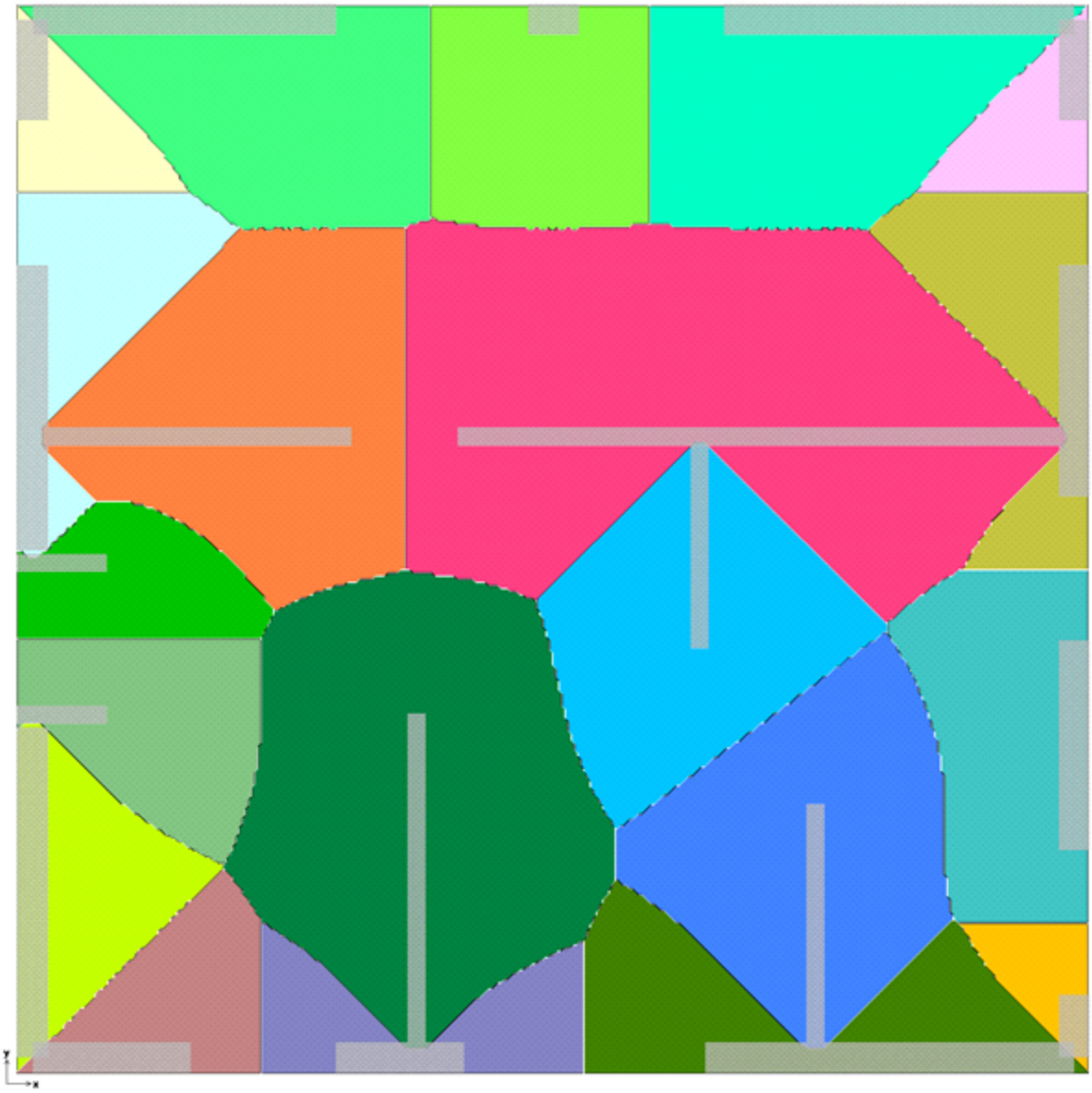

Figure 14-2 MFH-LWC: Tributary areas – ground floor

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[2] DIN EN 1996-1-1, 2013. Eurocode 6 - Design of masonry structures, Part 1-1: Common rules for reinforced and unreinforced masonry structures. Deutsches Institut für Normung e.V., Berlin.

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16 ANNEX 1: TYPICAL GROUND PLAN CONFIGURATIONS

16.1 Terraced houses

This section shows ground plan configurations of typical terraced houses in German earthquake regions. All examples are two-storey buildings with continuous shear walls over the building height and rigid floors. All terraced houses are characterised by strong shear walls in longitudinal direction, shorter wall sections in the external walls in transverse direction and two short walls along the staircase in transverse direction. This confirms the representative building configuration used for the parametric study.

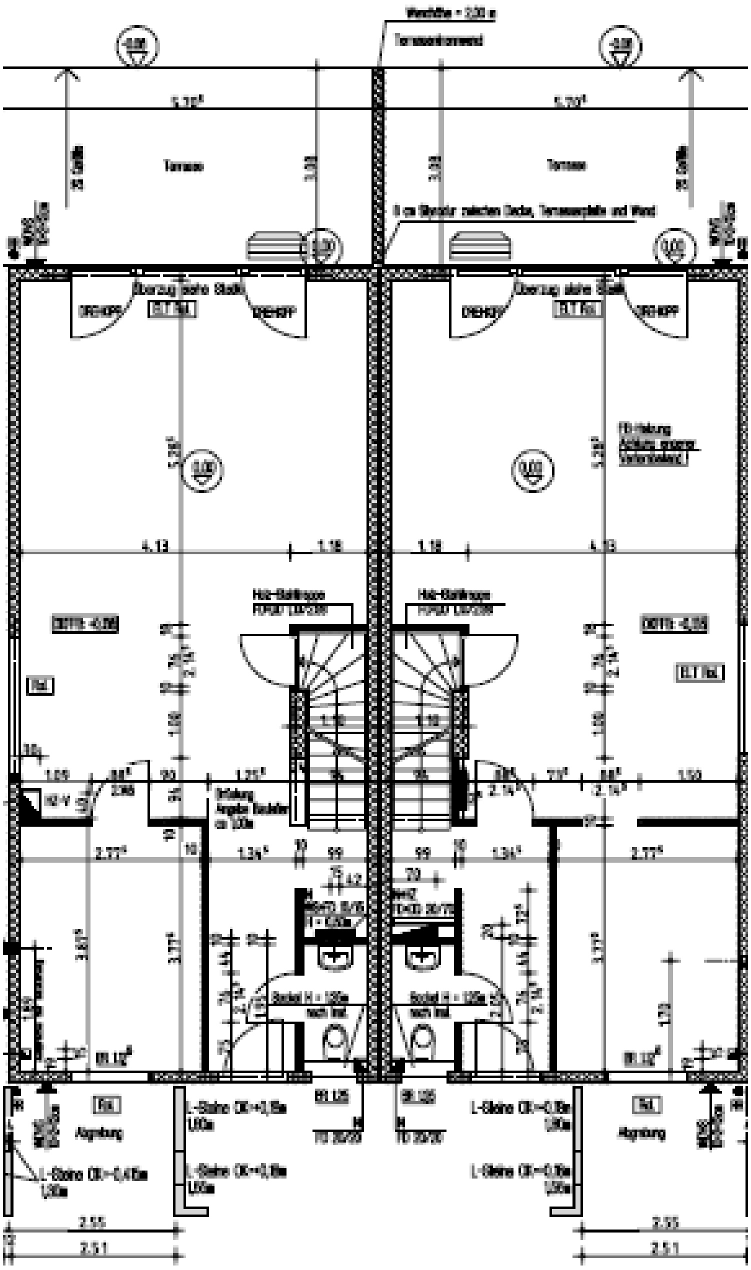


Figure 16-1: Terraced house, building location: Aachen, earthquake zone 3

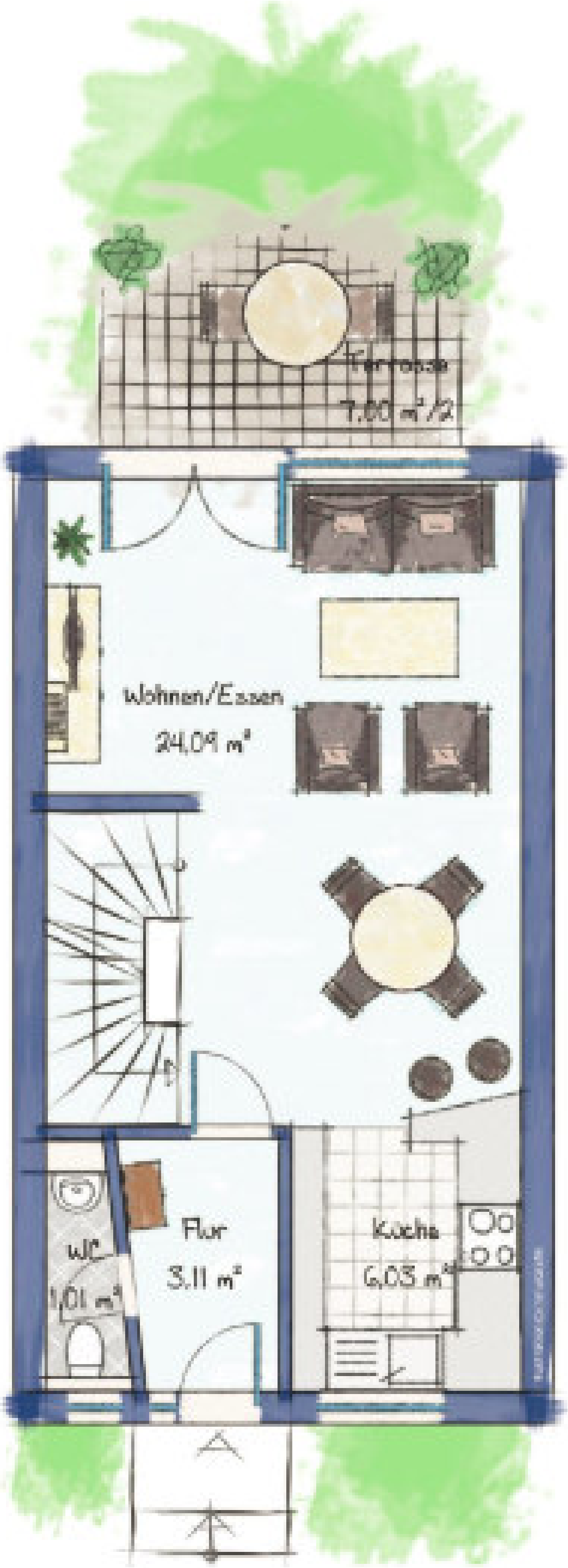


Figure 16-2: Terraced house, building location: Freiburg, earthquake zone 1, 2 [10]

Figure 16-3: Terraced house, building location: Freiburg, earthquake zone 1 [11]

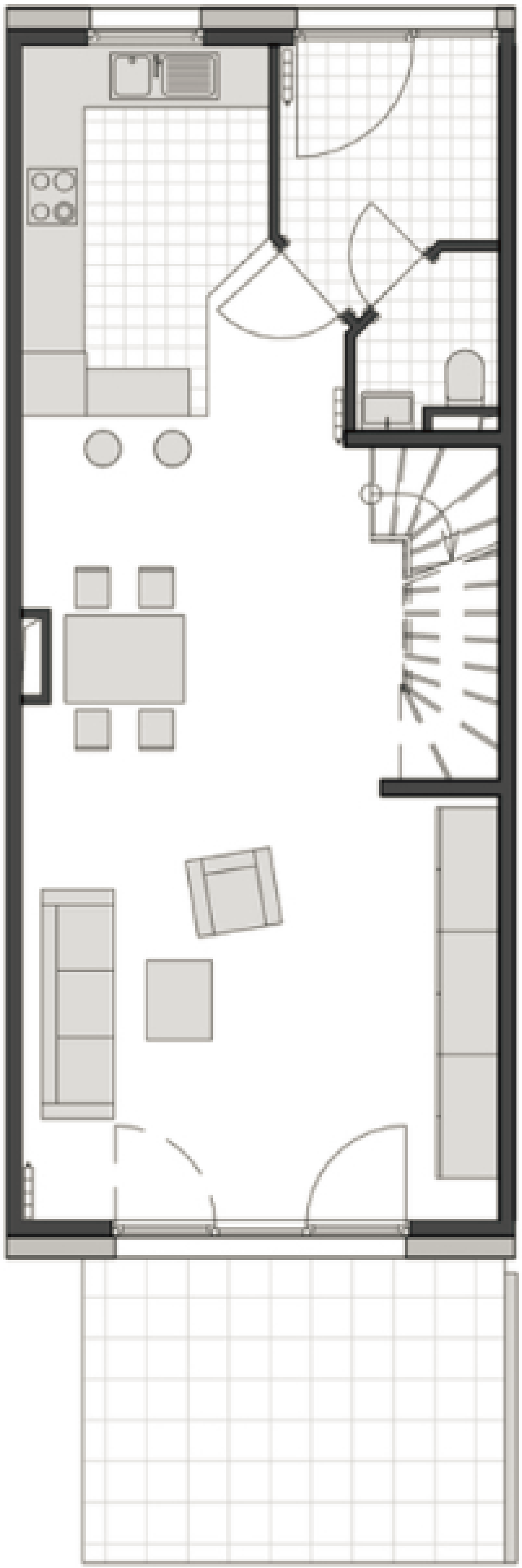


Figure 16-4: Terraced house, building location: Freiburg, Type RH 136 SD35, earthquake zone 1-3 [12]

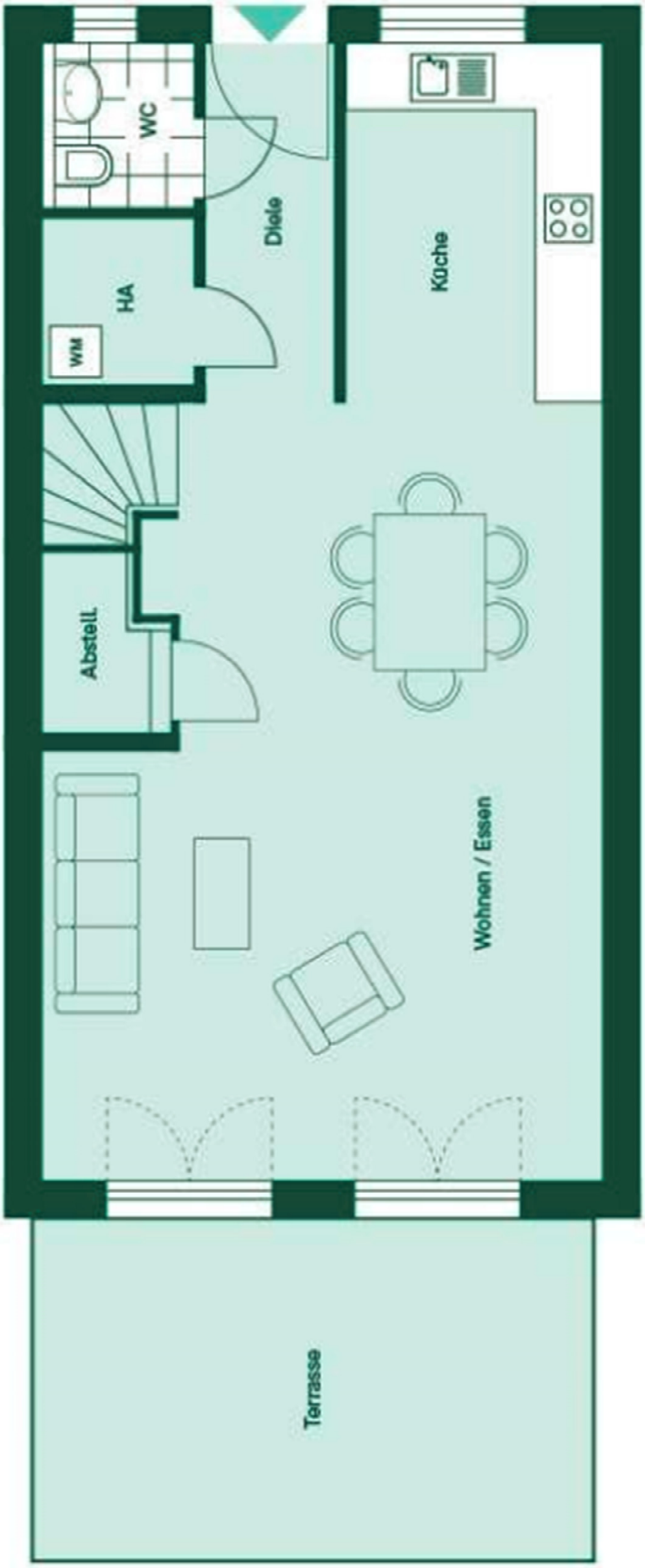
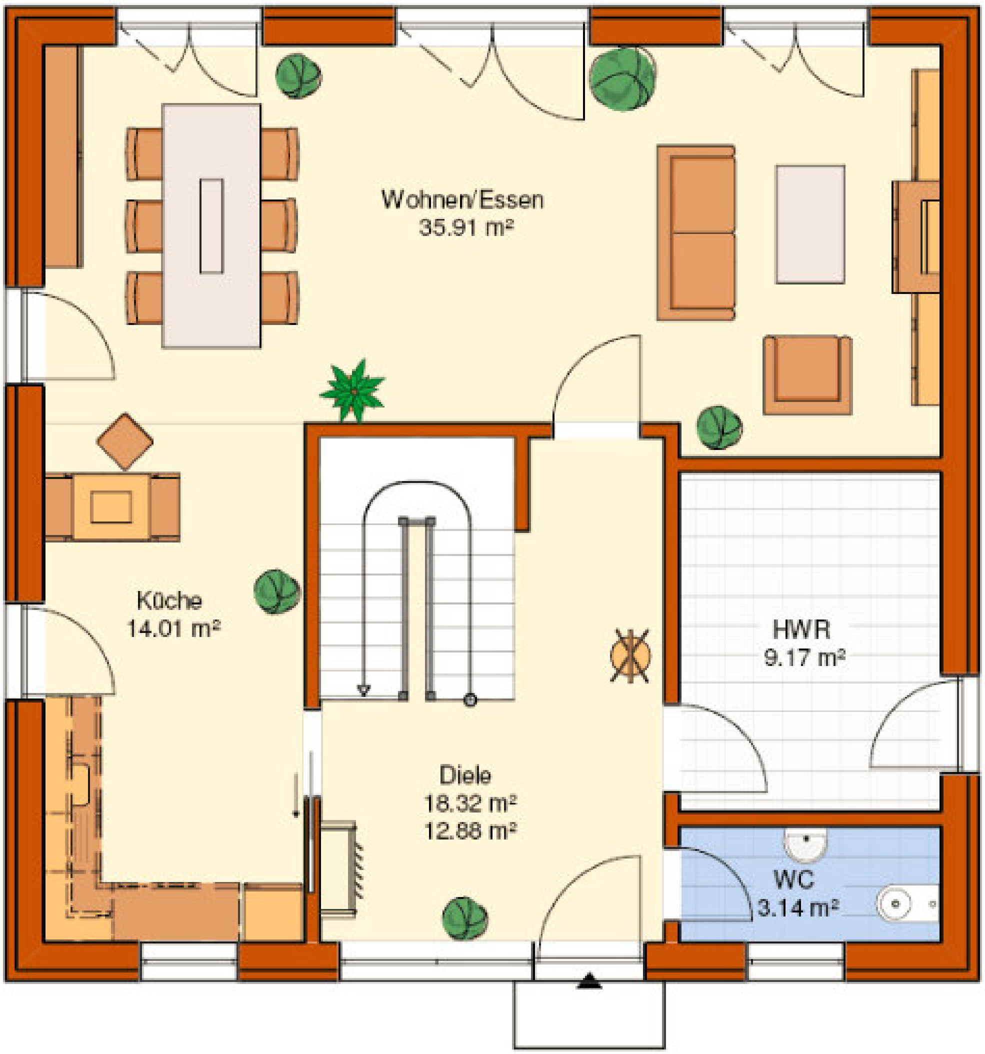


Figure 16-5: Terraced house, building location: Aachen, earthquake zone 2 [13]

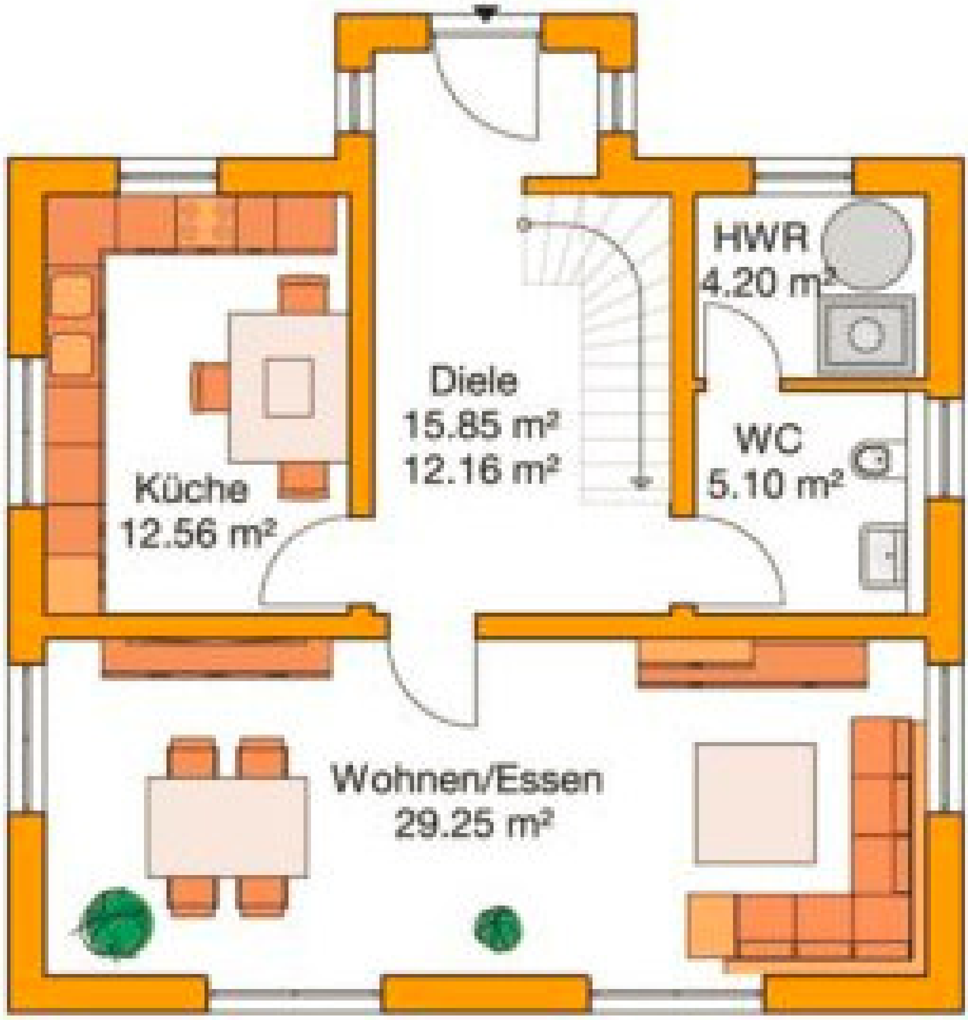
16.2 One-family houses

This section shows ground plan configurations of one-family houses in German earthquake regions. All examples are two or three storey buildings with continuous shear walls over the building height and rigid floors. One building configuration for each material (clay brick, calcium silicate, autoclaved aerated concrete, lightweight concrete) is presented. The collected ground plan configurations are quite similar to the representative configurations used for the parametric study.



Erdgeschoss:	74,60 m²
Obergeschoss:	69,28 m²
Zimmeranzahl:	6
Dachform:	Zeltdach
Dachneigung:	22 °
Dachüberstand:	50 cm
Außenmaße:	10,00 m x 10,00 m
Höhe:	8,60 m

Figure 16-6: One family house, clay bricks, Stadtvilla 140, total area 144 m^2 [14]



Hausdaten
Systemhaus 196
Wohnfläche
Erdgeschoss: 63,27 m²
Obergeschoss: 63,65 m²
Gesamtwohnfläche: 126,92 m²
Außenmaße: 9,60 m x 10,05 m

Figure 16-7: One family house, autoclaved aerated concrete, system house 196, total area 127 m^2 [15]

Abmessungen	9,23 x 8,81 m
Dachneigung	38°
Kniestock	75 cm
Nutzfläche	127,94 m² (nach DIN 277)
Wohnfläche	116,54 m² (nach WoFIV)

Figure 16-8: One family house, calcium siliacate, total area 128 m^2 [17]

Figure 16-9: One family house, lightweight concrete, total area 120 m^2

16.3 Multi-family houses

This section shows ground plan configurations of multi-family houses in German earthquake regions. The examples are two, three or four storey buildings with continuous shear walls over the building height and rigid floors. One building configuration for each material (clay brick, calcium silicate, autoclaved aerated concrete, lightweight concrete) is presented. The collected ground plan configurations are quite similar to the representative configurations used for the parametric study.

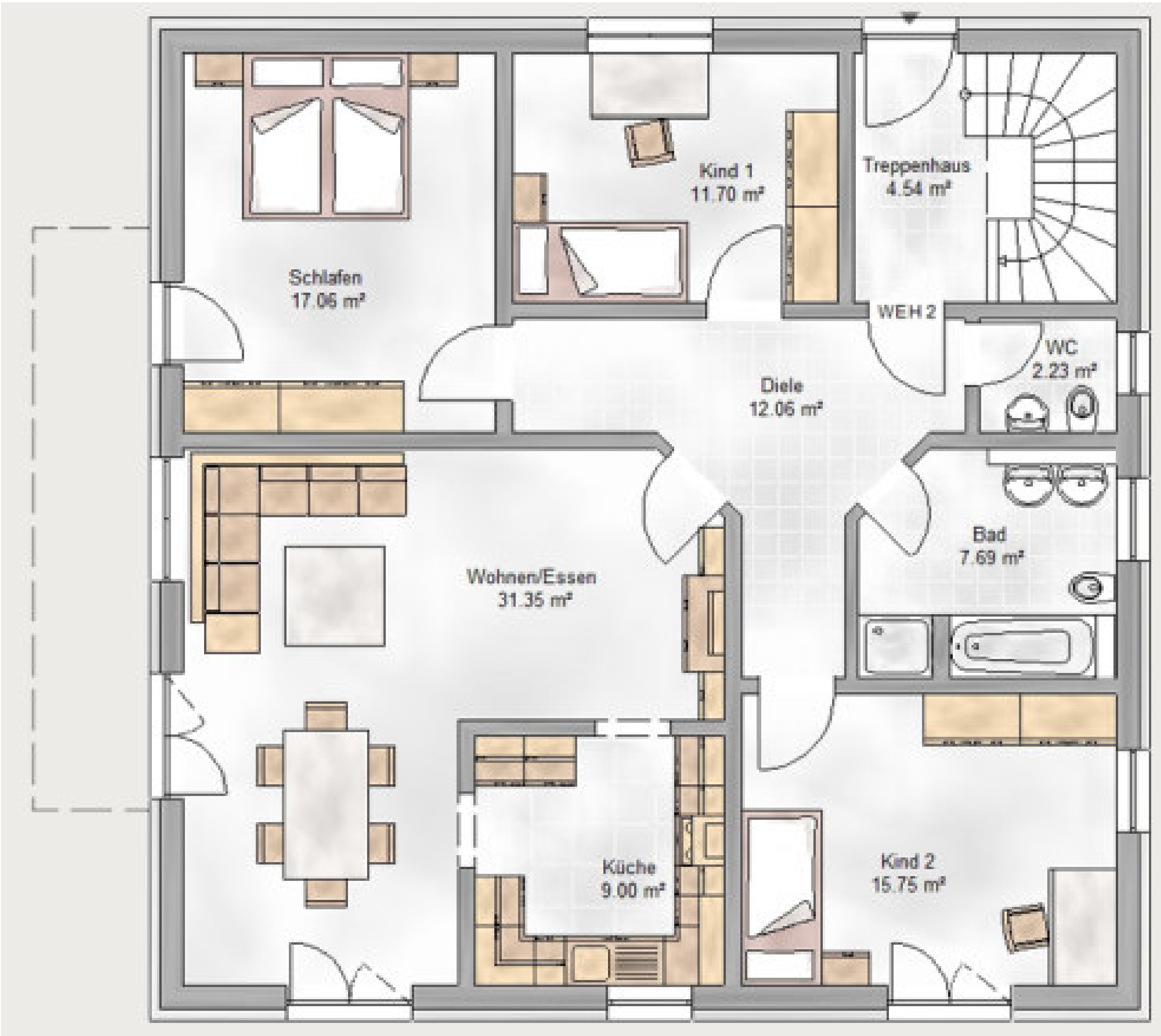
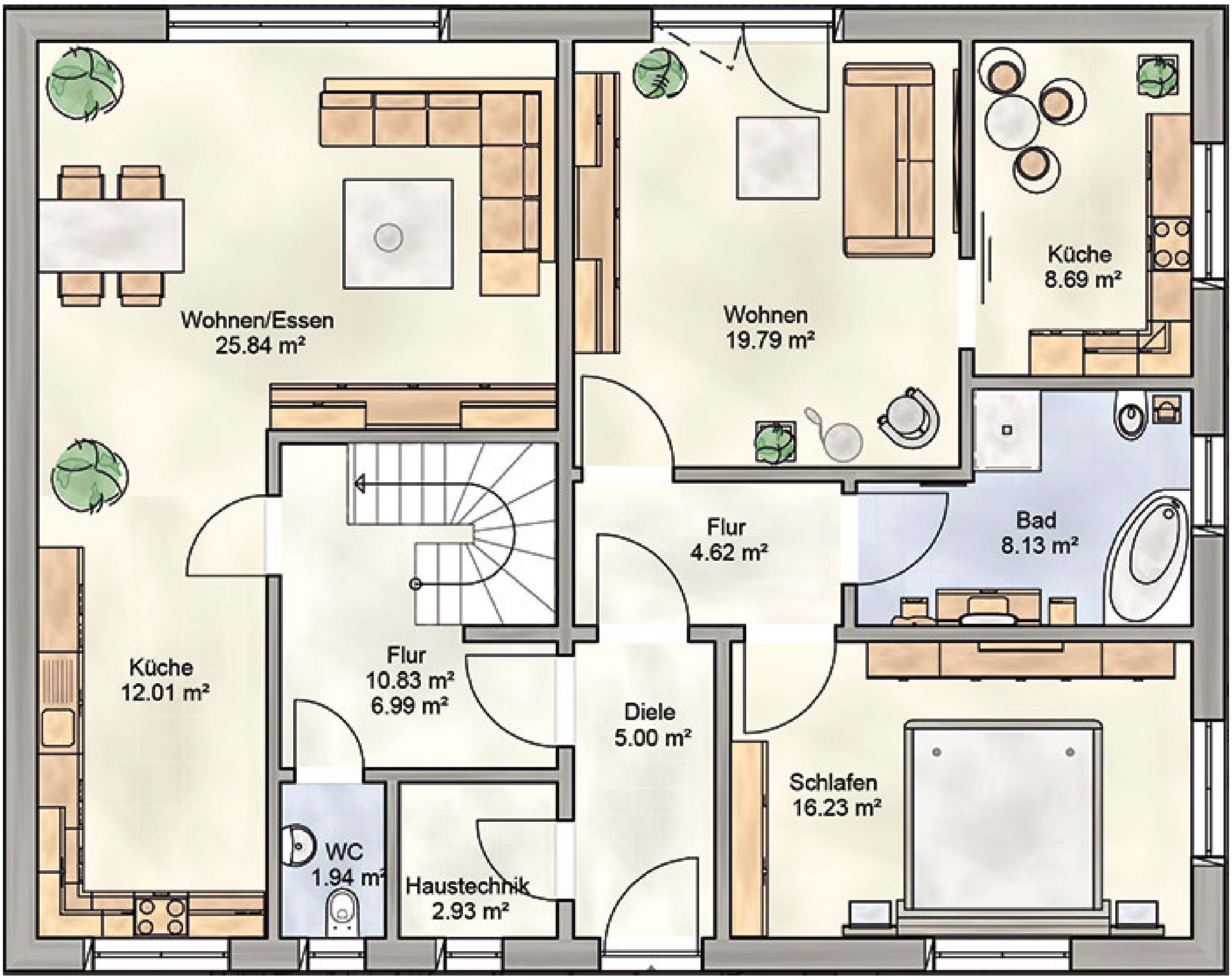


Figure 16-10: Multi-family house, clay bricks, House type 330, Total area 330 m^2 [14]



- **Wohnfläche:** 214,88 qm
- **Fläche Hauptwohnung:** 157,42 qm
- **Fläche Einliegerwohnung:** 57,46 qm

Figure 16-11: Multi-family house, autoclaved aerated concrete, total area 215 m^2 [16]

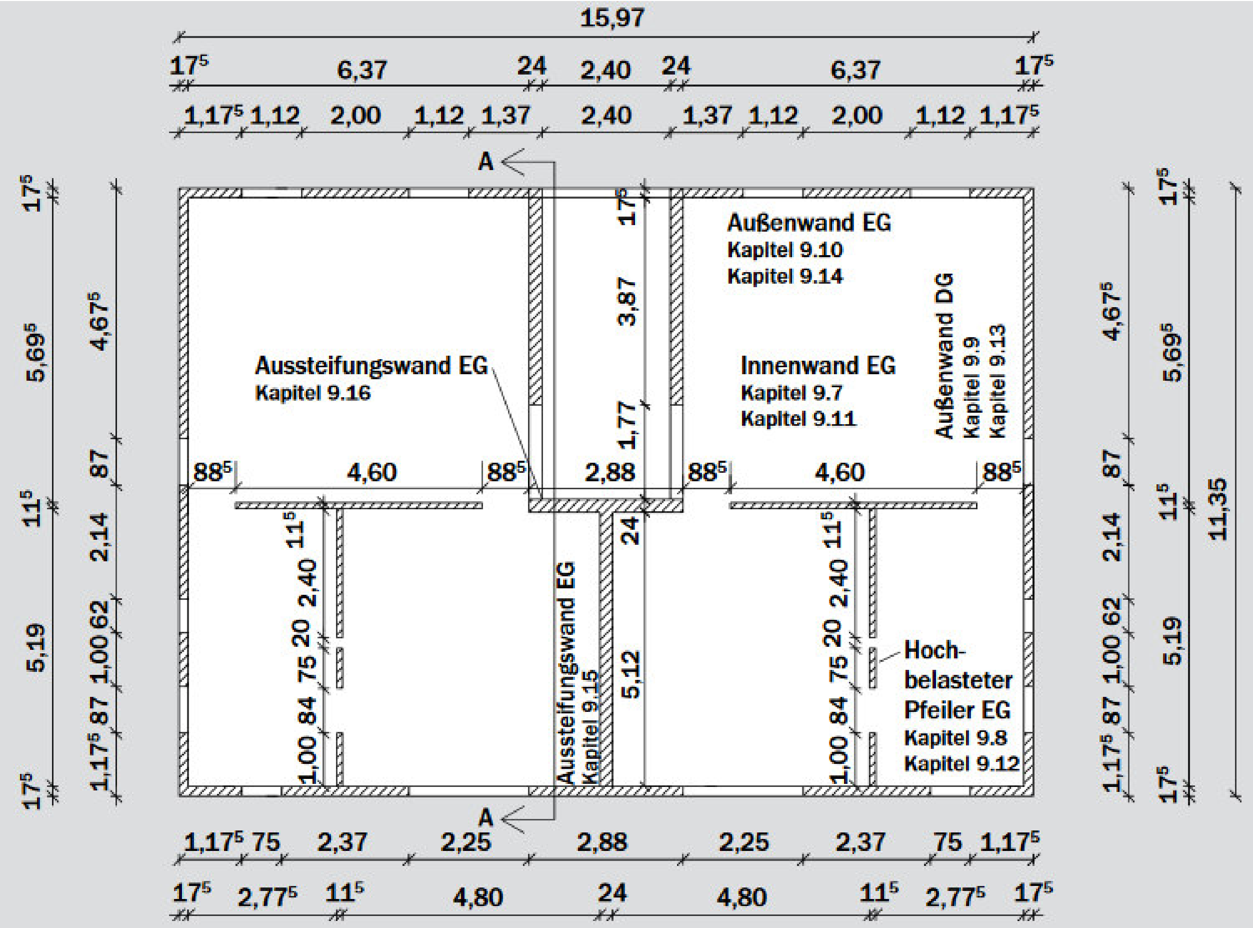
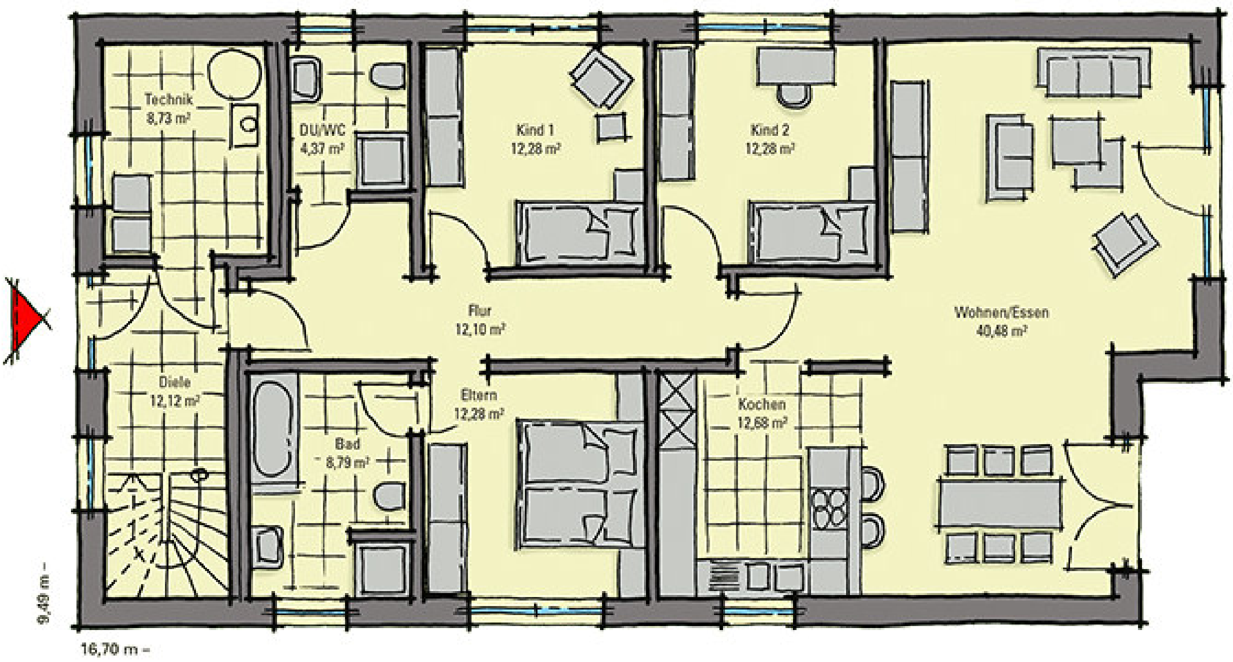


Figure 16-12: Multi family house, calcium silicate [16]



Dachform	Flachdach
Außenmaße	16,70 x 9,49 m + 1,30 x 5,69 m
Wohnung EG	
Raumgrundfläche EG	115.26 m²
Wohnung DG	
Raumgrundfläche DG	119.93 m²
Gemeintschaftsfläche	26,78 qm
Wohnfläche Gesamt	253.99 m²
Bebaute Fläche	165.88 m²
Raumgrundfläche Gesamt	261.97 m²

Figure 16-13: Multi-family house, autoclaved aerated concrete, Wohnfläche 262 m^2 [18]

DIBt-Project

Report WP3-RWTH
Parametric study

Improved seismic design concepts for masonry buildings in Germany

(Verbesserte seismische Nachweiskonzepte für Mauerwerksbauten in Deutschland)

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Annex 1: Results of linear calculations
Annex 2: Results of non-linear static analyses

1 INTRODUCTION

This report summarizes and evaluates the results of the parametric study for representative unreinforced masonry buildings German earthquake regions. Linear and nonlinear calculations according to the masonry codes DIN EN 1996-1-1 [1] and DIN EN 1996-1-1/NA-[2] in combination with the seismic codes DIN EN 1998-1 [6], DIN 1998-1/NA-2011 [7] and DIN 1998-1/NA-2018 [11] are carried out for each building. This report refers to the first two reports WP1-RWTH [12] and WP2-RWTH [13]. Report WP1-RWTH [12] includes the basic assumptions and descriptions of the applied calculation methods and verification approaches. Report WP2-RWTH [13] presents the representative buildings selected for the parametric study and provides building geometries and all material and strength properties.

The objective of the parametric study is a better utilization of the load-bearing reserves within the application of the traditional force-based design by taking into account deformation capacity, energy dissipation and overstrength of masonry structures in more realistic way. The ductility of a specific building can be calculated by using the rule of equal energy dissipation and making use of the displacement ductility. Alternatively, the dissipated energy can be determined by the evaluation of hysteresis curves at different displacement levels. The overstrength can be estimated as the results of material related overstrength and load redistribution.

It is intended to apply in the end only one resulting behaviour factor in order to keep the traditional force-based approach as the standard calculation procedure in practice as simple as possible. The proposed solution subdivides the resulting behaviour factor into three parts, which account for overstrength, deformation capacity and energy dissipation capacity. This approach is completely in line with the new draft of DIN EN 1998-1-2018 [3].

Finally, it has to be pointed out that the force-based design concept is only an estimation of the real behaviour of masonry buildings under seismic loading. As the basic idea of the behaviour factor is a general and simplified consideration of all nonlinear effects, the utilization of the load-bearing reserves is quite limited in comparison to non-linear calculations for specific masonry buildings. However, the present report will significantly improve the force-based design of masonry buildings in Germany in comparison to the normative rules available so far. The results of the parametric study are limited to typical masonry buildings in Germany and cannot be transferred to other countries with differing construction types.

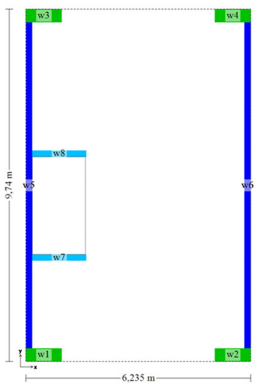
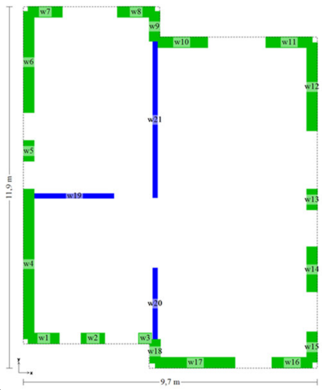
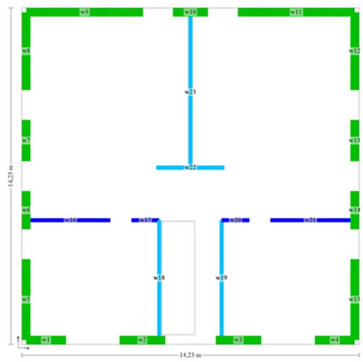
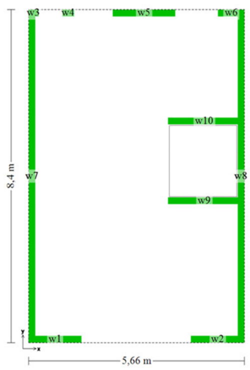
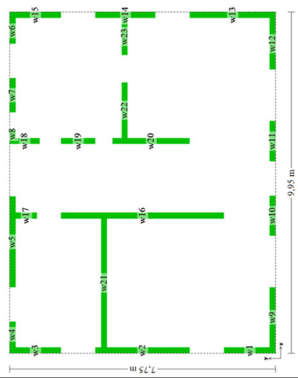
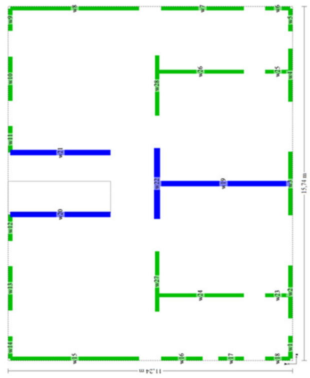
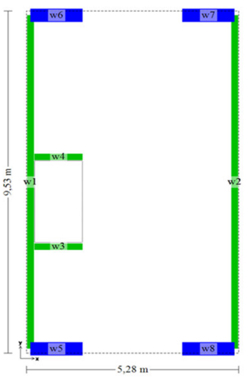
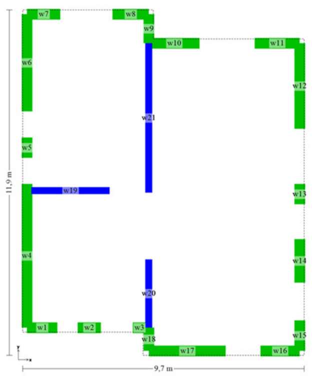
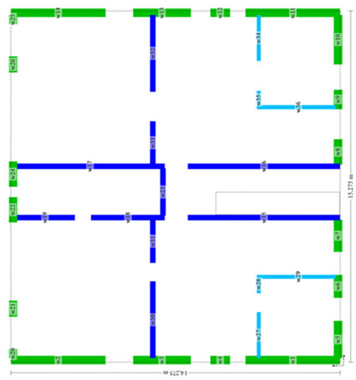
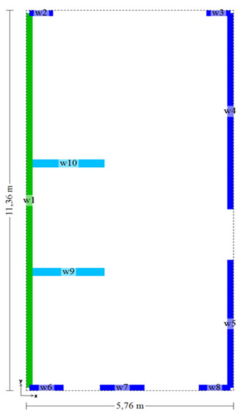
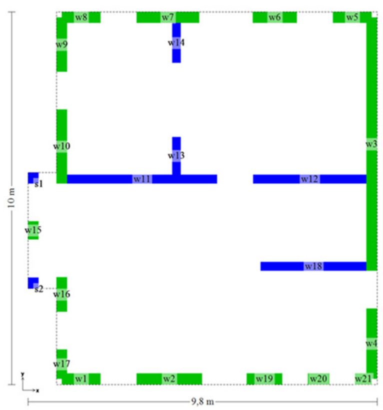
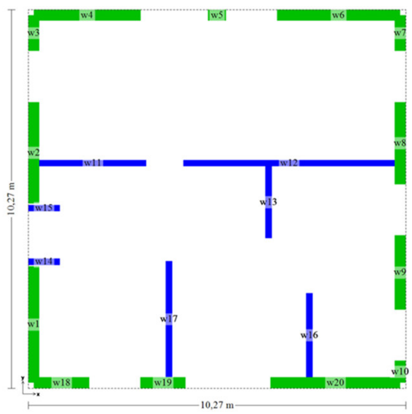
2 SAMPLE BUILDINGS

Table 2-1 presents the list of the representative buildings selected for the parametric study. More detailed information are provided in Report WP2-RWTH [13]. Table 2-2 presents the ground plan configurations of all buildings.

Table 2-1 Overview of the parametric study

Building	Building type	Stories	Material
TH-CB-1	Terraced house	1	Clay brick
TH-CB-2	Terraced house	2	Clay brick
FH-CB-2	Single-family house	2	Clay brick
FH-CB-3	Single-family house	3	Clay brick
MFH-CB-2	Multi-family house	2	Clay brick
MFH-CB-3	Multi-family house	3	Clay brick
MFH-CB-4	Multi-family house	4	Clay brick
TH-CS-1	Terraced house	1	Calcium silicate
TH-CS-2	Terraced house	2	Calcium silicate
FH-CS-2	Single-family house	2	Calcium silicate
FH-CS-3	Single-family house	3	Calcium silicate
MFH-CS-2	Multi-family house	2	Calcium silicate
MFH-CS-3	Multi-family house	3	Calcium silicate
MFH-CS-4	Multi-family house	4	Calcium silicate
TH-AC-1	Terraced house	1	Autoclaved aerated concrete
TH-AC-2	Terraced house	2	Autoclaved aerated concrete
FH-AC-2	Single-family house	2	Autoclaved aerated concrete
FH-AC-3	Single-family house	3	Autoclaved aerated concrete
MFH-AC-2	Multi-family house	2	Autoclaved aerated concrete
MFH-AC-3	Multi-family house	3	Autoclaved aerated concrete
MFH-AC-4	Multi-family house	4	Autoclaved aerated concrete
TH-LWC-1	Terraced house	1	Lightweight concrete
TH-LWC-2	Terraced house	2	Lightweight concrete
FH-LWC-2	Single-family house	2	Lightweight concrete
FH-LWC-3	Single-family house	3	Lightweight concrete
MFH-LWC-2	Multi-family house	2	Lightweight concrete
MFH-LWC-3	Multi-family house	3	Lightweight concrete
MFH-LWC-4	Multi-family house	4	Lightweight concrete

Table 2-2 Ground plan configurations of the investigated buildings

	Terraced house (1-2 stories)	Single-famlily house (2-3 stories)	Multi-family house (2-4 stories)
Clay brick			
Calcium Silicate			
Autoclaved Aerated concrete			
Lightweight concrete			

3 BEHAVIOUR AND OVERSTRENGTH FACTORS IN THE CODES

3.1 DIN EN 1998-1 (2010) and DIN EN 1998-1/NA (2011)

3.1.1 General part

DIN EN 1998-1 [6] describes in the general part the background of the behaviour factor to be applied to reduce the elastic response spectrum. In case of non-dissipative structures, a behaviour factor of q = 1.5 is recommended, which accounts only for the structural overstrength. Hysteretic energy dissipation is not taken into account. This is described in DIN EN 1998-1, Section 2.2.2 [6]:

2.2.2 Ultimate limit state

(1)P It shall be verified that the structural system has the resistance and energy-dissipation capacity specified in the relevant Parts of EN 1998.

(2) The resistance and energy-dissipation capacity to be assigned to the structure are related to the extent to which its non-linear response is to be exploited. In operational terms such balance between resistance and energy-dissipation capacity is characterised by the values of the behaviour factor q and the associated ductility classification, which are given in the relevant Parts of EN 1998. As a limiting case, for the design of structures classified as non-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor may not be taken, in general, as being greater than the value of 1,5 considered to account for overstrengths. For steel or composite steel concrete buildings, this limiting value of the q factor may be taken as being between 1,5 and 2 (see Note 1 of Table 6.1 or Note 1 of Table 7.1, respectively). For dissipative structures the behaviour factor is taken as being greater than these limiting values accounting for the hysteretic energy dissipation that mainly occurs in specifically designed zones, called dissipative zones or critical regions.

NOTE The value of the behaviour factor q should be limited by the limit state of dynamic stability of the structure and by the damage due to low-cycle fatigue of structural details (especially connections). The most unfavourable limiting condition shall be applied when the values of the q factor are determined. The values of the q factor given in the various Parts of EN 1998 are deemed to conform to this requirement.

In case of dissipative structure with ductility classes DCL or DCM, it is referenced in DIN EN 1998-1, Section 3.2.2.5 [6] to the material related parts:

3.2.2.5 Design spectrum for elastic analysis

(3)P The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

3.1.2 Material related parts

The definition of the behaviour factor in the material related parts for reinforced concrete, steel and composite structures is based on a basic value q_0 increased by a redistribution factor α_u/α_1:

$$q = q_0 \cdot \alpha_u/\alpha_1$$

q_0 Basic behaviour factor accounting for deformation capacity and energy dissipation

α_u/α_1 Overstrength factor due to force redistribution

α_u Multiplier of the horizontal seismic action at formation of a global plastic mechanism

α_1 Multiplier of the horizontal seismic action at formation of the first plastic hinge

The definition of the behaviour factor for timber and masonry structures is just given as resulting values q, not subdivided into the basic behaviour factor q_0 and the redistribution factor α_u/α_1. Thus, no overstrength has been taken into account for these two structural types so far. In the following sections the ranges of the basic behaviour factors q_0 and the redistribution factors α_u/α_1 are shortly summarized.

3.1.2.1 *Reinforced concrete structures*

The range of the basic behaviour factors and load redistribution factors for reinforced concrete structures according to DIN EN 1998-1, Chapter 5 [6] are defined as follows:

q_0 DCM: 1.5 - 3 and DCH: 2 – 4.5

α_u/α_1 1.0 – 1.3 for regular load-bearing systems without building specific pushover analysis

1.0 – 1.15 for irregular load-bearing systems without building specific pushover analysis

1.0 – 1.5 if the redistribution is verified by a building specific pushover analysis

Note: *Redistribution can be considered in DCM and DCH!*

For buildings which are not regular in elevation, the value of q_0 should be reduced by 20%, but need not be taken less than 1.5.

3.1.2.2 *Steel structures*

The range of the basic behaviour factors and load redistribution factors for steel structures according to DIN EN 1998-1, Chapter 6 [6] are defined as follows:

q_0 DCM: 2 - 4 and DCH: 2 – 5

α_u/α_1 1.0 – 1.3 for regular load-bearing systems without building specific pushover analysis

1.0 – 1.15 for irregular load-bearing systems without building specific pushover analysis

1.0 – 1.6 if the redistribution is verified by a building specific pushover analysis

Note: *Redistribution is only considered in DCH!*

For buildings which are not regular in elevation, the value of q_0 should be reduced by 20%, but need not be taken less than 1.5.

3.1.2.3 *Timber structures*

The range of the behaviour factors for timber structures according to DIN EN 1998-1, Chapter 8 [6] are defined as follows:

q DCM: 2 – 2.5 and DCH: 3-5

α_u/α_1 Definition of overstrength factors is not included

For buildings which are not regular in elevation, the value of *q* should be reduced by 20%, but need not be taken less than 1.5.

3.1.2.4 *Composite structures*

The range of the basic behaviour factors and load redistribution factors for composite structures according to DIN EN 1998-1, Chapter 7 [6] are defined as follows:

q_0 DCM: 3 and DCH: 4 - 4.5

α_u/α_1 1.0 – 1.2 for regular load-bearing systems without building specific pushover analysis

1.0 – 1.15 for irregular load-bearing systems without building specific pushover analysis

1.0 – 1.6 if the redistribution is verified by a building specific pushover analysis

Note: *Redistribution can only be considered in DCM and DCH!*

For buildings which are not regular in elevation, the value of q_0 should be reduced by 20%, but need not be taken less than 1.5.

3.1.2.5 *Unreinforced masonry structures*

The range of the basic behaviour factors and load redistribution factors for masonry structures according to DIN EN 1998-1, Chapter 7 [6] are defined as follows:

q 1.5 – 2.5 (recommended value: 1.5)

α_u/α_1 Definition of overstrength factors is not included

The definition of the behaviour factor in the German National Annexes DIN EN 1998-1/NA-2011 [7] and DIN EN 1998-1/NA-2018 [11] is defined as follows:

q 1.5 (h/l < 1) and h/l ≥ 1.6 (intermediate values can be interpolated)

For buildings which are not regular in elevation, the value of *q* should be reduced by 20%, but need not be taken less than 1.5.

3.2 Draft of DIN EN 1998-1 (2018)

The current draft of DIN EN 1998-1-2018 [11] proposes in the non-material related part a modified approach to reduce the elastic response spectrum for the traditional force-based design. The proposal subdivides the behaviour factor *q* into three parts to account for overstrength, deformation capacity and dissipation capacity:

$$q = q_R \cdot q_D \cdot q_S$$

q_R Overstrength due to redistribution

q_D Deformation capacity and energy dissipation

q_S Overstrength due to all other sources

The relevant part of the current draft of DIN EN 1998-1-2018 [11] has the following content:

6.3.2 Reduced spectrum for the force-based approach

(1) In the force-based approach the seismic action should take the form of a reduced spectrum, derived from the elastic response spectrum by introducing the behaviour factor q, which accounts for overstrength, deformation capacity and energy dissipation capacity and is given by Formula (6.2).

$$q = q_R\, q_S\, q_D \tag{6.2}$$

where:

q_R is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures;

q_S is the behaviour factor component accounting for overstrength due to all other sources;

q_D is the behaviour factor component accounting for the deformation capacity and energy dissipation capacity.

(2) EN 1998 is conceived in such a way that behaviour factor component values should be:

a) $q_R = 1$ if not otherwise specified in the relevant part of EN 1998. For buildings, q_R is specified in **7.3.3.1**;

b) $q_S = 1{,}5$;

c) $q_D = 1$ for DCL.

NOTE An exception to the generic rule $q_S = 1{,}5$ is possible when dealing with specific devices that do not have any intrinsic overstrength by design.

(3) Other q_D values are controlled by the ductility classes. DCM values should not be lower than 1 and DCH values not lower than DCM ones. q_D values are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. According to **4.4.1(1)P**, they should be selected so as to keep a significant margin with respect to the NC deformation capacity.

(4) q_D values may be different in different horizontal directions, although the ductility class should be the same in all directions.

The behaviour factors must be defined for each material. Their definition is an ongoing work.

3.3 Summary: DIN EN 1998-1 (2018), DIN EN 1998-1 (2010), DIN EN 1998-1/NA (2011)

The definition of the behaviour factor for unreinforced masonry structures in the recent version DIN EN 1998-1 [6] and the German National Annexes DIN EN 1998-1/NA-2011 [7] and DIN EN 1998-1/NA-2018 [11] is only based on a behaviour factor that takes into account deformation capacity and energy dissipation. The effects of overstrength are not considered in contrast to most of the other building materials.

3.4 Behaviour factors for unreinforced masonry structures in international codes

Behaviour factors are prescribed in almost all international seismic design codes. The nomenclature of the behaviour factors varies between the codes. The general form of DIN EN 1998-1 [6]

$$q = q_0 \cdot \alpha_u/\alpha_1$$

corresponds to the definition in the US-codes and the New Zeeland code:

$$R = R_\mu \cdot OSR$$

R_μ is the component of the force reduction factor associated with the inherent energy dissipation capacity, that is, the ductility of the structural system under consideration. The structural ductility factor, μ is defined as the ultimate displacement of the structure divided by the yield displacement of the structure. Table 3-1 compares the use of the factors in international codes.

Table 3-1 Comparison of structural ductility and overstrength factors for unreinforced masonry in international codes [26]

CODE	ORIGIN	Design Code Nomenclature				Recommended Code Values for URM			
		μ	R_μ	OSR	$R^\dagger$	μ	R_μ	OSR	$R^\dagger$
NZS1170.5 (2004) NZSEE (2006)	NZ	μ	$k_\mu = 1+(\mu-1)T_1/0.7 \leq \mu$ ($\xi = 5\%$) $k_\mu = 1/0.65 = 1.54$ ($\xi = 15\%$)	$1/S_p$	k_μ/S_p	1.0-1.5	1.0-1.54	1.0	1.0 - 1.54
AS1170.4 (2007)	Australia	μ	μ	$1/S_p$	μ/S_p	1.25	1.25	1.3	1.62
OPCM 3274 (2003)	Italy	-	q`	α_u/α_1	q	-	1.5-2.5	1.4 - 2.5	2.1 – 5.0
Eurocode 8 (2004)	Europe	-	-	-	q	-	-	-	1.5 - 2.5
ASCE-41 (2007)	USA	-	-	-	$m \times \kappa$	-	-	-	$\kappa \times 3h_{eff}/L \geq 1.25$ (Rocking)
FEMA 356 (2000) FEMA 273 (1997)	USA	-	-	-	$m \times \kappa$	-	-	-	$\kappa \times 3.0 \geq 2.25$ (Bed joint sliding) $\kappa \times 3h_{eff}/L \geq 1.25$ (Rocking)

Eurocode 8 (DIN EN 1998-1) [6] and the Italian seismic design code OPCM 3274 [9], [10] express the force reduction factor *R* as *q*. In, OPCM 3274 [9], [10], the value of *q* is explicitly separated into the ductility and overstrength components q_0 and α_u/α_1 respectively. OPCM 3274 [9], [10] gives a range of q_0 values from 1.5 - 2.0 and overstrength values from 1.4 – 1.8 for unreinforced masonry buildings, depending on the structural regularity and number of floors present within the building. The values in OPCM 3274 [9], [10] are based on a comprehensive parametric study carried out by Morandi [22]. The following behaviour and overstrength values are proposed:

Unreinforced masonry buildings, regular in elevation — $2.0 \cdot \alpha_u/\alpha_1$
Unreinforced masonry buildings, irregular in elevation — $1.5 \cdot \alpha_u/\alpha_1$

Single-storey unreinforced masonry buildings — $\alpha_u/\alpha_1 = 1.4$
Double- or multi-storey unreinforced masonry buildings — $\alpha_u/\alpha_1 = 1.8$

Morandi [22] stated out, that the main problem of linear seismic design of masonry structures is the definition of the behaviour factor *q* to reduce the linear response spectrum. It is important to note, that the proposed behaviour and overstrength factors are just applicable in case of continuous ring beams at each floor level. Morandi [22] proposed the following reduced values, if the structural requirements are not fulfilled and the individual walls are uncoupled and not linked to each other by a ring beam:

Single-storey unreinforced masonry buildings, uncoupled walls — $\alpha_u/\alpha_1 = 1.0$
Double- or multi-storey unreinforced masonry buildings — $\alpha_u/\alpha_1 = 1.4$

The definition of the behaviour factor *q* for unreinforced masonry structures was modified recently in Italian code NTC 2018 [8] as follows:

Unreinforced masonry buildings $1.75 \cdot \alpha_u/\alpha_1 = 1.0$

α_u/α_1 1.7 for regular load-bearing systems without building specific pushover analysis.
1.0 – 1.35 for irregular load-bearing systems without building specific pushover analysis.

For buildings which are not regular in elevation, the value of q_0 should be reduced by 20%, but need not be taken less than 1.5.

4 PROCEDURE TO DERIVE BEHAVIOUR FACTORS FOR URM

The behaviour factors are derived by statistical evaluation of the results of the parametric study on 28 building configurations. As proposed in the draft of DIN EN 1998-1-2018 [3] the behaviour factor q is subdivided into three parts to account for overstrength, deformation capacity and dissipation capacity:

$$q = q_R \cdot q_D \cdot q_S$$

q_R Overstrength due to redistribution

q_D Deformation capacity and energy dissipation

q_S Overstrength due to all other sources

The definition and determination of the three components of the behaviour factor based on the nonlinear calculation results is explained in the following sections.

4.1 Behaviour factor q_R accounting for load redistribution

The behaviour factor q_R due to overstrength is defined as the ratio of the maximum base shear $F_{b,max}$ to the elastic base shear $F_{b,el}$ at the first wall failure as shown in Figure 4-1:

$$q_R = \frac{F_{b,max}}{F_{b,el}}$$

with

$F_{b,max}$ Maximum total shear force calculated with nonlinear analyses

$F_{b,el}$ Maximum total shear force at first wall failure calculated with linear analyses

The elastic base shear $F_{b,el}$ is simply calculated by a separate linear elastic calculation with identical material parameter and boundary conditions. Both calculations are carried out with mean values for the material and strength parameter.

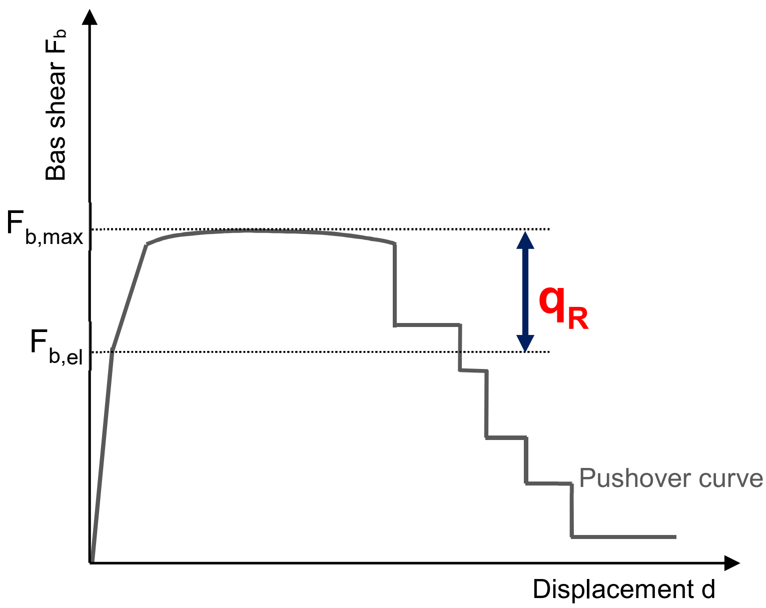


Figure 4-1: Definition of the overstrength ratio accounting for load redistribution

4.2 Behaviour factor q_D accounting for deformation capacity and energy dissipation

The behaviour factor q_D due to deformation capacity and energy dissipation of the overall building is defined as the ratio of the seismic force assuming linear-elastic structural behaviour to the ultimate capacity of the building taking into account deformation capacity and energy dissipation.

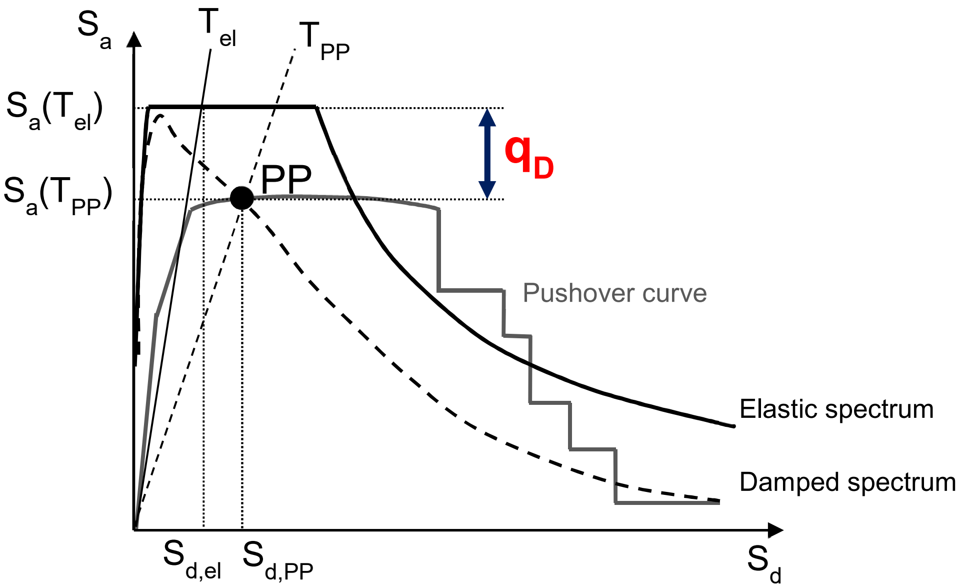


Figure 4-2: Definition of the behaviour factor accounting for deformation capacity and energy dissipation

Based on the given definition, the behaviour factor q_D is determined as the intersection point of the building capacity curve and the damped spectra in the common S_a-S_d diagram as shown in Figure 4-2:

$$q_D = \frac{S_a(T_{el}) \cdot \alpha_{el} \cdot M_{eff,el}}{S_a(T_{PP}) \cdot \alpha_{PP} \cdot M_{eff,PP}}$$

with

T_{el}	Fundamental period of the linear system
T_{PP}	Fundamental period of the inelastic system at the performance point
$S_a(T_{el})$	Spectral acceleration assuming linear elastic behaviour
$S_a(T_{PP})$	Spectral acceleration of the inelastic system at the performance point
α_{el}	Participation factor of the linear system
α_{PP}	Participation factor of the inelastic system at the performance point
$M_{eff,el}$	Total effective mass of the linear system
$M_{eff,PP}$	Total effective mass of the inelastic system at the performance point

4.3 Behaviour factor q_S accounting for all other sources of overstrength

The behaviour factor q_S accounts for all other sources of overstrength and the recommended value in the draft version of DIN EN 1998-1-2018 [3] is 1.5. However, as unreinforced masonry is a quite brittle material, the amount of further overstrength additionally to the already considered overstrength is rather limited. For this reason only the material overstrength caused by the usage of characteristic values reduced with partial safety factors can be considered. This component of the behaviour factor considers the fact that the characteristic design values are always conservative lower bound estimates of the actual probable strengths of the structural materials and their effective strengths in the as-constructed structure. The behaviour factor q_S can be calculated as follows:

$$q_S = \frac{F_{b,el}}{F_{b,des}}$$

with

$F_{b,el}$ Maximum total shear force calculated with linear analyses using mean values

$F_{b,des}$ Maximum total shear force calculated with linear analyses and characteristic values

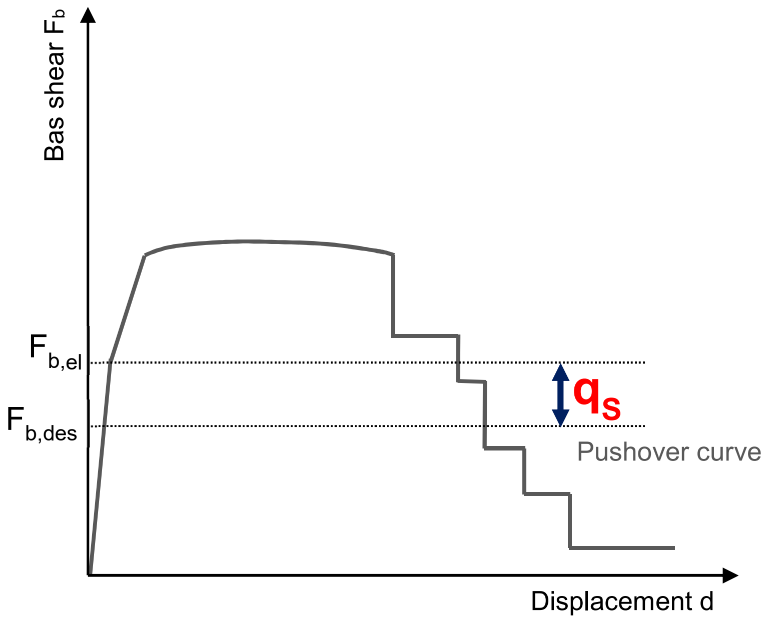


Figure 4-3: Definition of the overstrength ratio accounting for material overstrength

5 LINEAR AND NONLINEAR CALCULATIONS

In the following, 28 configurations of the 12 sample buildings introduced in Chapter 2 are investigated. The sample buildings are analysed by means of linear elastic and nonlinear pushover analyses according to DIN EN 1996-1-1/NA [2], DIN EN 1998-1-2011/NA [7], DIN EN 1998-1-2018/NA [11] in combination with the basic documents DIN EN 1996-1-1 [1] and DIN EN 1998-1 [6]. All calculations are carried out with MINEA-R [30]. More detailed information about the calculation procedures and sample buildings are summarized in the reports WP1-RWTH and WP2-RWTH [12], [13]. The background and theoretical basis of the linear and nonlinear calculation approach are detailed described in report WP1-RWTH [12] and all information of the sample buildings are collected in report WP2-RWTH [13].

5.1 Design response spectrum

The design response spectra are applied according to DIN EN 1998-1/NA-2011 [7] for the subsoil conditions C-R and C-S. The combination C-R is the most unfavourable subsoil condition with respect to the maximum soil amplification factor, and the subsoil condition C-S is characterized by a broader plateau range. For each calculation, the PGA-values are scaled up and down as long as a successful verification is achieved. Figure 5-1 and Figure 5-2 show the shapes of the design response spectra. The calculations neglect the first rising branch and the plateau is continued to the period T = 0s. As the initial stiffness of masonry structures is a complex issue and exhibit always a large scatter, the plateau range is elongated.

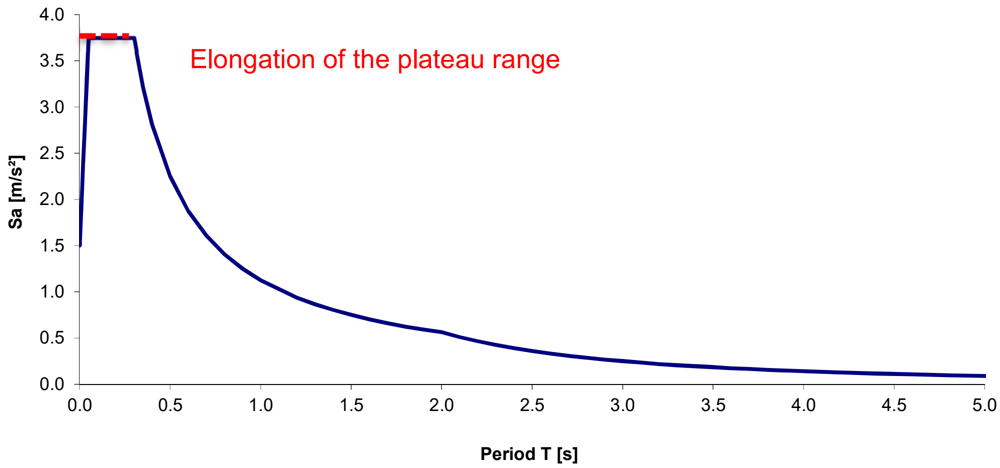


Figure 5-1 Design response spectrum for subsoil condition C-R, q = 1, a_g = 1.0

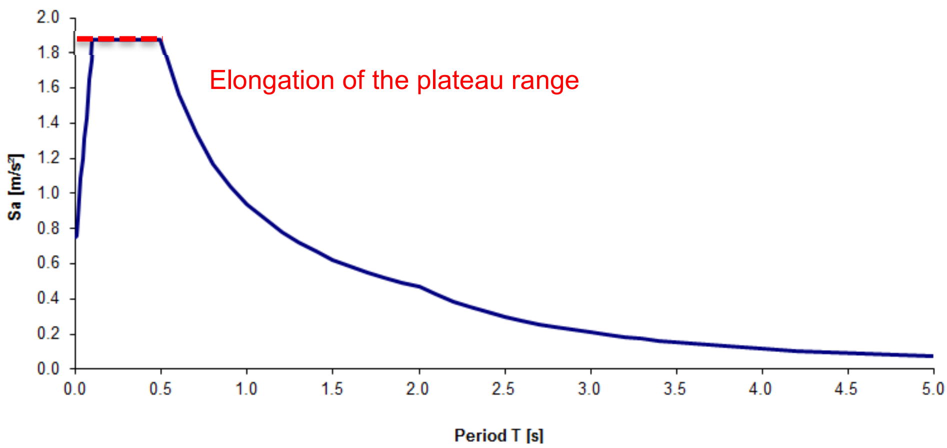


Figure 5-2 Design response spectrum for subsoil condition C-S, q = 1, a_g = 1.0

5.2 Basic assumptions of the calculations

5.2.1 General assumptions

The linear and nonlinear calculations of the parametric study are based on the following general assumptions, which are complementing the information given in WP1-RWTH [12]:

- Continuous shear walls over the building height.
- Wall areas with openings due to doors and windows are neglected.
- Shear transfer between intersecting walls is not considered.
- The slabs are rigid and distribute the horizontal loads to the shear walls. Double-sided slabs are assumed for the vertical load transfer.
- Vertical load redistribution is not taken into account.
- Combinations of the effects of seismic actions in two main orthogonal building directions are neglected. It is assumed, that the investigated buildings are regular in plan and elevation and that the walls are the only primary seismic elements in the two main horizontal directions.
- Torsional effects are considered according to DIN EN 1998-1/NA-2011, NA.D.4 [7].
- The calculations are based on the following structural systems, which differ with respect to the consideration of frame effects through the slabs. In each of the systems, the slab contribution is considered by the moment distribution factor as defined in DIN EN 1996-1-1, Annex K [2]:
    - Cantilever system: Although cantilever systems are too conservative, they are generally accepted in practise and can be regarded as the standard calculation models for linear analysis. The basic assumption for cantilever systems is a flexible slab. The corresponding moment distribution factors for the walls on ground floor level are calculated by means of a cantilever system of the building and listed in Table 5-1.
    - Systems with full restraint: These systems are based on the assumption of a full restraint of all walls through rigid slabs. This assumption is not realistic, but can be regarded as an upper limit of the moment redistribution factor. The factor is equal to 0.5 and causes a linear moment distribution with zero moment at half wall height.
    - Systems with partial restraint: These systems are based on moment redistribution factors with respect to the wall length and number of stories. The applied factors, summarized in Table 5-1, are based on preliminary calculations and literature review [23], [24], [25].

Table 5-1: Moment redistribution factors

	Cantilever	Partial restraint			Full restraint
Storey	All L_W	$L_W \leq 1.5m$	$1.5m < L_W < 2.5m$	$L_W \geq 2.5m$	All L_W
1	1	0.5	0.6	1.0	0.5
2	1.7	0.85	1.0	1.2	0.5
3	2.3	1.2	1.4	1.7	0.5
4	3.0	1.5	1.8	2.1	0.5

5.2.2 Assumptions for linear elastic analysis and design

- The design is carried out according to DIN EN 1996-1-1 [1] and DIN EN 1996-1-1/NA [2].
- The safety verification is governed by the failure of the first wall.
- All linear comparison calculation with nonlinear calculation results are based on mean values for material strength parameter and a material safety factor of γ_m = 1.0. All other linear calculations are carried out with characteristic values and a material safety factor of γ_m = 1.2.

5.2.3 Assumptions for nonlinear pushover analysis and verification

The nonlinear pushover analysis and verification is based on the following assumptions

- The calculations are carried out based on the material and strength parameter according to DIN EN 1996-1-1 [1] and DIN EN 1996-1-1/NA [2].
- The calculations are carried out with mean values of strength parameters.
- The conservative approach with damped spectra according to Norda [14] is applied with median damping curves.
- The drift limits are defined in accordance with DIN EN 1998-1/NA-2011 [7] and DIN EN 1998-1/NA-2018 [11]
- The nonlinear verification is fulfilled, if a performance point can be calculated before the failure of the first wall and accompanied with a drop down of the load-displacement curve.
- Minimum out-of-plane eccentricities are considered according to DIN EN 1996-1-1 [1] and DIN EN 1996-1-1/NA [2].

6 LINEAR AND NONLINEAR VERIFICATION RESULTS

In the following the verification results are summarized for both linear elastic and nonlinear static analyses. The comparison of the linear and nonlinear results is based on the maximum attainable PGA ($a_{gR} \cdot S$) with respect to the decisive building direction as explained in in the report WP2-RWTH [13]. All verifications are carried out in the most unfavourable building direction to be on the safe side and in line with the usual practise. The maximum attainable PGA will increase, if the seismic loading is applied in the stronger building direction.

6.1 Linear analyses

Table 6-1 summarizes the results of the linear verifications for the 28 building configurations and each type of structural system (Section 5.2). For each building configuration the maximum base shear F_b and the maximum attainable PGA are provided. The linear calculation results are mostly independent on the spectral shape, as the period T_1 of nearly all buildings is smaller than the corner periods T_c of the applied spectral shapes C-R and C-S. Only some of the cantilever systems have longer periods. Thus, the results for both subsoil conditions are only given for the cantilever system. A detailed summary of the verification results for the subsoil condition C-R is given in Annex 1.

Table 6-1: Results of linear verifications for the subsoil conditions C-R and C-S

System information				Cantilever, C-R		Cantilever, C-S		Partial restraint, C-R + C-S		Full restraint, C-R + C-S	
Building	Stories	q [-]	M [t]	F_b [kN]	max PGA [m/s^2]	F_b [kN]	max PGA [m/s^2]	F_b [kN]	max PGA [m/s^2]	F_b [kN]	max PGA [m/s^2]
01_TH-CB-1	1	2.0	59.2	51.8	0.56	51.8	0.56	103.6	1.12	103.6	1.12
02_TH-CB-2	2	2.0	108.8	55.8	0.33	55.8	0.33	111.5	0.66	190.4	1.12
03_FH-CB-2	2	1.5	213.9	96.2	0.25	96.2	0.25	139.0	0.37	264.6	0.70
04_FH-CB-3	3	1.5	313.2	94.0	0.17	94.0	0.17	137.0	0.25	348.4	0.63
05_MFH-CB-2	2	1.5	379.9	311.5	0.58	311.5	0.58	535.6	1.00	930.6	1.73
06_MFH-CB-3	3	1.5	553.9	332.3	0.42	332.3	0.42	570.5	0.73	1019.1	1.30
07_MFH-CB-4	4	1.5	727.9	336.3	0.26	336.3	0.26	591.4	0.46	1064.5	0.83
08_TH-CS-1	1	1.5	50.2	81.2	0.97	81.2	0.97	129.9	1.55	150.0	1.79
09_TH-CS-2	2	1.5	96.6	85.8	0.43	85.8	0.43	146.2	0.73	228.3	1.13
10_FH-CS-2	2	1.5	162.1	119.5	0.42	119.5	0.42	168.1	0.59	224.9	0.78
11_FH-CS-3	3	1.5	238.9	101.5	0.24	101.5	0.24	152.3	0.36	232.9	0.55
12_MFH-CS-2	2	1.5	371.0	301.4	0.46	301.4	0.46	412.7	0.63	658.5	1.00
13_MFH-CS-3	3	1.5	549.0	308.8	0.32	308.8	0.32	432.4	0.44	830.4	0.85
14_MFH-CS-4	4	1.5	727.1	309.0	0.30	309.0	0.24	445.3	0.35	990.6	0.77
15_TH-AC-1	1	2.0	48.5	40.7	0.54	40.7	0.54	82.5	1.09	82.5	1.09
16_TH-AC-2	2	2.0	87.8	40.6	0.30	40.6	0.30	83.4	0.61	139.4	1.02
17_FH-AC-2	2	1.5	193.6	79.9	0.23	79.9	0.23	121.0	0.35	164.6	0.48
18_FH-AC-3	3	1.5	279.4	80.3	0.16	80.3	0.16	104.8	0.21	181.6	0.37
19_MFH-AC-2	2	1.5	387.5	174.4	0.31	174.4	0.25	300.3	0.44	353.6	0.52
20_MFH-AC-3	3	1.5	559.8	181.9	0.42	181.9	0.18	293.9	0.30	384.9	0.39
21_MFH-AC-4	4	1.5	732.1	183.0	0.45	183.0	0.14	283.7	0.22	411.8	0.32
22_TH-LWC-1	1	1.5	70.3	79.1	0.64	79.1	0.64	116.0	0.79	130.0	0.89
23_TH-LWC-2	2	1.5	131.4	90.4	0.33	90.4	0.33	138.0	0.50	138.0	0.50
24_FH-LWC-2	2	1.5	185.5	68.9	0.21	68.9	0.21	119.5	0.37	137.9	0.42
25_FH-LWC-3	3	1.5	271.7	70.6	0.15	70.6	0.15	107.6	0.23	144.5	0.30
26_MFH-LWC-2	2	1.5	207.7	150.6	0.41	150.6	0.41	215.5	0.59	283.0	0.77
27_MFH-LWC-3	3	1.5	303.5	136.6	0.25	136.6	0.25	201.1	0.37	307.3	0.57
28_MFH-LWC-4	4	1.5	399.3	139.7	0.31	139.7	0.20	194.6	0.28	334.4	0.47

q Behaviour factor according to DIN EIN 1998-1/NA-2011 [7] and DIN EIN 1998-1/NA-2018 [11] [-]
M Total mass [t]
F_b Total base shear [kN]
max PGA Maximum attainable PGA ($a_{gR} \cdot S$)

6.2 Nonlinear static analyses

Table 6-2 summarizes the results of the nonlinear static analyses for the 28 building configurations and each type of structural system (Section 5.2). All calculations were carried out using the software package MINEA-R [30]. For each building configuration the maximum base shear $F_{b,max}$ and the maximum PGA are provided. A detailed summary of the verification results for the subsoil condition C-R is given in Annex 2. The nonlinear calculation results of the cantilever system and the system with full restraint are only given for the subsoil condition C-R. Furthermore, the calculations are carried out for the subsoil condition C-S for the most realistic structural system with partial restraint and moment redistribution factors according to Table 5-1. A detailed summary of the verification results for the subsoil condition C-R is given in Annex 2.

Table 6-2: Results of nonlinear verifications for the subsoil conditions C-R and C-S

		Cantilever, C-R		Partial restraint, C-R		Partial restraint , CS		Full restraint, C-R	
Building	Stories	$F_{b,max}$ [kN]	max PGA [m/s²]	$F_{b,max}$ [kN]	max PGA [m/s²]	$F_{b,max}$ [kN]	max PGA [m/s²]	$F_{b,max}$ [kN]	max PGA [m/s²]
01_TH-CB-1	1	72.1	2.30	142.0	2.10	142.0	1.57	142.0	2.10
02_TH-CB-2	2	76.7	1.74	153.3	2.28	153.3	1.34	249.8	2.04
03_FH-CB-2	2	133.9	1.22	215.4	1.55	215.4	0.91	408.4	1.90
04_FH-CB-3	3	141.5	1.04	227.7	1.31	227.7	0.78	561.4	1.84
05_MFH-CB-2	2	619.8	1.48	903.8	2.14	903.8	1.57	1380.1	2.64
06_MFH-CB-3	3	621.1	1.26	912.5	1.50	912.5	1.03	1663.4	2.12
07_MFH-CB-4	4	573.7	1.22	928.8	1.38	928.8	0.83	1882.7	1.98
08_TH-CS-1	1	74.7	2.28	130.8	2.35	130.8	1.67	149.3	2.27
09_TH-CS-2	2	86.2	1.75	151.7	2.34	151.7	1.37	258.6	2.26
10_FH-CS-2	2	149.7	1.24	240.4	1.56	240.4	0.93	408.5	2.18
11_FH-CS-3	3	151.7	1.07	249.4	1.34	249.4	0.81	527.1	2.04
12_MFH-CS-2	2	403.7	1.49	610.4	1.82	610.4	1.06	1018.9	2.34
13_MFH-CS-3	3	425.4	1.26	648.2	1.55	648.2	0.90	1375.4	2.23
14_MFH-CS-4	4	405.4	1.13	674.3	1.30	674.3	0.78	1693.5	2.16
15_TH-AC-1	1	54.3	2.41	107.6	1.93	107.6	1.38	107.6	1.93
16_TH-AC-2	2	55.9	1.82	107.9	1.44	107.9	0.85	166.7	1.90
17_FH-AC-2	2	115.4	1.04	188.9	1.31	188.9	0.78	281.1	1.68
18_FH-AC-3	3	105.7	1.01	191.3	1.15	191.3	0.69	333.1	1.55
19_MFH-AC-2	2	301.8	1.18	484.5	1.50	484.5	0.90	644.1	1.55
20_MFH-AC-3	3	252.2	0.96	456.9	1.26	456.9	0.75	764.2	1.44
21_MFH-AC-4	4	213.9	1.07	434.1	1.02	434.1	0.60	854.1	1.38
22_TH-LWC-1	1	92.2	1.45	156.0	1.98	156.0	1.42	180.1	2.21
23_TH-LWC-2	2	104.1	1.61	176.9	1.24	176.9	0.78	288.3	1.79
24_FH-LWC-2	2	145.8	1.21	217.1	1.44	217.1	0.86	286.2	1.69
25_FH-LWC-3	3	147.3	1.10	222.2	1.21	222.2	0.72	350.1	1.34
26_MFH-LWC-2	2	232.5	1.38	362.8	1.86	362.8	1.11	569.8	2.34
27_MFH-LWC-3	3	237.4	1.19	372.8	1.48	372.8	0.87	718.2	2.18
28_MFH-LWC-4	4	215.4	1.18	378.5	1.37	378.5	0.81	857.0	2.10

$F_{b,max}$ — *Total base shear [kN]*
max PGA — *Maximum attainable PGA ($a_{gR} \cdot S$)*

6.3 Summary of linear and nonlinear verifications

6.3.1 Linear verification results

The linear verifications based on the assumption of a cantilever system lead to rather low maximum attainable PGA values. Especially the maximum attainable PGA values for buildings with three or four floors are quite low. The results clarify, that the combination of the cantilever system and the linear verification wall by wall without force redistribution among the walls in combination with a conservative behaviour factor for ductility and dissipation cannot be successfully applied in case of seismic actions. The results are completely in line with the findings of the first project part [15], [16], [19], [20].

The maximum attainable PGA values are significantly increased, if moment redistribution due to partial and fully restraint is considered. In the case of a fully restraint the maximum attainable PGA values are almost three times higher in comparison to the cantilever system. However, this is just a theoretical high level of restraint to show the influence of the frame action controlled by the moment redistribution factor. The more realistic application of partial restraints leads to results between the two extreme assumptions of cantilever and full restraint. As a consequence of the considered frame actions, the slabs must be designed for additional moments.

Furthermore, it has to be pointed out, that the preliminary definition of frame actions with the proposed moment redistribution factors of Table 5-1 is only an estimation of the wall behaviour, as the level of restraint and moment redistribution is influenced by several factors not considered in the calculation approach. These factors can only be captured in a detailed three-dimensional model considering the interaction effects between walls and slab, which is not available so far.

6.3.2 Nonlinear verification results

The nonlinear verifications lead to much more realistic maximum attainable PGA values as already verified in the first project part [15], [16], [19], [20]. Furthermore, the results show higher maximum attainable PGA values with increasing levels of moment redistribution. However, the increase is limited by the deformation capacity of the building, as the displacement-based verification takes into account both, the deformation and load-bearing capacity.

6.3.3 Comparison of linear and nonlinear verification results

The comparison of the linear and nonlinear verification results shows clearly the deficiency of the linear verification approach. Although the nonlinear verification approach can be regarded as conservative, the maximum attainable PGA values for both subsoil conditions are significant higher in comparison to the linear verification. This fact is further illustrated by the comparison of the maximum attainable PGA values obtained by linear and nonlinear verification. Figure 6-2 shows the PGA values for the subsoil condition C-R for the structural types cantilever, partial and full restraint. On average, the maximum attainable PGA values are 4.6 times higher for cantilever systems, 3.4 times higher for systems with partial restraint and 2.9 times higher for systems with full restraint.

Additionally, the comparison for the subsoil condition C-S and the most realistic structural type partial restraint is shown in Figure 6-1. The spectrum with the subsoil condition C-S is characterised by a higher corner period T_C (Figure 5-2) and leads on average to only two times higher maximum attainable PGA values than the linear verification. Furthermore, for some particular building configuration (e.g. 08-TH-CS-01) linear and nonlinear verification results are very close to each other.

The application of higher restraint levels in combination with higher corner periods T_C shows an important shortcoming of the linear verification approach, as this approach does not take into account the limitation of the deformation capacity of the building. But keeping in mind, that higher restraint levels are usually reached at higher nonlinear deformation levels, the application of higher restraint levels in linear verifications is a crucial point. As higher restraint levels are the most influencing parameter for the linear safety verification, they must be applied carefully and on a reliable mechanical background with respect to the regarded building configuration – or even better they should be adequately reproduced in the calculation model. This is even more important for response spectra with higher corner periods T_c. For higher corner periods the displacement demand is increasing, which is captured quite well by the nonlinear verification approach. But the linear verification approach with higher restraint levels increases significantly the load-bearing capacity without taking the increasing displacement demand and the decreasing displacement capacity into account. This fact can lead to unsafe design results.

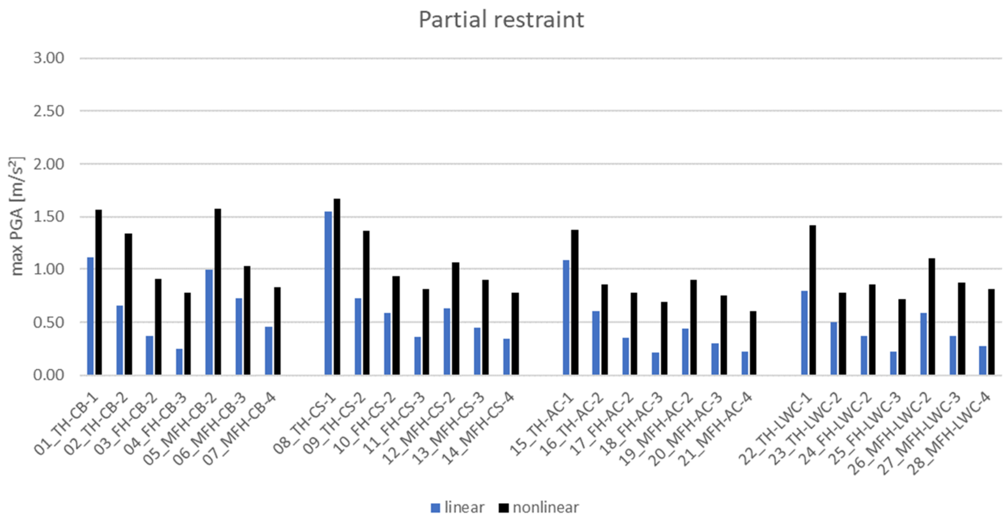


Figure 6-1: Maximum attainable PGA values for linear and nonlinear analyses – subsoil condition C-S

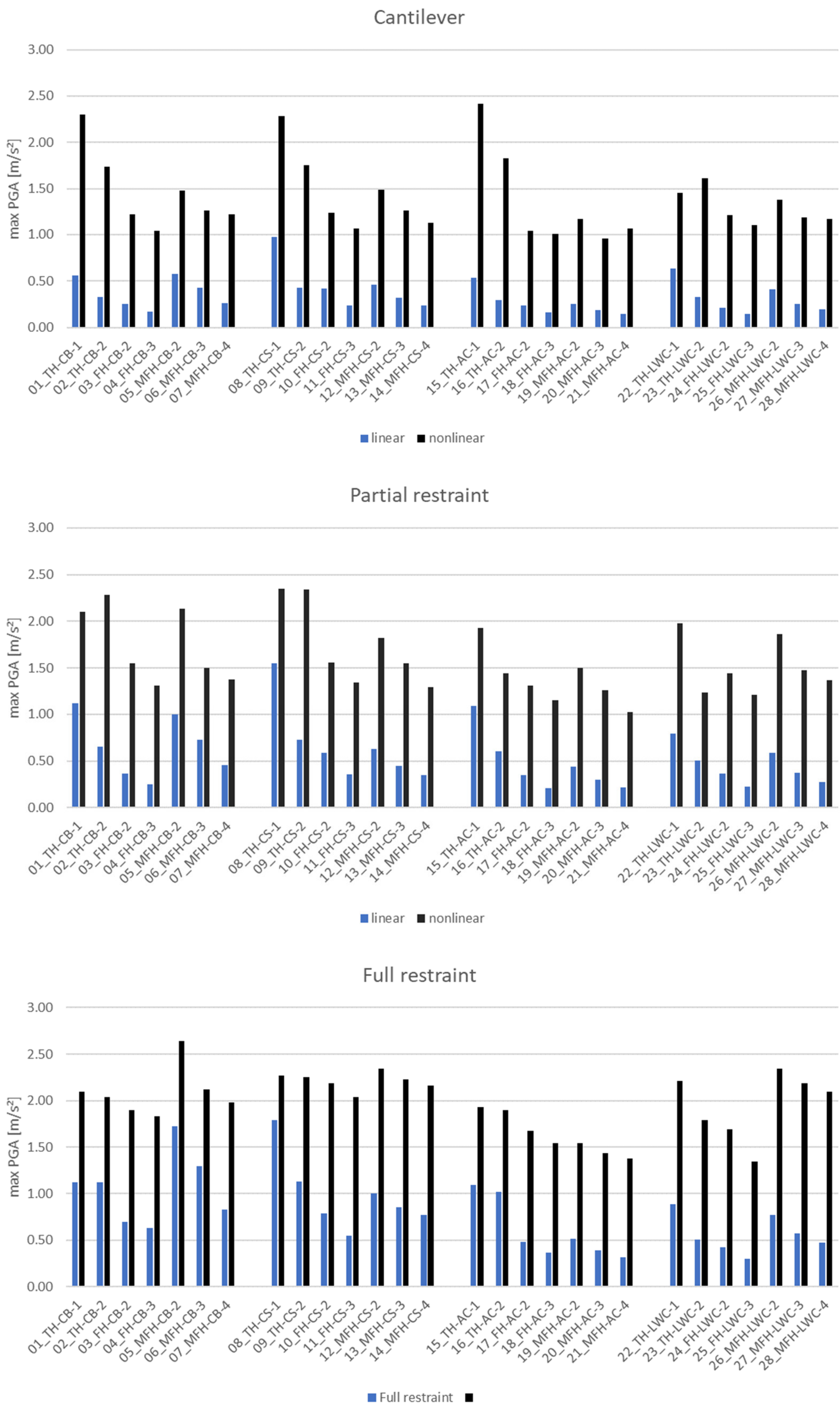


Figure 6-2: Maximum attainable PGA values for linear and nonlinear analyses – subsoil condition C-R

7 DERIVATION OF BEHAVIOUR FACTORS WITH PUSHOVER ANALYSIS

7.1 Behaviour factor q_R accounting for load redistribution

The behaviour factor q_R due to the overstrength by load redistribution is calculated according to Section 4.1. The resulting behaviour factors q_R for the 28 building configurations and each type of structural system (Section 5.2) are summarized in Table 7-1 and illustrated in Figure 7-1.

Table 7-1: Behaviour factors q_R accounting for load redistribution

		Cantilever			Partial restraint			Full restraint		
Building	**Stories**	**$F_{b,el}$ [kN]**	**$F_{b,max}$ [kN]**	**q_R [-]**	**$F_{b,el}$ [kN]**	**$F_{b,max}$ [kN]**	**q_R [-]**	**$F_{b,el}$ [kN]**	**$F_{b,max}$ [kN]**	**q_R [-]**
01_TH-CB-1	1	51.8	72.1	1.392	103.6	142.0	1.370	103.6	142.0	1.370
02_TH-CB-2	2	55.8	76.7	1.375	111.5	153.3	1.375	190.4	249.8	1.312
03_FH-CB-2	2	96.2	133.9	1.392	139.0	215.4	1.550	264.6	408.4	1.543
04_FH-CB-3	3	94.0	141.5	1.506	137.0	227.7	1.662	348.4	561.4	1.611
05_MFH-CB-2	2	311.5	619.8	1.990	535.6	903.8	1.688	930.6	1380.1	1.483
06_MFH-CB-3	3	332.3	621.1	1.869	570.5	912.5	1.600	1019.1	1663.4	1.632
07_MFH-CB-4	4	336.3	573.7	1.706	591.4	928.8	1.571	1064.5	1882.7	1.769
								0.0	0.0	
08_TH-CS-1	1	81.2	74.7	0.919	129.9	130.8	1.007	150.0	149.3	0.996
09_TH-CS-2	2	85.8	86.2	1.005	146.2	151.7	1.038	228.3	258.6	1.132
10_FH-CS-2	2	119.5	149.7	1.252	168.1	240.4	1.430	224.9	408.5	1.817
11_FH-CS-3	3	101.5	151.7	1.494	152.3	249.4	1.637	232.9	527.1	2.263
12_MFH-CS-2	2	301.4	403.7	1.339	412.7	610.4	1.479	658.5	1018.9	1.547
13_MFH-CS-3	3	308.8	425.4	1.377	432.4	648.2	1.499	830.4	1375.4	1.656
14_MFH-CS-4	4	309.0	405.4	1.312	445.3	674.3	1.514	990.6	1693.5	1.710
15_TH-AC-1	1	40.7	54.3	1.337	82.5	107.6	1.303	82.5	107.6	1.303
16_TH-AC-2	2	40.6	55.9	1.377	83.4	107.9	1.293	139.4	166.7	1.196
17_FH-AC-2	2	79.9	115.4	1.445	121.0	188.9	1.561	164.6	281.1	1.708
18_FH-AC-3	3	80.3	105.7	1.316	104.8	191.3	1.826	181.6	333.1	1.834
19_MFH-AC-2	2	174.4	301.8	1.730	300.3	484.5	1.613	353.6	644.1	1.822
20_MFH-AC-3	3	181.9	252.2	1.386	293.9	456.9	1.555	384.9	764.2	1.986
21_MFH-AC-4	4	183.0	213.9	1.169	283.7	434.1	1.530	411.8	854.1	2.074
22_TH-LWC-1	1	79.1	92.2	1.167	116.0	156.0	1.345	130.0	180.1	1.385
23_TH-LWC-2	2	90.4	104.1	1.152	138.0	176.9	1.282	138.0	288.3	2.089
24_FH-LWC-2	2	68.9	145.8	2.116	119.5	217.1	1.817	137.9	286.2	2.076
25_FH-LWC-3	3	70.6	147.3	2.086	107.6	222.2	2.066	144.5	350.1	2.422
26_MFH-LWC-2	2	150.6	232.5	1.544	215.5	362.8	1.684	283.0	569.8	2.014
27_MFH-LWC-3	3	136.6	237.4	1.738	201.1	372.8	1.854	307.3	718.2	2.337
28_MFH-LWC-4	4	139.7	215.4	1.542	194.6	378.5	1.944	334.4	857.0	2.563

$F_{b,el}$ — *Total base shear at the first wall failure of the linear calculation [kN]*
$F_{b,max}$ — *Maximum total base shear determined by pushover analysis [kN]*
q_R — *Load redistribution factor [-]*

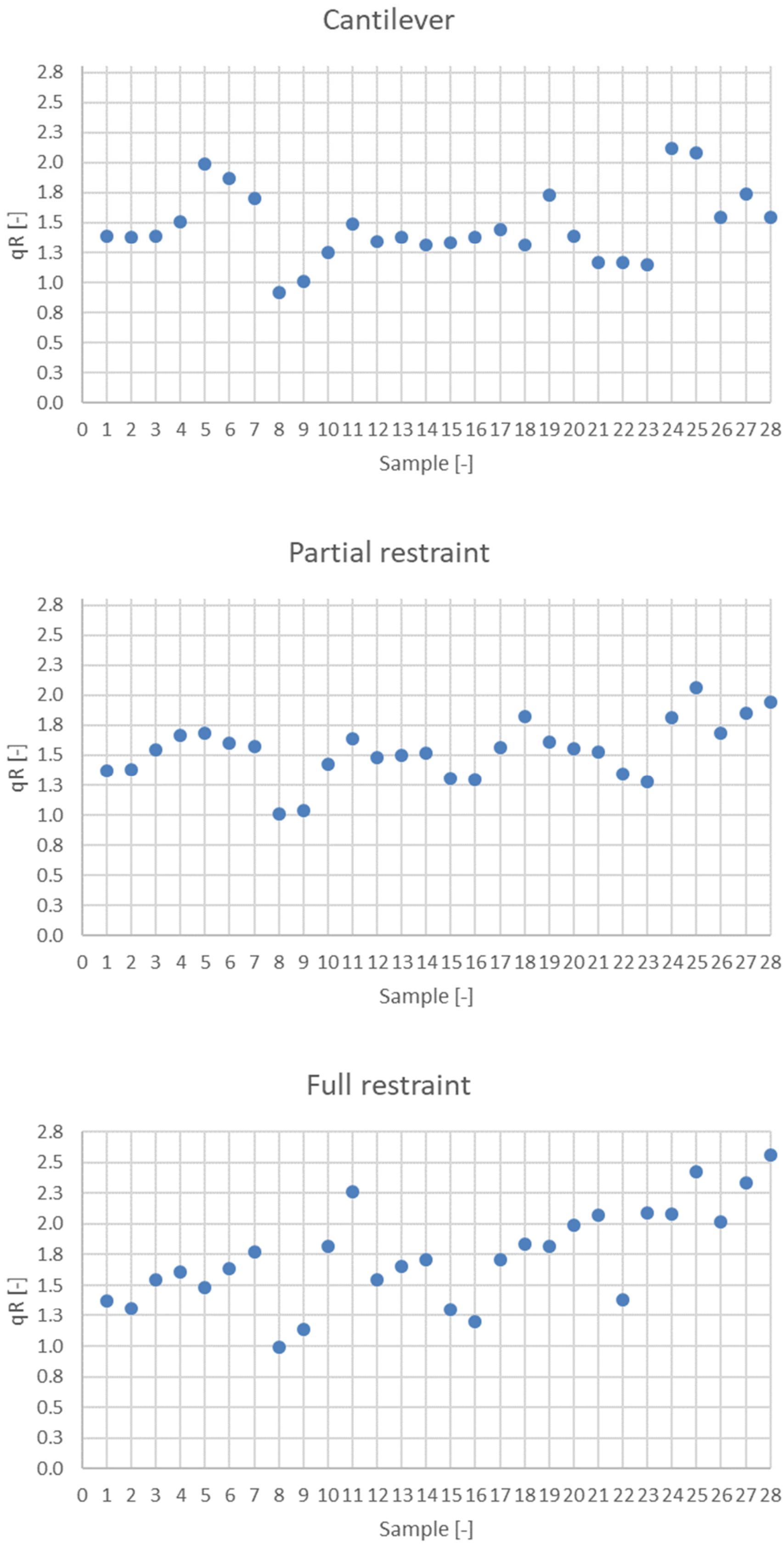
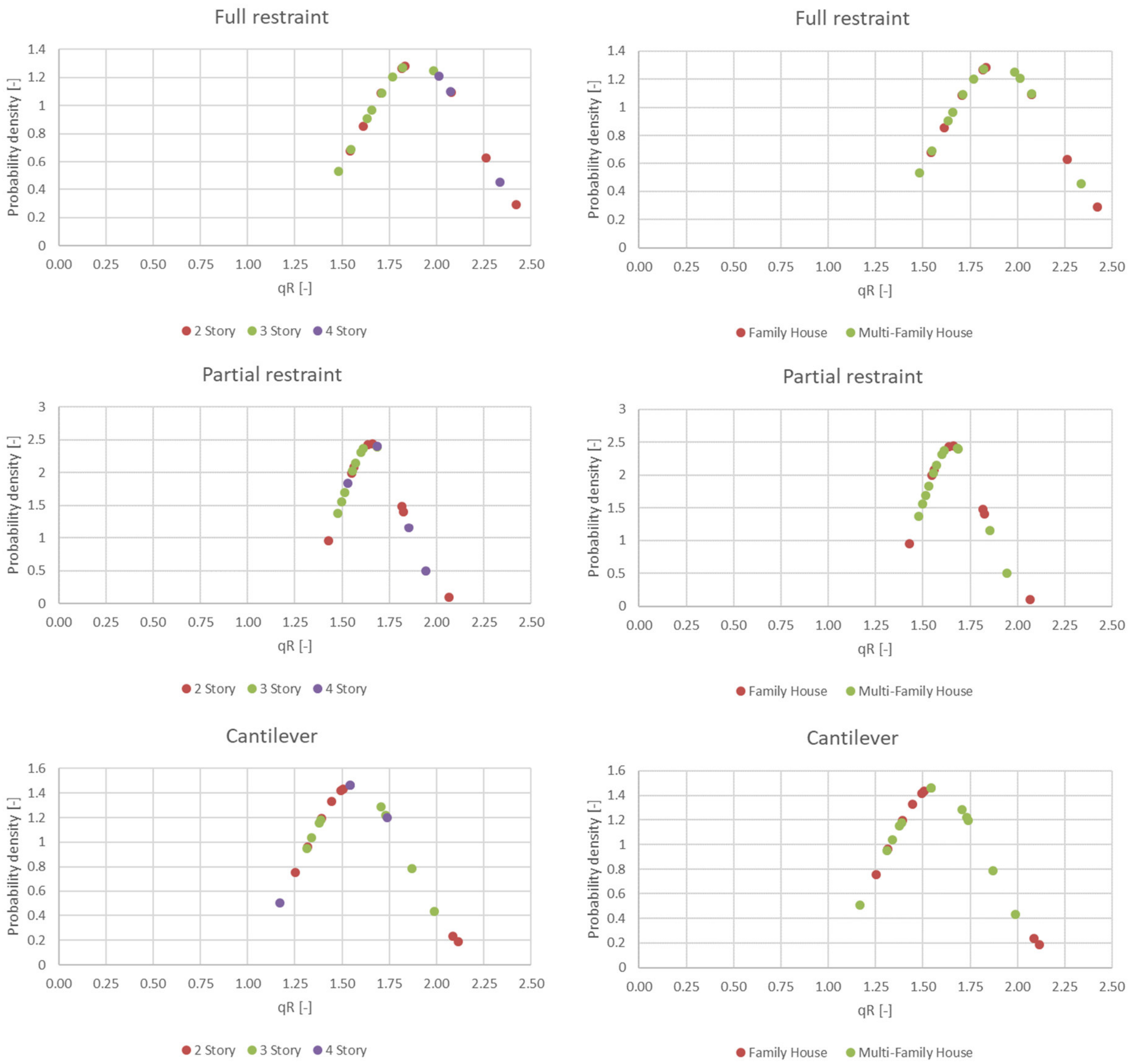


Figure 7-1: Distribution of behaviour factor q_R for each type of structural system

The statistical evaluation of the obtained behaviour factors q_R is summarized in Table 7-2. The evaluation of the terraced houses is carried out separately, as their horizontal load-bearing system consists only of few short shear walls and cannot be compared to the ground plan configurations of single-family and multi-family houses, especially with respect to load redistribution. The load distribution factors of the terraced houses 08-TH-CS-1 and 08-TH-CS-2 are close or equal to 1.0, since the load redistribution cannot take part due to similar wall lengths, identical wall thicknesses and identical material of the walls in transverse direction.

Table 7-2: Statistical evaluation of the behaviour factors q_R for terraced houses (TH), single-family houses (FH) and multi-family houses (MFH)

	Cantilever		**Partial restraint**		**Full restraint**	
Parameter	**TH**	**FH & MFH**	**TH**	**FH & MFH**	**TH**	**FH & MFH**
Sample size	8	20	8	20	8	20
Mean value	0.93	1.57	1.25	1.65	1.35	1.89
5%-percentile/Minimum	0.92	1.17	1.00	1.43	1.00	1.48
Standard derivation	0.17	0.27	0.14	0.16	0.31	0.31



(a) (b)

Figure 7-2: Normal distribution of q_R with respect to the building type, number of stories and structural system

The corresponding normal distributions of the results for single-family houses and multi-family houses are shown in Figure 7-2. The normal distributions are illustrated with respect to the building type (a) and the number of stories (b). The following findings can be summarized:

- The maximum mean value is obtained for systems with full restraint, while the lowest mean values results for the cantilever approach.
- The results of the systems with full restraint show the highest standard deviation.
- The scatter of the results is relatively low.
- No clear correlation of the behaviour factor q_R is observed to the number of stories.
- No clear correlation of the behaviour factor q_R is observed to the used types of bricks.
- Smaller load redistribution factors q_R are achieved for terraced houses.

7.2 Behaviour factor q_D accounting for deformation capacity and energy dissipation

The behaviour factor q_D due to deformation capacity and energy dissipation is calculated according to Section 4.2. The resulting behaviour factors q_D of the 28 building configurations and each type of structural system (Section 5.2) are illustrated in Figure 7-3 and summarized in Table 7-3.

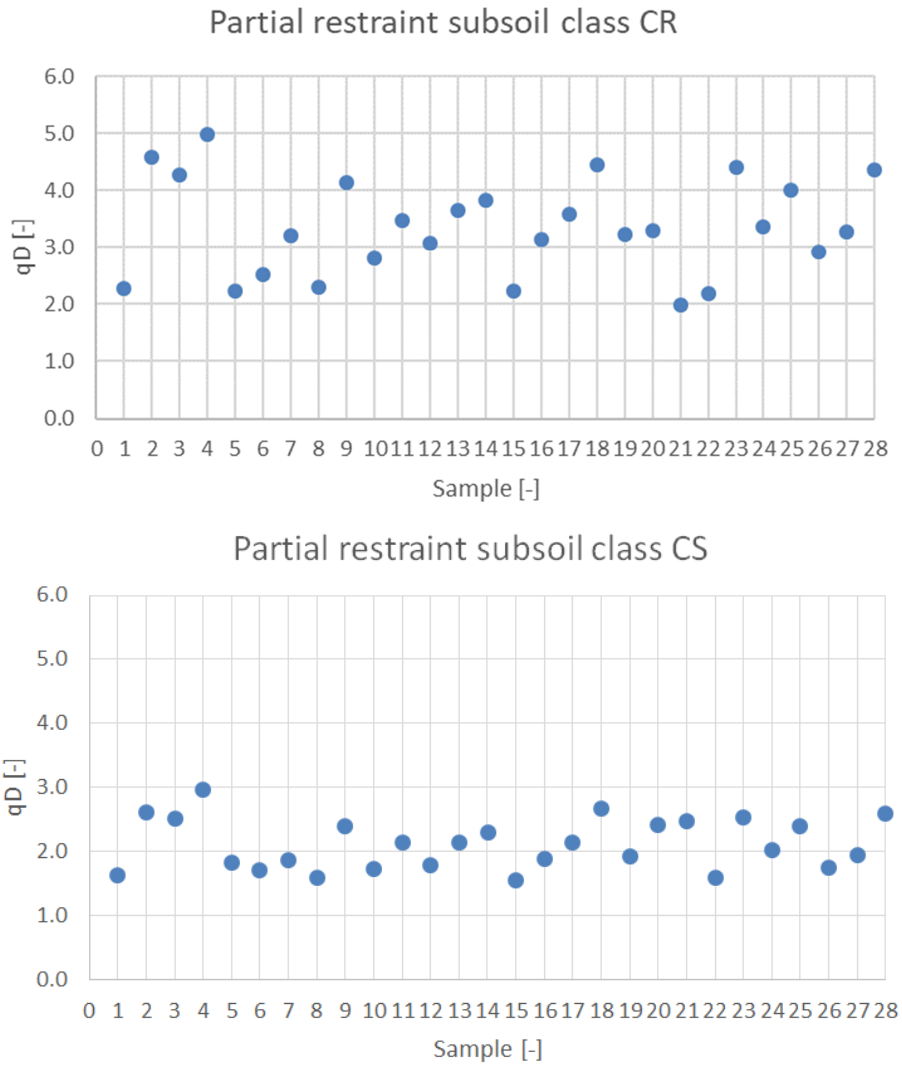


Figure 7-3: Distribution of q_D-factors for each type of structural system

Table 7-3: Behaviour factors q_D accounting for deformation capacity and energy dissipation

						Partial restraint, C-R			Partial restraint, C-S		
Building	**Stories**	α_{PP} [-]	α_{el} [-]	$M_{eff,PP}$ [t]	$M_{eff,el}$ [t]	$S_a(T_{PP})$ [m/s²]	$S_a(T_{el})$ [m/s²]	q_D [-]	$S_a(T_{PP})$ [m/s²]	$S_a(T_{el})$ [m/s²]	q_D [-]
01_TH-CB-1	1	1.00	1.00	59.2	59.2	2.32	5.30	2.28	2.40	3.92	1.63
02_TH-CB-2	2	1.00	1.15	110.2	104.1	1.35	5.70	4.59	1.39	3.35	2.60
03_FH-CB-2	2	1.01	1.16	216.5	204.6	0.99	3.87	4.23	0.99	2.28	2.49
04_FH-CB-3	3	1.02	1.21	318.4	290.5	0.71	3.27	4.96	0.72	1.95	2.95
05_MFH-CB-2	2	1.02	1.16	384.6	363.4	2.32	4.74	2.20	2.35	3.93	1.80
06_MFH-CB-3	3	1.04	1.21	562.7	513.9	1.59	3.75	2.51	1.62	2.58	1.69
07_MFH-CB-4	4	1.08	1.32	749.5	670.6	1.22	3.50	3.14	1.24	2.07	1.71
08_TH-CS-1	1	1.00	1.00	50.2	50.2	2.56	5.88	2.30	2.61	4.17	1.60
09_TH-CS-2	2	1.01	1.16	97.7	92.5	1.53	5.84	4.15	1.55	3.42	2.41
10_FH-CS-2	2	1.01	1.16	163.9	155.1	1.51	3.90	2.81	1.47	2.33	1.73
11_FH-CS-3	3	1.03	1.21	242.5	221.6	1.05	3.36	3.43	1.03	2.03	2.12
12_MFH-CS-2	2	1.01	1.17	375.2	355.0	1.63	4.56	3.07	1.63	2.66	1.78
13_MFH-CS-3	3	1.02	1.22	557.3	509.1	1.17	3.87	3.61	1.16	2.25	2.11
14_MFH-CS-4	4	1.05	1.24	738.1	660.5	0.91	3.24	3.76	0.91	1.95	2.25
15_TH-AC-1	1	1.00	1.00	48.5	48.5	2.17	4.83	2.23	2.21	3.45	1.56
16_TH-AC-2	2	1.04	1.15	88.8	84.0	1.20	3.60	3.14	1.21	2.18	1.88
17_FH-AC-2	2	1.04	1.15	196.1	185.2	0.96	3.27	3.56	0.96	1.95	2.12
18_FH-AC-3	3	1.08	1.21	282.4	259.4	0.67	2.85	4.38	0.67	1.73	2.61
19_MFH-AC-2	2	1.04	1.15	391.8	370.8	1.23	3.75	3.19	1.23	2.25	1.91
20_MFH-AC-3	3	1.09	1.21	565.2	519.7	0.80	2.55	3.25	0.80	1.88	2.36
21_MFH-AC-4	4	1.18	1.23	712.1	666.6	0.60	1.22	1.98	0.60	1.50	2.46
22_TH-LWC-1	1	1.00	1.00	70.3	70.3	2.26	4.95	2.19	2.22	3.54	1.59
23_TH-LWC-2	2	1.01	1.16	133.1	125.7	1.26	5.10	4.4	1.22	2.85	2.54
24_FH-LWC-2	2	1.02	1.16	187.7	177.5	1.16	3.60	3.3	1.16	2.15	2.00
25_FH-LWC-3	3	1.04	1.21	275.8	252.1	0.81	3.03	4.0	0.80	1.80	2.38
26_MFH-LWC-2	2	1.01	1.16	210.3	198.7	1.73	4.65	2.9	1.73	2.78	1.73
27_MFH-LWC-3	3	1.03	1.21	308.4	281.5	1.22	3.69	3.2	1.21	2.18	1.93
28_MFH-LWC-4	4	1.08	1.23	404.8	363.1	0.94	3.42	3.7	0.93	2.03	2.22

T_{el}	*Fundamental period of the linear system*
T_{PP}	*Fundamental period of the inelastic system at the performance point*
$S_a(T_{el})$	*Spectral acceleration assuming linear elastic behaviour*
$S_a(T_{PP})$	*Spectral acceleration of the inelastic system at the performance point*
α_{el}	*Participation factor of the linear system*
α_{PP}	*Participation factor of the inelastic system at the performance point*
$M_{eff,el}$	*Total effective mass of the linear system*
$M_{eff,PP}$	*Total effective mass of the inelastic system at the performance point*

The calculations are only carried out for the most realistic systems with partial restraint. Both subsoil conditions, C-R and C-S are considered. These two spectral shapes cover the maximum soil amplification and the maximum corner period T_c of the spectra to be applied in German earthquake regions. The results clarify, that the subsoil conditions C-S are decisive for most of the building configurations, as a higher reduction of the elastic spectra is required to get an intersection point of the building capacity curve with the seismic demand.

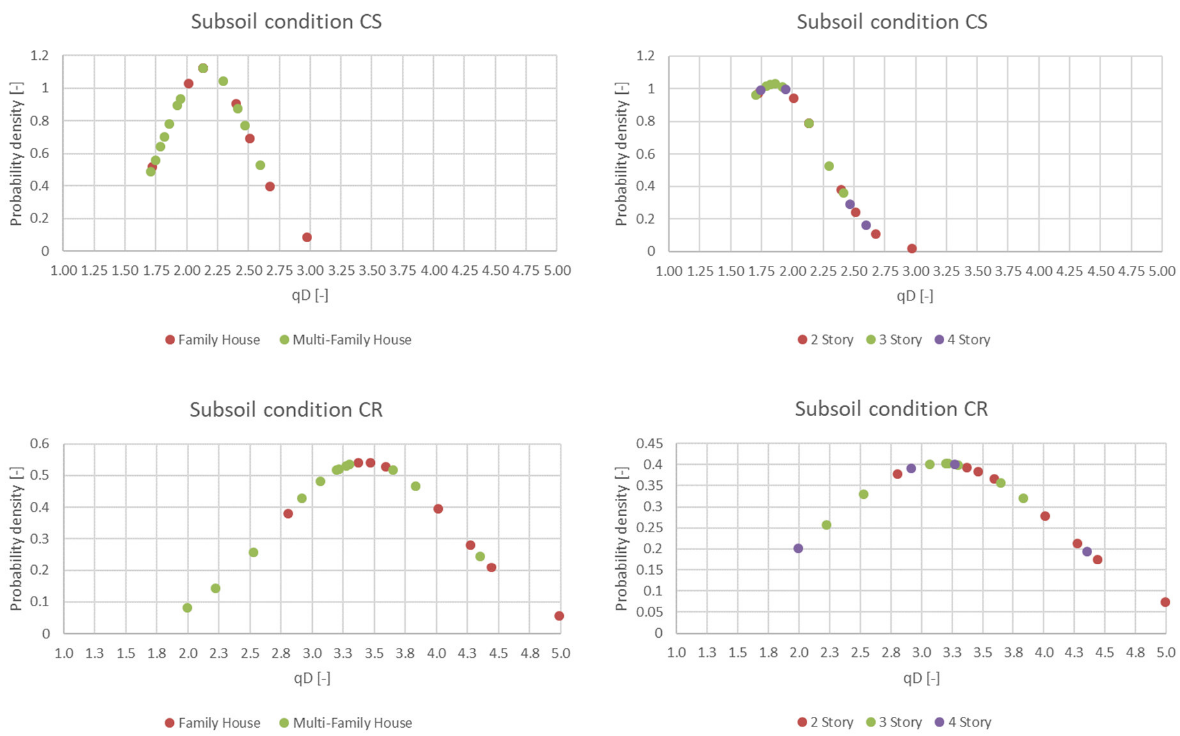
The statistical evaluation of the calculated behaviour factors q_D is summarized in Table 7-4. The evaluation of the terraced houses is carried out separately, as their horizontal load-bearing system consists only of few short shear walls and cannot be compared to the ground plan configurations of single-family and multi-family houses. In addition, the terraced houses for one and two stories are evaluated separately, as the standard terraced house has usually two stories. The evaluation results for the single-storey terraced houses are only used to derive the behaviour factor q_D for single-storey buildings, but they cannot be regarded as representative buildings in practise.

Table 7-4: Statistical evaluation of the behaviour factor q_D for terraced houses (TH), single family (FH) and multi-family houses (MFH)

	Subsoil condition C-S			Subsoil condition C-R		
Parameter	**TH (1 storey)**	**TH (2 stories)**	**FH & MFH**	**TH (1 storey)**	**TH (2 stories)**	**FH & MFH**
Sample size	4	4	20	4	4	20
Mean value	1.56	2.35	2.16	2.25	4.07	3.42
5%-percentile/Minimum	1.55	1.88	1.71	2.19	3.14	2.00
Standard derivation	0.03	0.29	0.35	0.04	0.56	0.74

The normal distributions of the behaviour factor q_D are shown on the example of single-family and multi-family houses in Figure 7-4. The normal distributions are illustrated with respect to the building type (a) and the number of stories (b). The following findings can be summarized:

- The mean values for subsoil condition C-R are greater than the mean values of C-S.
- The results for subsoil condition C-R show higher deviations in comparison to C-S.
- Smaller behaviour factor q_D are achieved for terraced houses with single stories.
- No clear correlation of the behaviour factor q_D is observed to the number of stories.
- No correlation of the behaviour factor q_D is observed to the used types of bricks.



(a) (b)

Figure 7-4: Normal distribution of the behaviour factor q_D with respect to the building type and number of stories

7.3 Behaviour factor q_S accounting for all other sources of overstrength

The behaviour factor q_S considers the fact that the design values are always conservative lower bound estimates of the actual probable strengths of the structural materials and their effective strengths in the as-constructed structure. In case of masonry structures the source of further overstrength is limited. Therefore, it is proposed to estimate the behaviour factor q_S as the ratio of the maximum total shear force calculated with linear analyses using mean values to the maximum total shear force calculated with linear analyses and characteristic values, as already introduced in Section 4.3. However, the behaviour factor q_S is assumed here to 1.0, as rather low ratios are expected. Furthermore, no sufficiently statistically verified investigations can be carried out within the scope of this study. The determination of this factor requires further investigations in future works.

8 BEHAVIOUR FACTORS FOR MASONRY BUILDINGS IN GERMANY

The resulting behaviour factor q for unreinforced masonry buildings in German earthquake regions can be determined with respect to the draft of DIN EN 1998-1-2018 [3]:

$$q = q_R \cdot q_D \cdot q_S$$

q_R Overstrength due to redistribution

q_D Deformation capacity and energy dissipation

q_S Overstrength due to all other sources

The three components of the resulting behaviour factor q are presented in Table 8-1Table 8-1 for terraced houses and in Table 8-2 for single-family and multi-family houses. The application of both tables requires regularity in elevation.

Table 8-1: Behaviour factors q_R, q_D, q_S for two storey terraced houses made of unreinforced masonry

Behaviour factor accounting for load redistribution effects	$q_R = 1.0$*
Behaviour factor accounting for deformation capacity and energy dissipation	$q_D = 1.9$
Behaviour factor accounting for overstrength due to all other sources	$q_S = 1.0$
Resulting behaviour factor *q*	$\boldsymbol{q = 2.0 - 2.4}$

** The standard value is 1.0. Higher values may be applied, provided that they are confirmed through a global nonlinear static analysis of the building. The maximum value to be used in the design is equal to 1.25.*

Table 8-2: Behaviour factors q_R, q_D, q_S for two and more storey single-family and multi-family houses made of unreinforced masonry

Behaviour factor accounting for load redistribution effects	$q_R = 1.4$
Behaviour factor accounting for deformation capacity and energy dissipation	$q_D = 1.7$
Behaviour factor accounting for overstrength due to all other sources	$q_S = 1.0$
Resulting behaviour factor *q*	$\boldsymbol{q = 2.4}$

In the particular case of single-storey buildings the energy dissipation and the redistribution of the forces after the first damage in the walls is limited. Therefore, the behaviour factors specified in Table 8-1 shall be applied.

Table 8-3: Behaviour factors q_R, q_D, q_S for single-storey unreinforced masonry buildings

Behaviour factor accounting for load redistribution effects	$q_R = 1.25$*
Behaviour factor accounting for deformation capacity and energy dissipation	$q_D = 1.5$
Behaviour factor accounting for overstrength due to all other sources	$q_S = 1.0$
Resulting behaviour factor *q*	$\boldsymbol{q = 1.9}$

** The q_R value for terraced houses must be reduced to 1.0. Higher values may be applied, provided that they are confirmed through a global nonlinear static analysis of the building. The maximum value to be used in the design is equal to 1.25.*

The behaviour factors shall be reduced in case of irregularities:

- For buildings which are not regular in elevation, the value of q_D should be reduced by 20%, but need not be taken less than 1.5.

For buildings which are not regular in plan, q_R may be calculated as the average of 1.0 and the values specified in Table 8-1, Table 8-2 and Table 8-3.

Furthermore, the following requirement shall be fulfilled:

- The slabs must be designed for the frame actions due to the considered moment redistribution according to Section 5.2.

In addition, the terraced houses shall satisfy the following constructive rules:

- The torsional stiffness of the terraced house shall be guaranteed by a minimum of two parallel continuous exterior walls in longitudinal building direction, the length of each wall being greater than 30% of the building length in longitudinal direction.
- The arrangement of at least two parallel interior walls with a minimum length of $l \geq 1.50$ m in transverse building direction is required. In addition, a minimum of four exterior parallel walls with a minimum length of 1.00 m are to be arranged in transverse direction.

Higher values as the tabulated behaviour factors q_R and q_D may be applied, if they are verified through the results of a global nonlinear static analysis. The derivation of the behaviour factors shall be carried out according to the procedure presented in Section 4. The maximum behaviour factor that may be used in the design shall be limited to $q = 3.0$.

9 APPLICATION TO THE BUILDINGS OF THE PARAMETRIC STUDY

9.1 Terraced houses

The proposed behaviour factor q is applied to the terraced houses of the parametric study. For this purpose, the linear verifications, as described in the report WP1-RWTH [12], are performed with characteristic material strengths and a partial safety factor of $\gamma_M = 1.2$ for masonry strength parameter. The following behaviour factors are applied for two storey buildings according to Table 8-2:

$$q_R = 1.00$$

$$q_D = 1.90$$

$$q_S = 1.00$$

For single-storey buildings the behaviour factors according to Table 8-3 are applied:

$$q_R = 1.00$$

$$q_D = 1.50$$

$$q_S = 1.00$$

9.2 Single-family and multi-family houses

The proposed behaviour factor q is applied to the single-family and multi-family houses of the parametric study. For this purpose, the linear verifications, as described in the report WP1-RWTH [12], are performed with characteristic material strengths and a partial safety factor of $\gamma_M = 1.2$ for masonry. According to Table 8-2 the following behaviour factors are applied:

$$q_R = 1.40$$

$$q_D = 1.70$$

$$q_S = 1.00$$

9.3 Comparison of linear and nonlinear calculation results

The results are summarized in Table 9-1 in terms of the maximum attainable PGA value. The linear and nonlinear calculation results are compared for the subsoil condition C-S considering all requirements of DIN EN 1998-1/NA-2018, Section 4.3.3.4.2.1(1) [11]. The comparison shows, that the verification results are still on the safe side as the maximum attainable PGA values of the nonlinear verifications are on average 1.41 times higher for the terraced houses and 1.63 times higher for the single-family and multi-family houses.

Table 9-1: Comparison of linear and nonlinear verifications for the 28 building configurations considering q_R, q_D, q_S

Building	Stories	q_R [-]	q_S [-]	q_D [-]	max PGA linear [m/s²]	max PGA nonlinear with γ = 1.5 (C-S) [m/s²]
01_TH-CB-1	1	1.00	1.00	1.50	0.798	1.548
02_TH-CB-2	2	1.00	1.00	1.90	0.608	1.036
03_FH-CB-2	2	1.40	1.00	1.70	0.428	0.720
04_FH-CB-3	3	1.40	1.00	1.70	0.295	0.618
05_MFH-CB-2	2	1.40	1.00	1.70	1.180	1.560
06_MFH-CB-3	3	1.40	1.00	1.70	0.942	1.008
07_MFH-CB-4	4	1.40	1.00	1.70	0.514	0.804
08_TH-CS-1	1	1.00	1.00	1.50	1.440	1.590
09_TH-CS-2	2	1.00	1.00	1.90	0.859	1.080
10_FH-CS-2	2	1.40	1.00	1.70	0.714	0.870
11_FH-CS-3	3	1.40	1.00	1.70	0.438	0.660
12_MFH-CS-2	2	1.40	1.00	1.70	0.771	0.972
13_MFH-CS-3	3	1.40	1.00	1.70	0.543	0.679
14_MFH-CS-4	4	1.40	1.00	1.70	0.438	0.624
15_TH-AC-1	1	1.00	1.00	1.50	0.774	1.320
16_TH-AC-2	2	1.00	1.00	1.90	0.555	0.702
17_FH-AC-2	2	1.40	1.00	1.70	0.390	0.624
18_FH-AC-3	3	1.40	1.00	1.70	0.238	0.552
19_MFH-AC-2	2	1.40	1.00	1.70	0.562	0.762
20_MFH-AC-3	3	1.40	1.00	1.70	0.352	0.600
21_MFH-AC-4	4	1.40	1.00	1.70	0.257	0.358
22_TH-LWC-1	1	1.00	1.00	1.50	0.642	1.338
23_TH-LWC-2	2	1.00	1.00	1.90	0.479	0.756
24_FH-LWC-2	2	1.40	1.00	1.70	0.428	0.732
25_FH-LWC-3	3	1.40	1.00	1.70	0.257	0.582
26_MFH-LWC-2	2	1.40	1.00	1.70	0.619	1.032
27_MFH-LWC-3	3	1.40	1.00	1.70	0.390	0.750
28_MFH-LWC-4	4	1.40	1.00	1.70	0.286	0.660

10 APPLICATION TO THE EMILIA ROMAGNA BUILDINGS

In the following linear verifications of the buildings Po1 and Pr3, investigated in first project part [15], [16], [19], [20] are carried out with the proposed behaviour factor *q*. The two buildings are located in the Emilia Romagna region in Northern Italy and survived the seismic events May 20 and 29, 2012 without any damages and cracks. For these buildings the maximum attainable PGA values are calculated with respect to the site-specific response spectra of the May 29 seismic event [16].

10.1 Building Po1

Site specific spectrum:	$S_{e,max} = 7.71\ m/s^2$
	$S_e(T=0) = 2.84\ m/s^2$
	$\beta = \frac{S_{d,max}}{S_e(T=0)} = \frac{7.71}{2.84} = 2.71$
Design spectral acceleration:	$S_{d,max} = 0.79\ m/s^2$
Requirements fulfilled:	yes
Moment distribution factor:	Partial restraint according to Table 5-1
Behaviour factors:	$q_D = 1.7\ , q_R = 1.4,\ q_S = 1.0$
Elastic spectral acceleration:	$S_{e,max} = 0.79 \cdot 1.4 \cdot 1.7 \cdot 1.0 = 1.88\ m/s^2$
Maximum attainable PGA:	$S_e(T=0) = \frac{S_{e,max}}{\beta} = 0.69\ m/s^2$

The maximum attainable PGA of 0.69 m/s^2 corresponds to 24% of the estimated site-specific PGA and to 66% of the maximum attainable PGA of 1.04 m/s^2 calculated by nonlinear verification using damped spectra. Compared to the linear verification using a cantilever model, the maximum attainable PGA is increased about nine times.

10.2 Building Pr3

Site specific spectrum:	$S_{e,max} = 3.13\ m/s^2$
	$S_e(T=0) = 1.45\ m/s^2$
	$\beta = \frac{S_{d,max}}{S_e(T=0)} = \frac{3.13}{1.45} = 2.16$
Design spectral acceleration:	$S_{d,max} = 0.42\ m/s^2$
Requirements fulfilled:	yes
Moment distribution factor:	Partial restraint according to Table 5-1
Behaviour factors:	$q_D = 1.7,\ q_R = 1.4,\ q_S = 1.0$
Elastic spectral acceleration:	$S_{e,max} = 0.42 \cdot 1.4 \cdot 1.7 \cdot 1.0 = 1.0\ m/s^2$
Maximum attainable PGA:	$S_e(T=0) = \frac{S_{e,max}}{\beta} = 0.46\ m/s^2$

The maximum attainable PGA of 0.46 m/s^2 corresponds to 32% of the estimated site-specific PGA and to 62% of the maximum attainable PGA of 0.74 m/s^2 calculated by nonlinear verification using damped spectra. Compared to the linear verification using a cantilever model, the maximum attainable PGA is increased about three times.

11 COMPARISON OF ALL CALCULATION APPROACHES

Finally, the calculation results of the following verification approaches are compared with each other:

- Shear wall tables according to DIN 1998-1/NA-2011 [7]
- Shear wall tables according to the proposal of Kassel University [28], [29]
- Linear forced based verification with cantilever system and recent q-factor concept
- Pushover analysis with subsoil conditions C-R
- Pushover analysis with subsoil conditions C-S
- Linear calculation with the new proposal for the behaviour factor *q*

The comparison of the different verification approaches is illustrated in Figure 11-1 and Figure 11-2 and can be summarized as follows:

- The new proposal with increased q-factors and moment distribution leads to much more realistic maximum attainable PGA values. The mean value is about two times larger in comparison to the traditional approach using a cantilever model. But there is still a gap to the verification results of the pushover analyses, which better utilize nonlinear load-bearing reserves. However, the safety margin seems reasonable, as the q-factors are only an estimation of the nonlinear behaviour.

- The application of the recent shear wall tables in DIN 1998-1/NA-2011 [7] leads to verification results between the linear calculation based on a cantilever system and the new q-factor proposal. The shear wall tables can be regarded as conservative for most of the typical building configurations. Furthermore, it has to be pointed out, that several calculations failed because of the limitations to apply the table (e.g. strength class 2).

- The comparison calculations clarify, that the application of the shear wall tables proposed by Kassel University leads to verification results close to the verifications based on pushover analysis in case of unfavourable subsoil conditions. Furthermore, the results show that the proposed shear wall tables can exceed the maximum attainable PGA values of the corresponding pushover analyses.

- The following problems arise during the application of the new shear wall table with higher levels of restraints to the typical 28 masonry buildings:

  - Reference lengths < 2.5m and >1.3m for three- and four-story buildings are not covered
  - Reference lengths >> 2.5m are not covered by the proposed table
  - Minimum reference length for one and two storey buildings was not sufficient for all buildings
  - The minimum wall length of 0.4 x L_{max} caused problems. Modifications of the wall arrangements were required (e.g. subdivision of a longer wall) to successfully apply the tables. The interpretation in practice is questionable.

- As a consequence of the high levels of restraints, additional moments are acting on the slabs. This effect is considered in the provided shear wall tables [28], [29] by moment distribution factors on each storey level. The check of the moment distribution factors is not the objective of this report. However, it would be helpful to provide simple constructive measures (e.g. slab reinforcement) in the code, as the old code version DIN EN 1998-1/NA [7] does not provide any simple rules for the slabs. Otherwise, the slabs must be designed for the additional bending moments. This is an additional effort and contradictory to the application of simple tables.

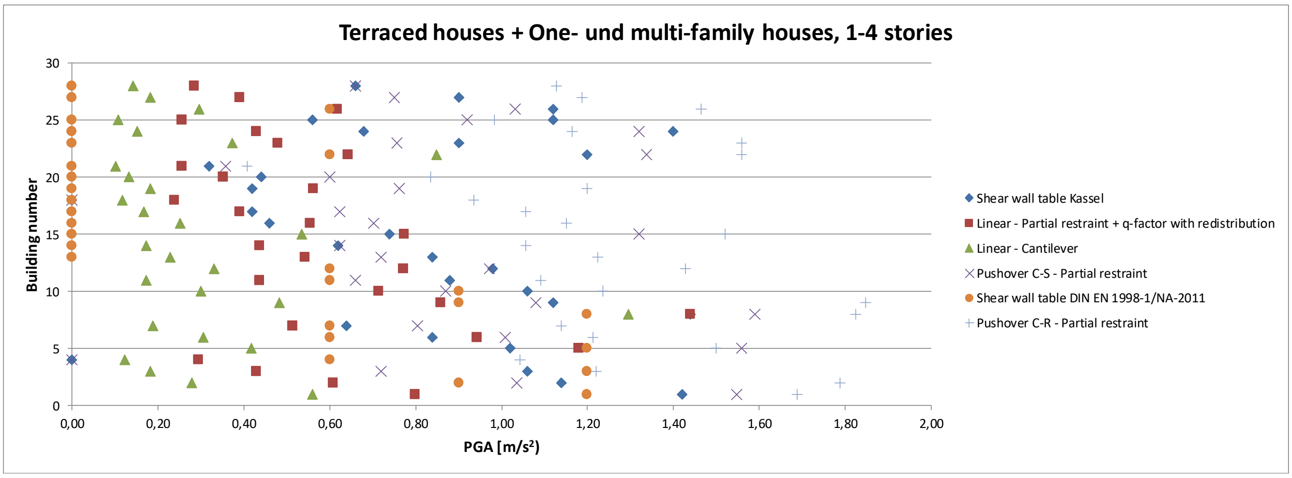


Figure 11-1: Results of the investigated 28 building configurations for all verification approaches

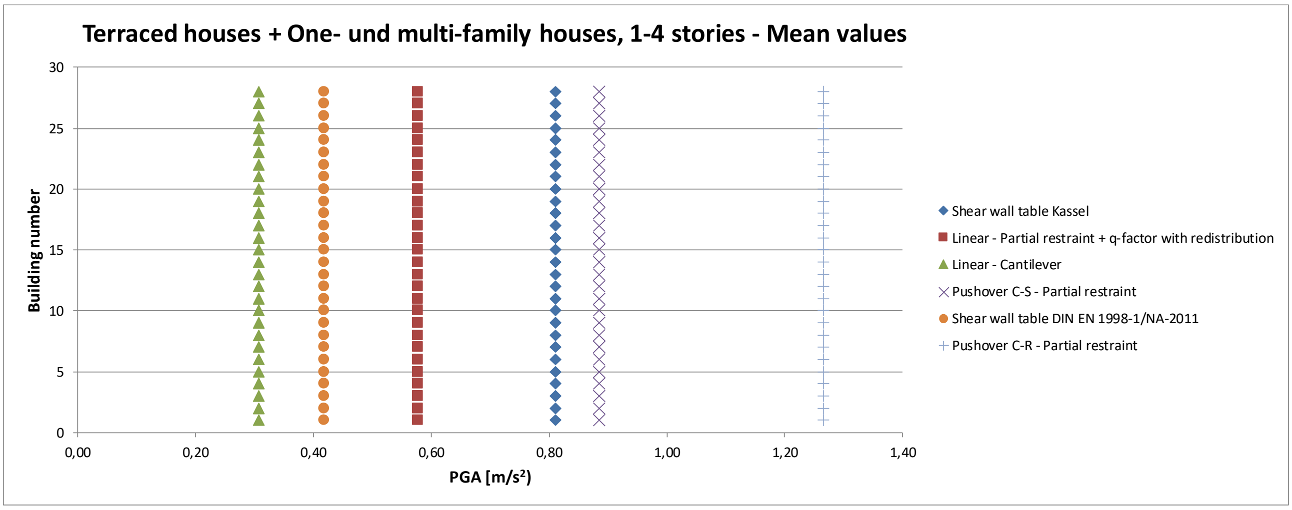


Figure 11-2: Mean values of maximum attainable PGA for the 28 building configurations

The comparison of the tables is carried out without considering eccentricities of the outer walls, as the nonlinear calculations only consider the minimum eccentricities (WP1-RWTH [12]). All buildings were checked according to the requirements to apply the tables. Some of the buildings were slightly modified to fulfil the requirements (Table 11-1). The buildings 4 and 18 are not considered in Figure 11-1 and Figure 11-2, as they do not match the minimum wall lengths. If the percentage of shear wall areas differs more than 20% the bracket values for terraced houses given in the tables are applied.

Figure 11-4 to Figure 11-6 show the comparison of the required shear wall areas according to DIN EN 1998-1 [6], DIN 1998-1/NA-2011 [7] and the shear wall tables according to Kassel University [28], [29] for strength class 12. The comparison confirms the more optimistic approach with higher attainable PGA-values according to the new proposal [28], [29].

Table 11-1: Modifications and reference lengths to meet the requirements to apply the shear wall tables

Building	Remarks
01_TH-CB-1	L_{ref} = 1.30m < 1.3m, wall lengths increased (l = 1,1m, l = 1,7m) - bracket values
02_TH-CB-2	L_{ref} = 1.30m < 1.3m, wall lengths increased (l = 1,1m, l = 1,7m) bracket values
03_FH-CB-2	L_{ref} = 1.76m - bracket values
04_FH-CB-3	L_{ref} = 1.76m < 2.5m: **Not applicable**
05_MFH-CB-2	L_{ref} = 3.9m >> 2.5m
06_MFH-CB-3	L_{ref} = 3.9m >> 2.5m
07_MFH-CB-4	L_{ref} = 3.9m >> 2.5m
08_TH-CS-1	L_{ref} = 1.48m
09_TH-CS-2	L_{ref} = 1.48m
10_FH-CS-2	L_{ref} = 2.25m >> 1.3m
11_FH-CS-3	L_{ref} = 2.25m < 2.5m: slightly violated
12_MFH-CS-2	L_{ref} = 2.50m (bracket values)
13_MFH-CS-3	L_{ref} = 2.50m (bracket values)
14_MFH-CS-4	L_{ref} = 2.50m (bracket values)
15_TH-AC-1	L_{ref} = 1.26m
16_TH-AC-2	L_{ref} = 1.26m
17_FH-AC-2	L_{ref} = 1.77m > 1.3
18_FH-AC-3	L_{ref} = 1.77m < 2.5m: **Not applicable**
19_MFH-AC-2	L_{ref} = 2.16m >> 1.3 - bracket values
20_MFH-AC-3	L_{ref} = 2.53m (Wall 31, 32, 23 substituted by two walls of 2.91, W7 not considered) - bracket values
21_MFH-AC-4	Lref = 2.53m (Wall 31, 32, 23 substituted by two walls of 2.91, W7 not considered) - bracket values
22_TH-LWC-1	L_{ref} = 1.41m
23_TH-LWC-2	L_{ref} = 1.41m
24_FH-LWC-2-X	L_{ref} = 2.79m
25_FH-LWC-3-X	L_{ref} = 2.79m
24_FH-LWC-2-Y	Lref = 2.53m (Wall 3 subdivided) - bracket values
25_FH-LWC-3-Y	Lref = 2.53m (Wall 3 subdivided) - bracket values
26_MFH-LWC-2	L_{ref} = 3.69m >> 2.5m
27_MFH-LWC-3	L_{ref} = 3.69m >> 2.5m
28_MFH-LWC-4	L_{ref} = 3.69m >> 2.5m

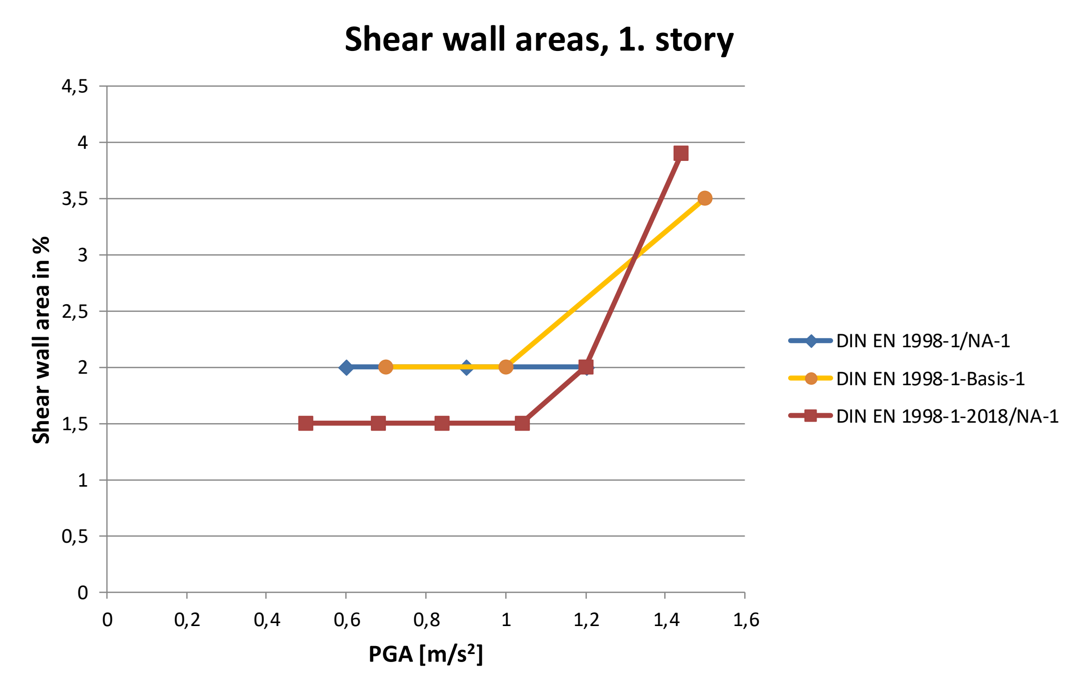


Figure 11-3: Required shear wall areas of the different code approaches in terms of PGA – 1 story

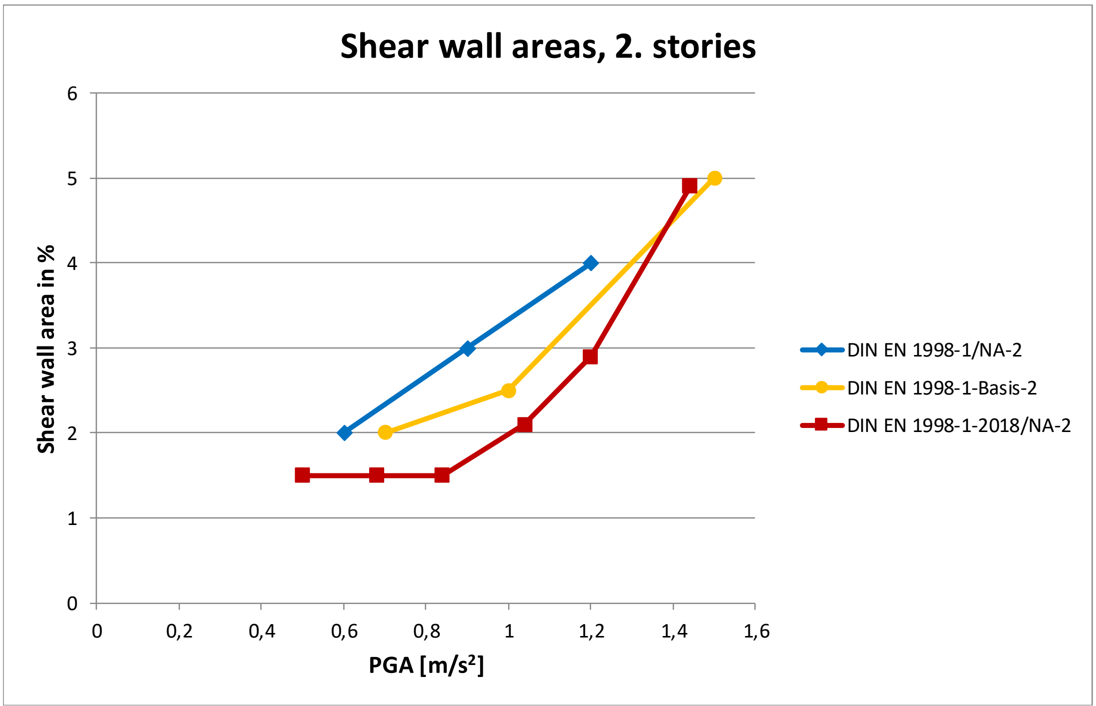


Figure 11-4: Required shear wall areas of the different code approaches in terms of PGA – 2 stories

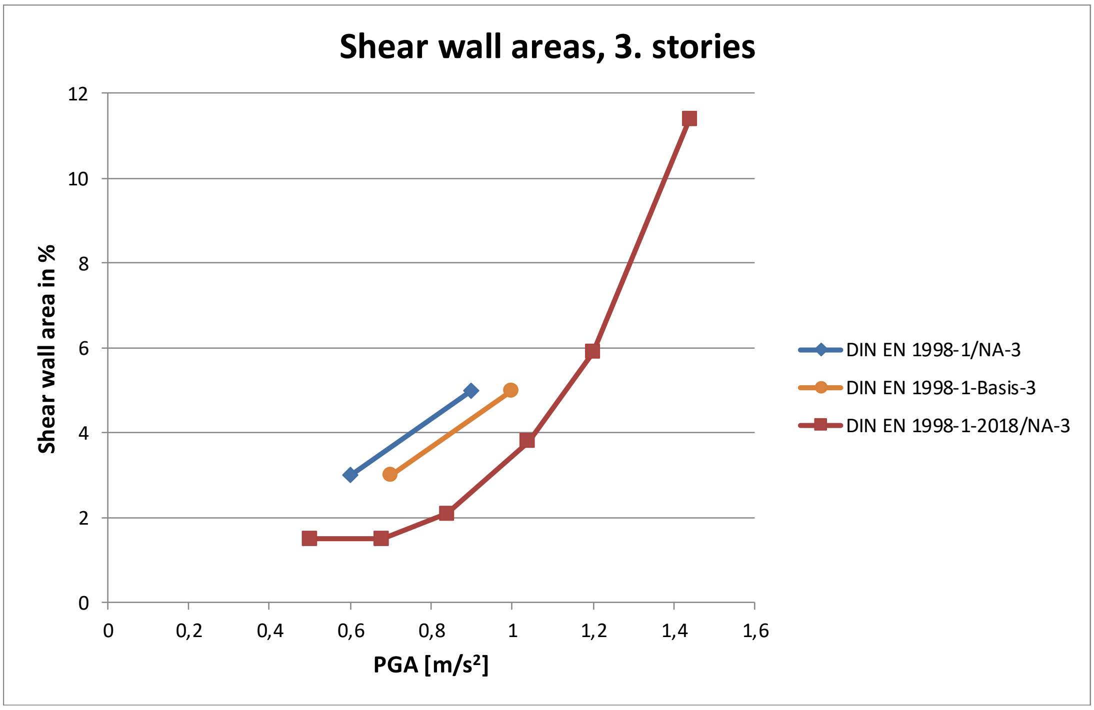


Figure 11-5: Required shear wall areas of the different code approaches in terms of PGA – 3 stories

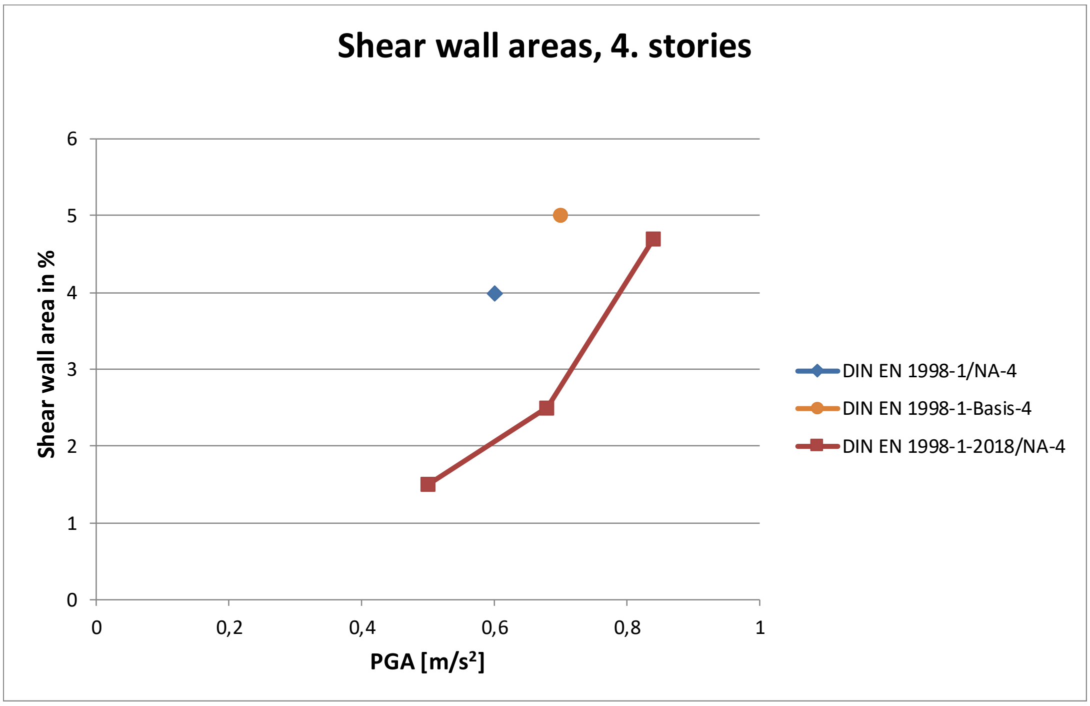


Figure 11-6: Required shear wall areas of the different code approaches in terms of PGA – 4 stories

As the comparison based on the 28 building configurations is quite difficult because of the complex wall arrangements, different strength classes and interpolations between them, the results of the newly developed shear wall tables are compared with the nonlinear calculation results on the example of a simple ground plan configuration (Figure 11-7). It is assumed, that all walls are made of clay bricks of strength class 12. Three wall lengths of 1.5m, 2.0m and 2.5m are considered. The dimensions of the ground plan are 10mx10m. For each building configurations the partial restraint used for the derivation of the new q-factors with redistribution and the larger restraint levels applied for the development of the new shear wall tables are investigated. All calculations are carried out for the subsoil conditions C-S with a broader plateau range. The comparison in Table 11-2 clarifies, that the pushover analysis with the partial restraint lead to smaller maximum attainable PGA-values than the shear wall tables. The PGA values given in the brackets are determined by using the code document NA 005-51-06 AA N 1159. The new tables lead to slightly higher PGA values, but the differences are negligible.

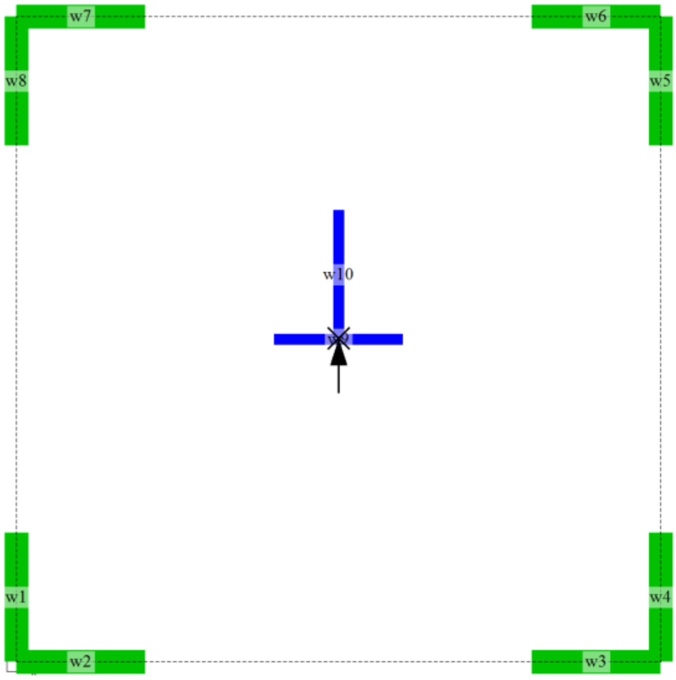


Figure 11-7: Wall arrangement of the simple ground plan

Table 11-2: Comparison of the maximum attainable PGA values [m/s^2] based on pushover analysis and shear wall tables

Level of restraint	**PGA [m/s^2] Pushover**	**PGA [m/s^2] Shear wall table [28], [29] and NA 005-51-06 AA N 1159**	**Ratio: Table/Pushover**
System, l = 1,5 m, 2 stories			
0.85	0.68	0.96 (1.04)	1.41
0.68	0.84	0.96 (1.04)	1.14
System. l = 2.0 m. 2 stories			
1.0	0.84	1.1 (1.17)	1.31
0.68	1.16	1.1 (1.17)	1.05
System. l = 2.5 m. 2 stories			
1.0	0.96	1.2 (1.27)	1.25
0.68	1.52	1.2 (1.27)	0.79
System. l = 2.5 m. 3 stories			
1.4	0.68	0.96 (1.0)	1.41
0.9	1.08	0.96 (1.0)	0.89

12 SUMMARY AND RECOMMENDATIONS

The need to improve the traditional force based design concept was clearly shown in the first part of the project, in which the behaviour of three modern clay brick masonry buildings in the Emilia Romagna region during the seismic events on May 20, 2012 and May 29, 2012 was analysed [15], [16], [17], [18], [19], [20], [21]. It was shown, that the combination of the building idealisation as a simple cantilever system and the linear verification wall by wall without load redistribution among the walls cannot be successfully applied in case of seismic actions. The linear verifications of all three buildings were far away to match the real behaviour of the three buildings. The application of nonlinear verifications led to more realistic results in comparison to the linear verification, but there was still a remarkable gap between the estimated PGA of the regarded seismic events and the calculated maximum attainable PGA values. Therefore, it can be concluded, that the nonlinear verifications using the simplified calculation model and the subsequent verification with damped spectra exhibit safety margins and can be regarded as conservative.

The conservative nonlinear verification concept is also applied to derive new behaviour factors for unreinforced masonry buildings in German earthquake regions. The new behaviour factors consist of three components accounting for deformation capacity and energy dissipation, force redistribution and overstrength due to all other sources. The application of the new behaviour factors improves the utilization of the load-bearing reserves of unreinforced masonry buildings within the traditional force based verification. The proposed concept corresponds to the approach of the new draft of Eurocode 8-1-2018 [3].

The new behaviour factors are derived based on a comprehensive parametric study of representative unreinforced masonry buildings in Germany by means of linear and nonlinear calculations according to the masonry codes DIN EN 1996-1-1 [1], DIN EN 1996-1-1/NA-[2] in combination with the seismic code DIN EN 1998-1 [6] and the National Annexes DIN 1998-1/NA-2011 [7] and DIN 1998-1/NA-2018 [11]. The behaviour factors are determined by statistical evaluation of all calculation results for the 28 building configurations. The behaviour factors for terraced houses are determined separately, as their ground plan configuration is quite different to single-family and multi-family houses. Furthermore, the special case of single-storey buildings is regulated separately. The behaviour factors are summarized in Table 12-1. The new behaviour factors are significantly higher than the standard value of 1.5, which has to be applied so far.

Table 12-1: Comparison of linear and nonlinear verifications for the 28 building configurations considering q_R, q_D, q_S

Component of behaviour factor	Single-family houses and multi-family houses	Terraced houses (≥ 2 stories)	Single-storey houses
Behaviour factor accounting for load redistribution effects	$q_R = 1.4$	$q_R = 1.0$*	$q_R = 1.25$**
Behaviour factor accounting for deformation capacity and energy dissipation	$q_D = 1.7$	$q_D = 1.9$	$q_D = 1.5$
Behaviour factor accounting for overstrength due to all other sources	$q_S = 1.0$	$q_S = 1.0$	$q_S = 1.0$
Resulting behaviour factor *q*	$\boldsymbol{q = 2.4}$	$\boldsymbol{q = 1.9 - 2.4}$	$\boldsymbol{q = 1.9}$

** The standard value is 1.0. Higher values may be applied, provided that they are confirmed through a global nonlinear static analysis of the building. The maximum value to be used in the design is equal to 1.25.*
*** The q_R value for terraced houses must be reduced to 1.0. Higher values may be applied, provided that they are confirmed through a global nonlinear static analysis of the building. The maximum value to be used in the design is equal to 1.25.*

The application of the proposed behaviour factors within the traditional linear verification concept was checked for the 28 buildings of the parametric study and two modern clay brick masonry buildings located in the Emilia Romagna region affected by the seismic event on May 29, 2012. The comparison of the linear verification results with the nonlinear verification based on the simplified calculation model and the subsequent verification with damped spectra clarifies that the verification results are still conservative. However, it has to be pointed out, that the force-based design concept with behaviour factors is only an estimation of the real behaviour of masonry buildings under seismic loading. As the basic idea of the behaviour factor is a general and simplified consideration of all nonlinear effects, the utilization of the load-bearing reserves is limited in comparison to non-linear calculations for specific masonry buildings. Anyway, the present proposal will significantly improve the force-based design of masonry buildings in Germany in comparison to the normative rules available so far.

Furthermore, it is important to clarify that the results of the parametric study are limited to typical unreinforced masonry buildings in Germany and cannot be transferred to other countries with differing building types. Furthermore, it is necessary to consider the limitations and assumptions for the application of the proposed behaviour factors:

- The behaviour factors are derived for the moment redistribution factors for partial restraint provided in Table 5-1. The application of the behaviour factors in combination with significantly different moment redistribution factors (e.g. weak slabs, full restraint) is not allowed. The application of higher moment redistribution factors in combination with increased behaviour factors may lead to unsafe design results, as the linear approach does not take into account the reduced deformation capacity in case of higher restraint levels.
- The slabs must be designed for the frame actions due to the considered moment redistribution factors given in Section 5.2. The development of constructive rules and design approaches for considering the wall-slab interaction was not the subject of this project. However, further investigations and additional rules are required to introduce and apply the new behaviour factors in practise.
- All requirements introduced in Section 8 shall be considered.

It has to be pointed out that the governed input parameters for the design are the maximum inter-storey drifts and the consideration of frame actions. The drift limits are based on the recent version of the German National Annex [11], which were defined a couple of years ago based on experimental results of the European project INSYSME. The drift limits need to be checked again, as further experimental test data are available and new recommendations for the drift limits can be expected within the development of the new Eurocode generation, in which a European data base for shear wall tests on masonry walls will be prepared. Especially, in-plane shear wall tests with reduced support lengths and out-of-plane eccentricities at the top shall be taken into account and analysed.

Finally, the first project part showed that the original N2 method is not applicable to short period masonry structures as already recognized and published by several researchers. For this reason the more conservative approach with damped spectra was applied. However, it seems necessary to check the applicability of new approaches with inelastic spectra for unreinforced masonry buildings in Germany. This requires comprehensive investigations similar to already published investigations and proposals [27].

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DIBt-Project

Report WP4-RWTH
Final Report

Improved seismic design concepts for masonry buildings in Germany

(Verbesserte seismische Nachweiskonzepte für Mauerwerksbauten in Deutschland)

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1 ZUSAMMENFASSUNG UND ERGEBNISSE: PROJEKTTEIL 1 – DGFM

1.1 Hintergrund und Zielsetzung

Am 20.05.2012 ereignete sich in der Emilia Romagna in Italien ein Erdbeben mit einer Magnitude von 5.9, das sieben Menschenleben kostete, immense Schäden an historischen Bauwerken verursachte, zahlreiche Industriebauwerke stark beschädigte und etwa 7000 Menschen obdachlos werden ließ. Diesem Erdbebenereignis folgte am 29.05.2012 ein weiteres schweres Erdbeben mit einer Magnitude von 5.9. Das zweite Erdbeben verursachte zahlreiche neue Schäden, führte aber auch an den bereits durch das erste Beben vorgeschädigten Gebäuden zu erheblichen Schadenszunahmen bis hin zu Gebäudeeinstürzen. Zudem kam es nach dem Hauptereignis zu weiteren Nachbeben, die lokal hohe Bodenbeschleunigungen verursachten.

Obwohl die gemessenen maximalen Bodenbeschleunigungen im Bereich oder über den seismischen Bemessungswerten der Norm lagen, traten an zahlreichen modernen unbewehrten Mauerwerksbauten häufig keine oder nur geringe Schäden auf. Diese Beobachtungen waren der Anlass, im Rahmen des ersten Projektteils gezielt drei unbewehrte moderne Mauerwerksgebäude aus Ziegelmauerwerk für detaillierte Untersuchungen auszuwählen. Ein wesentliches Kriterium für die Auswahl war neben dem Gebäudetyp eine möglichst genaue Kenntnis der seismischen Eingangsbelastung durch nahe gelegene Messstationen. Dies führte zu der Auswahl der in Abbildung 1-1 dargestellten drei modernen unbewehrten Mauerwerksgebäude in der unmittelbaren Nähe von Messstationen.

Abbildung 1-1: Ausgewählte Gebäude und Epizentren der Beben

Die Zielsetzung des ersten Projektteils war der Nachweis der ausgewählten Gebäude auf Grundlage von linearen und nichtlinearen Berechnungsmodellen unterschiedlicher Komplexität unter der Belastung der in Standortnähe während der Beben gemessenen seismischen Eingangsbelastungen. Die Nachweisergebnisse wurden mit den aufgetretenen Gebäudeschäden verglichen, um eine Aussage über die Konservativitäten der Berechnungs- und Bemessungsansätze abzuleiten. Ergänzend dazu wurden probabilistische Analysen durchgeführt, mit denen das Sicherheitsniveau der Gebäude am Standort genauer beurteilt wurde. Die Ergebnisse sind in den folgenden Berichten zusammengefasst: [13], [14], [15], [16], [17], [18], [19].

1.2 Projektergebnisse und Handlungsbedarf

Nachfolgend sind die Ergebnisse der Gebäudeuntersuchungen getrennt für lineare und nichtlineare Nachweisverfahren zusammengestellt. Diese Unterscheidung wird getroffen, da der lineare Nachweis in der Praxis das Standardverfahren darstellt und auf Grund der einfachen Anwendung und Akzeptanz auch zukünftig bleiben wird. Nichtlineare Nachweisverfahren werden langfristig mehr und mehr Eingang in die Praxis finden, müssen aber für eine breitere und sichere praktische Anwendung hinsichtlich der verwendeten Grundlagen transparent und anwendungsfreundlich aufbereitet werden. Auf Grundlage der Ergebnisse wird der Handlungsbedarf für verbesserte normative seismische Nachweise von Mauerwerksbauten abgeleitet. Abschließend erfolgt eine Bewertung der untersuchten Gebäude auf Grundlage der Ergebnisse der probabilistischen Analysen.

1.2.1 Lineare Nachweisverfahren

Die Anwendung der linearen Nachweisverfahren für die drei Gebäude erfolgte auf Grundlage eines vereinfachten Kragarmmodells und eines äquivalenten Rahmenmodells. Die Nachweise erfolgten für normative Belastungen und für die seismischen Eingangsbelastungen der Haupterdbebenereignisse am 20.05. und 29.05.2012. Die Nachweisführung erfolgte durch die Universität Pavia mit dem Programm ANDILWall [24] und durch die SDA-engineering GmbH mit dem Programmsystem MINEA-R [23].

Die mit den linearen Verfahren ermittelten *<u>maximal aufnehmbaren Bodenbeschleunigungen lagen für die drei betrachteten Gebäude im Mittel unter 5% der am Standort ermittelten maximalen Bodenbeschleunigungen</u>*. Für keines der Gebäude war ein erfolgreicher Nachweis möglich, obwohl alle Gebäude den Erdbebenereignissen standhielten und an zwei Gebäuden nicht einmal Risse festzustellen waren. Das Nachweisergebnis für die am Gebäudestandort ermittelte seismische Belastung steht somit im Widerspruch zu dem beobachteten seismischen Gebäudeverhalten während der Erdbeben im Jahre 2012.

<u>Fazit</u>

Lineare Nachweise liefern extrem konservative und damit auch unwirtschaftliche Nachweisergebnisse und können die realen Tragfähigkeiten nicht abbilden. Der Abstand zwischen den Nachweisergebnissen und dem tatsächlichen Gebäudeverhalten während der betrachteten Erdbebenereignisse ist so groß, dass ein linearer Nachweis auf Grundlage der aktuellen normativen Regelungen nicht empfehlenswert ist. Die Defizite in der linearen Nachweisführung liegen in gleicher Weise auch für andere Mauerwerksmaterialien (Kalksandstein, Leichtbeton, Porenbeton) vor.

<u>Handlungsbedarf</u>

Lineare seismische Nachweisansätze müssen auf Grund ihrer einfachen und nachvollziehbaren Anwendbarkeit auch zukünftig in den Normen verankert sein. Damit diese realistischere Ergebnisse liefern, ist es erforderlich, diese Ansätze weiterzuentwickeln. Ansätze zur Verbesserung sind eine Berücksichtigung der Lastumverteilung im Grundriss und der Momentenumverteilung durch die Rahmentragwirkung der Deckenmitwirkung.

1.2.2 Nichtlineare Nachweisverfahren

Die Anwendung statisch nichtlinearer Nachweisverfahren auf die drei ausgewählten Gebäude lieferte wesentlich realistischere Nachweisergebnisse. Die Untersuchungen ergaben im Mittel für die drei betrachteten Gebäude *<u>maximal aufnehmbare Bodenbeschleunigungen von 56% (Vereinfachtes Balkenmodell) bzw. 83% (Äquivalentes Rahmenmodell) bezogen auf die am Standort für die Beben ermittelten maximalen Bodenbeschleunigungen</u>*.

Fazit

Statisch nichtlineare Nachweisverfahren liefern realistischere Ergebnisse und nutzen die Tragwerksreserven besser aus. Mit zunehmendem Genauigkeitsgrad der Berechnungsmodelle spiegeln die Nachweisverfahren das tatsächliche seismische Gebäudeverhalten wesentlich besser wieder. Diese Aussage kann auch auf andere Mauerwerksmaterialien (Kalksandstein, Leichtbeton, Porenbeton) übertragen werden.

Handlungsbedarf

Die Anwendung der Verfahren auf Grundlage des DIN EN 1998-1 [6] und des Nationalen Anwendungsdokuments DIN EN 1998-1/NA-2011 [7] und DIN EN 1998-1-2018 [9] auf die drei Gebäude verdeutlichten die zurzeit noch unzureichenden normativen Regelungen dieser Verfahren.

Den größten Einfluss auf die Ergebnisse hat die Definition der maximalen gegenseitigen Stockwerksverschiebungen, die im Basisdokument DIN EN 1998-1 [6] nicht angegeben sind. Die Werte im DIN EN 1998-3-2005 [5] können nicht auf moderne Mauerwerksmaterialien und die heute verwendeten Dünnbettmörtel übertragen werden, und der neue Entwurf der DIN EN 1998-3-2018 [4].

Das nationale Anwendungsdokument DIN EN 1998-1-2018 [9] beinhaltet maximale Stockwerksverschiebungen, diese basieren aber im Wesentlichen auf den Versuchen aus dem europäischen Projekt INSYSME. Inzwischen stehen aber weitere Wandversuche zur Verfügung, die in die der Norm zugrundeliegenden Auswertungen noch nicht eingeflossen sind. Zudem ist in den Wandversuchen die exzentrische Lasteinleitung am Wandkopf bislang unberücksichtigt geblieben. Es ist unbedingt erforderlich sich aktiv an der sich im Aufbau befindlichen europäischen Datenbank zu beteiligen, damit die für die Bemessung erforderlichen spezifischen Eigenschaften der Mauerwerksprodukte des deutschen Markts ausreichende Berücksichtigung finden. Erste Auszüge aus der Datenbank lassen vermuten, dass die Verformungsfähigkeiten für Schub in dem Nationalen Anwendungsdokument DIN EN 1998-1-2018 [9] zu optimistisch sind.

Weiterhin zeigt der Ergebnisvergleich zwischen den Projektpartnern, dass die Regelungen für die Schubnachweise in dem deutschen Anwendungsdokument DIN EN 19096-1-1 [2] strikter sind, so dass diese schneller maßgebend werden und zu deutlich konservativeren Nachweisergebnissen führen. Dieser Aspekt ist zusammen mit der Auswertung und Abgrenzung der Versagensmoden auf Grundlage von Wandversuchen vertiefter zu prüfen. Maßgebend in der Schubbemessung wird hierbei häufig das Steinzugversagen.

Schlussendlich ergeben sich große Unterschiede in den Nachweisen bei Anwendung der Nachweisformen mit gedämpften und inelastischen Spektren. Die Regelungen im aktuellen Basisdokument DIN EN 1998-1 [6] sind nicht auf Mauerwerksbauten anwendbar, weshalb diese in der italienischen Norm NTC2018 [8] bereits modifiziert wurden. Im deutschen Anwendungsdokument sind entsprechende Modifikationen noch nicht enthalten, stattdessen wird dem Anwender die Entscheidung überlassen. Hier besteht weiterer Regelungsbedarf, um ein ausreichendes Sicherheitsniveau zu erhalten.

1.2.3 Ergebnisse der probabilistischen Sicherheitsbetrachtung

Auf Grundlage der durchgeführten probabilistischen Analysen wurden Fragilitätskurven ermittelt, die mit den seismischen Gefährdungskurven überlagert wurden, um die jährlichen Versagenswahrscheinlichkeiten der drei Gebäude zu ermitteln. Als Ergebnis ergab sich, dass die normative Zielversagenswahrscheinlichkeit für die Bemessungssituation Erdbeben für die an den Gebäudestandorten vorliegenden seismischen Gefährdungen für keines der drei Gebäude erreicht wurde. Trotz der Nichteinhaltung der Zielversagenswahrscheinlichkeit zeigten die Gebäude in der Erdbebenserie aber einen ausreichenden seismischen Widerstand.

Fazit

Mit den probabilistischen Analysen der Gebäude wurde nachgewiesen, dass die normative geforderte Zielversagenswahrscheinlichkeit für keines der untersuchten Gebäude erreicht wird. Damit liegt im Sinne der Norm ein Sicherheitsdefizit vor, das aber in dem beobachteten seismischen Verhalten der Gebäude während der Erdbebenserie in 2012 nicht festzustellen war. Dies Ergebnis ergab sich sowohl für vereinfachte als auch für komplexere Rechenmodelle.

Handlungsbedarf

Verbesserung der linearen und nichtlinearen Nachweisverfahren entsprechend des in den Abschnitten und beschrieben Handlungsbedarfs.

2 ZUSAMMENFASSUNG UND ERGEBNISSE: PROJEKTTEIL 2 – DIBT

2.1 Arbeitspakete und Arbeitsaufteilung

Das Projekt mit einer Laufzeit von sieben Monaten wurde entsprechend der in dem Antrag ausgewiesenen Arbeitspakete vollständig bearbeitet. Nachfolgend sind die Arbeitspakete und die zugehörigen Arbeitsberichte angegeben.

AP 1: Grundlagen für die Anwendung statisch nichtlinearer Verfahren

Bericht: DIBT-REPORT-WP1-RWTH-R1 vom 13.02.2019

AP 2: Festlegung und Modellierung repräsentativer Mauerwerksgebäude

Bericht: DIBT-REPORT-WP2-RWTH-R1 vom 13.02.2019

AP 3: Kraftbasierte und verformungsbasierte Berechnungen

Bericht: DIBT-REPORT-WP3-RWTH-R1 vom 13.02.2019

AP 4: Modifikation des kraftbasierten Berechnungsansatzes

Bericht: DIBT-REPORT-WP4-RWTH-R1 vom 13.02.2019

AP 5: Zusammenfassungen in verschiedenen Sprachen

DIBT-SUMMARY-DEUTSCH-WP5-R0, DIBT-SUMMARY-FRANZÖSISCH-WP5-R0 und DIBT-SUMMARY-ENGLISCH-WP5-R0

AP 6: Projekttreffen in Aachen und Kassel

Das Projekttreffen in Kassel wurde per Videokonferenz durchgeführt. Das Treffen in Aachen wurde and das DIBt nach Berlin verlegt.

In den aufgeführten Berichten der einzelnen Arbeitspakete sind die in dem Antrag beschriebenen Arbeiten und Zielstellungen vollständig beschrieben. Insbesondere wurden die im Kapitel 1 des Antrags formulierten Zielstellungen in dem Projekt erreicht. Dies sind im Einzelnen:

- Ausarbeitung von fehlenden Grundlagen für eine Anwendung von verformungsbasierten Verfahren in der Baupraxis
- Verbesserung des kraftbasierten Bemessungsansatzes durch Berücksichtigung der Lastumverteilung und der Dissipation
- Vergleich des neuen basierten Bemessungsansatzes mit den Ergebnissen nichtlinearer Berechnungen

2.2 Ergebnistransfer

Die Ergebnisse des ersten Projektteils wurden auf der 16th European Conference on Earthquake Engineering und in der Zeitschrift Mauerwerk publiziert. Beide Veröffentlichungen sind Berichten beigelegt. Aktuell wird eine weitere Publikation für die D-A-CH Tagung in Innsbruck vorbereitet. Zudem ist eine Publikation im Bauingenieur geplant. Weiterhin wurden bereits im Rahmen des

Projekts Vorschläge aufbereitet, um die Ergebnisse in die Normung einzubringen. Konkret wurden Vorschläge für die Verbesserung der Normtexte zu den verformungsbasierten Verfahren und zu den Verhaltensbeiwerten unter Berücksichtigung einer Lastumverteilung ausgearbeitet und in den Normausschuss eingereicht.

2.3 Arbeitsaufteilung

In dem ursprünglichen Projektplan war eine Verteilung der Arbeiten auf die RWTH Aachen, die Universität Kassel und die Bauhaus Universität Weimar (EDAC) vorgesehen. Im Laufe der Bearbeitung wurden in Abstimmung mit den Projektpartnern sämtliche Arbeiten von der RWTH Aachen übernommen. Die Universität Kassel und die Bauhaus Universität Weimar (EDAC) waren aber über die gesamte Projektzeit über die Projektaktivitäten und Projektinhalte informiert, tragen die Projektergebnisse mit und unterstützen die Überführung der Ergebnisse in die Normen. Zudem flossen die Erfahrungen der Universität Kassel aus zahlreichen Versuchen an Mauerwerk sowie die Expertise aus Weimar in dem Bereich der Schadensauswertungen an Mauerwerksbauten zurückliegender Erdbeben ein.

2.4 Verbesserung der linearen Nachweisverfahren

2.4.1 Aufbereitung der Ergebnisse für die Verwendung in DIN EN 1998-1-2018

Zur Verbesserung der linearen Nachweisverfahren wird für unbewehrte Mauerwerksbauten in deutschen Erdbebengebieten ein neuer Verhaltensbeiwert q in Anlehnung an den neuen Entwurf der DIN EN 1998-1-2018 [3] eingeführt:

$$q = q_R \cdot q_D \cdot q_S$$

q_R Verhaltensbeiwert durch Überfestigkeiten infolge Lastumverteilung

q_D Verhaltensbeiwert infolge Verformungsvermögen und Energiedissipation

q_S Verhaltensbeiwert zur Erfassung aller weiteren Überfestigkeiten

Die drei Anteile zur Berechnung des Verhaltensbeiwerts q sind für Reihenhäuser in Tabelle 2-1 und für Ein- und Mehrfamilienhäuser in Tabelle 2-2 zusammengestellt. Die Anwendung der Tabellenwerte setzt eine Regelmäßigkeit im Auf- und Grundriss voraus.

Tabelle 2-1: Verhaltensbeiwerte q_R, q_D, q_S für zweigeschossige Reihenhäuser

Verhaltensbeiwert durch Überfestigkeiten infolge Lastumverteilung	$q_R = 1.0$*
Verhaltensbeiwert infolge Verformungsvermögen und Energiedissipation	$q_D = 1.9$
Verhaltensbeiwert zur Erfassung aller weiteren Überfestigkeiten	$q_S = 1.0$
Resultierender Verhaltensbeiwert *q*	$\boldsymbol{q = 1.9 - 2.4}$

** Der Standardwert ist 1.0. Größere Werte für q_R können angesetzt werden, wenn diese durch eine nichtlineare statische Berechnung am Gesamtsystem nachgewiesen werden. Der maximal ansetzbare Wert für q_R ist 1.25.*

Tabelle 2-2: Verhaltensbeiwerte q_R, q_D, q_S für Ein- und Mehrfamilienhäuser mit mindestens zwei Geschossen

Verhaltensbeiwert durch Überfestigkeiten infolge Lastumverteilung	$q_R = 1.4$
Verhaltensbeiwert infolge Verformungsvermögen und Energiedissipation	$q_D = 1.7$
Verhaltensbeiwert zur Erfassung aller weiteren Überfestigkeiten	$q_S = 1.0$
Resultierender Verhaltensbeiwert *q*	$\boldsymbol{q = 2.4}$

Für eingeschossige Gebäude sind die Werte in Tabelle 2-3 anzusetzen. Die Werte berücksichtigen, dass die Energiedissipation und Lastumverteilung für eingeschossige Gebäude beschränkt ist.

Tabelle 2-3: Verhaltensbeiwerte q_R, q_D, q_S für Reihenhäuser und Ein- und Mehrfamilienhäuser mit einem Geschoss

Verhaltensbeiwert durch Überfestigkeiten infolge Lastumverteilung	$q_R = 1.25$*
Verhaltensbeiwert infolge Verformungsvermögen und Energiedissipation	$q_D = 1.5$
Verhaltensbeiwert zur Erfassung aller weiteren Überfestigkeiten	$q_S = 1.0$
Resultierender Verhaltensbeiwert *q*	$\boldsymbol{q = 1.9}$

** Für einstöckige Reihenhäuser ist der Wert q_R auf 1.0 zu reduzieren. Größere Werte für q_R können nur angesetzt werden, wenn diese durch eine nichtlineare statische Berechnung am Gesamtsystem nachgewiesen werden. Der maximal ansetzbare Wert für q_R ist 1.25.*

Die Verhaltensbeiwerte sind im Falle von Unregelmäßigkeiten wie folgt zu reduzieren:

- Für im Aufriss unregelmäßige Gebäude ist der Verhaltensbeiwert q_D um 20% zu reduzieren. Der Wert für q_D muss aber nicht kleiner als 1.5 angesetzt werden.

- Für im Grundriss unregelmäßige Gebäude kann der Verhaltensbeiwert q_R als Mittelwert von 1.0 und dem Tabellenwert anzusetzen.

Zusätzlich sind folgende Randbedingungen eingehalten:

- Die Decken sind für die Zusatzmomente infolge der angesetzten Rahmentragwirkung zu bemessen. Die Zusatzmomente sind entsprechend der in Tabelle 2-4 angegebenen Momentenverteilungsfaktoren zu berechnen, die der Berechnung der Tabellenwerte zugrunde liegen.

Tabelle 2-4: Momentenverteilungszahlen in Abhängigkeit von Geschossanzahl und Wandlänge

	Kragarm	**Teileinspannung**			**Volleinspannung**
Geschoss	**alle L_W**	**$L_W \leq 1.5$m**	**1.5m < L_W < 2.5m**	**$L_W \geq 2.5$m**	**alle L_W**
1	1	0.5	0.6	1.0	0.5
2	1.7	0.85	1.0	1.2	0.5
3	2.3	1.2	1.4	1.7	0.5
4	3.0	1.5	1.8	2.1	0.5

Zusätzlich sind folgende konstruktive Regeln einzuhalten:

- Die Torsionssteifigkeit von Reihenhäusern ist durch die Anordnung von zwei parallelen Außenwänden in Längsrichtung sicherzustellen. Die Wandlänge dieser Wände muss mindestens 30% der Gebäudeabmessung in Längsrichtung betragen.

- In Querrichtung sind in Reihenhäusern mindestens zwei parallele Innenwände mit einer Mindestwandlänge von 1.50 m anzuordnen. Zusätzlich sind je zwei parallele Außenwände auf jeder Gebäudeseite mit einer Mindestwandlänge von 1.0 m anzuordnen.

Höhere Verhaltensbeiwerte q_R and q_D können dann angesetzt werden, wenn diese durch eine globale nichtlineare statische Berechnung am Gesamtsystem nachgewiesen werden. Die Ableitung der Verhaltensbeiwerte muss entsprechend der Definition der Verhaltensbeiwerte q_R, q_D und q_S in dem Bericht WP3-RWTH [12] erfolgen. Der maximale resultierende Verhaltensbeiwert q ist auf 3.0 begrenzt. Da die Ausarbeitung der neuen DIN EN 1998-1-2018 [3] aktuell noch in der Bearbeitung ist, können die Vorschläge in die Ausarbeitung der Norm auf nationaler Ebene eingebracht werden

2.4.2 Aufbereitung der Ergebnisse für die Verwendung in DIN EN 1998-1-2010

Nachfolgend sind die Projektergebnisse für die Berücksichtigung in DIN EN 1998-1-2010 [6] aufbereitet. Dementsprechend wird der Verhaltensbeiwert q_D mit q_0 und der Verhaltensbeiwert q_R mit dem Beiwert α_u/α_1 angegeben. Der Vorschlag für die Norm lautet:

NDP zu 9.3(4) Bauwerkstypen und Verhaltensbeiwerte

Tabelle NA. 8 — Bauwerkstypen und Höchstbeträge der Verhaltensbeiwerte

Bauwerkstyp	**Verhaltensbeiwert** q_0
Unbewehrtes Mauerwerk nach DIN EN 1996-1	1,5
Unbewehrtes Mauerwerk nach DIN EN 1998-1	Tabelle NA.9
Eingefasstes Mauerwerk	Tabelle NA.9
Bewehrtes Mauerwerk	3,0

Der Verhaltensbeiwert q ist in jede Gebäuderichtung mit Tabelle NA.9 wie folgt zu ermitteln:

$$q = q_0 \cdot \alpha_u/\alpha_1$$

mit:

q_0 Grundwert des Verhaltensbeiwerts abhängig vom Tragwerkstyp und von seiner Regelmäßigkeit im Aufriss (siehe Abschnitt **9.3(5)**)

α_1 Multiplikator der horizontalen Erdbebenbemessungseinwirkung beim erstmaligen Erreichen der Biege- oder Schubfestigkeit einer Schubwand, während alle anderen Bemessungseinwirkungen konstant gehalten werden.

α_u Multiplikator der horizontalen Erdbebenbemessungseinwirkung bei Erreichen der maximalen Schubkapazität des Gebäudes, wobei alle anderen Bemessungseinwirkungen konstant gehalten werden.

Wird der Beiwert α_u/α_1 nicht durch explizite Berechnung bestimmt, dürfen für im Grundriss regelmäßige Hochbauten die in Tabelle NA.B angegebenen Näherungswerte von α_u/α_1 zur Lastumverteilung verwendet werden, wenn die Geschossdecken für die zusätzlichen Momente der in Tabelle NA.A angegebenen Momentenverteilungsfaktoren Ψ nach DIN EN 1996-1-1/NA, Anhang K [2] für jedes Vollgeschoss ausgelegt werden. Diese Verteilungszahlen sind auch der Schubwandbemessung zugrunde zu legen.

Tabelle NA. A — Momentenverteilungsfaktoren nach DIN EN 1998-1-1/NA, Anhang K [2]

Anzahl der Geschosse über betrachtetem Wandfuß	Momentenverteilungsfaktor Ψ , DIN EN 1996-1-1, Anhang K		
	$L_W \leq 1.5m$	$1.5m < L_W < 2.5m$	$L_W \geq 2.5m$
1	0,5	0,6	1,0
2	0,85	1,0	1,2
3	1,2	1,4	1,7
4	1,5	1,8	2,1
5	1,8	2,2	2,5
6	2,1	2,6	2,9

Tabelle NA. B — Beiwerte α_u/α_1 zur Berücksichtigung der Lastumverteilung

Anforderungen	α_u/α_1
Einstöckige Gebäude mit mindestens sechs Wänden in jeder Hauptrichtung und einer ausreichenden Wandlängenverteilung[a].	1,25
Mehrstöckige Gebäude mit mindestens sechs Wänden in jeder Hauptrichtung und einer ausreichenden Wandlängenverteilung[a].	1,4
Ein- und mehrstöckige Gebäude mit weniger als sechs Wänden oder einer nicht ausreichenden Wandlängenverteilung [a].	1,1

[a] *Eine ausreichende Wandlängenverteilung liegt vor, wenn das Verhältnis der Wandlängen der 75% und 25% Fraktilwerte der Wandlängen größer als 1,5 ist.*

Bei Gebäuden ohne Lastumlagerungsmöglichkeiten ist α_u/α_1 = 1,0 anzusetzen.

Für im Grundriss nicht regelmäßige Hochbauten (Abschnitt **4.2.3.2**) ist der Näherungswert von α_u/α_1, der verwendet werden darf, wenn zu seiner Bestimmung keine Pushover-Berechnungen durchgeführt werden, gleich dem Mittelwert von 1,0 und dem gewählten Wert von α_u/α_1.

Werte von α_u/α_1, die größer sind als die in diesem Unterabschnitt angegebenen, sind zulässig, sofern sie durch eine Pushover-Berechnung nach Abschnitt **4.3.3.4.2** auf Systemebene bestätigt werden.

Der maximal ansetzbare Verhaltensbeiwert ist q = 2,5.

Tabelle NA.9 — Verhaltensbeiwert für unbewehrtes und eingefasstes Mauerwerk nach DIN EN 1998-1

Mauerwerksart	Wandgeometrie	
	$h/l^{a} \leq 1$	$h/l \geq 1{,}6$
Unbewehrt, einstöckig [b, c]	$1{,}5 \cdot \alpha_u/\alpha_1$	
Unbewehrt, mehrstöckig [b, c]	$1{,}7 \cdot \alpha_u/\alpha_1$	$2{,}0 \cdot \alpha_u/\alpha_1$
Eingefasst	2,0	2,5

[a] h/l bezeichnet das Verhältnis der lichten Geschosshöhe zur Länge der längsten Wand in der betrachteten Gebäuderichtung.

[b] Die Verwendung von Grundwerten für Verhaltensbeiwerte $q > 1{,}5$ ist nur zulässig, wenn bei der Bemessungssituation infolge Erdbeben die mittlere Normalspannung in den entsprechenden Wänden 15 % der charakteristischen Mauerwerkdruckfestigkeit f_k nach DIN EN 1996-1-1 nicht überschreitet.

[c] Die Tragwerksmodellierung darf nach DIN EN 1996-1-1 erfolgen.

Zwischenwerte dürfen linear interpoliert werden.

2.5 Verbesserung der nichtlinearen Nachweisverfahren

Die nichtlinearen Berechnungen wurden in diesem Projekt mit den Verformungsfähigkeiten des Nationalen Anwendungsdokuments DIN EN 1998-1/NA-2018 [9] durchgeführt. Im Rahmen des europäischen Normungsprozesses sind die Verformungsfähigkeiten zu prüfen und um den Aspekt exzentrischer Lasteinleitungen am Wandkopf zu erweitern. Da der Einfluss der Verformungstragfähigkeiten auf die Nachweisergebnisse signifikant ist, sind die erforderlichen weiteren Untersuchungen von wesentlicher Bedeutung.

Die Nachweise wurden konservativ mit gedämpften Spektren durchgeführt. Die N2-Methode wurde nicht angewendet, da diese für kurzperiodische Mauerwerksbauten unsichere Ergebnisse liefert. Zukünftig sollten inelastische Spektren für die typische Konstruktionsweise von unbewehrten Mauerwerksbauten in Deutschland hergeleitet werden. Eine Herleitung inelastischer Spektren konnte auf Grund des Arbeitsumfangs im Rahmen dieses Projekts nicht durchgeführt werden. Die Ableitung der Spektren muss im Rahmen der Erarbeitung der neuen Eurocodes erfolgen. Erste Vergleichsberechnungen auf Grundlage eines Vorschlags von Guerrini et al. [20] zeigten, dass mit den inelastischen Spektren optimistischere Ergebnisse zu erzielen sind. Aktuell sind dafür aber keine ausreichenden Grundlagen vorhanden.

Weiterhin müssen die nichtlinearen Berechnungsmodelle verbessert werden. Weder das im Rahmen des Projekts verwendete nichtlineare Balkenmodell noch die Verwendung äquivalenter Rahmensysteme können die für deutsche Mauerwerksbauten komplexen Wand-Deckeninteraktionen ausreichend genau abbilden. Die Entwicklung geeigneter Makroelemente ist Gegenstand der Forschung und war somit nicht Bestandteil des Projekts.

2.6 Änderungsvorschläge des aktuellen normativen Textes in DIN EN 1998-1/NA-2018

Nachfolgend sind die aus diesem Projekt abgeleiteten Änderungsvorschläge für die DIN EN 1998-1/NA-2018 [9] zusammengestellt. Die erforderlichen Änderungen sind farbig markiert. Die Begründungen für die Änderungen sind in den dazugehörigen Fußnoten angegeben.

NCI zu 4.3.3.4.2.1(1) Nichtlineare statische (pushover) Berechnung

*Die Pushover-Berechnung dient zu Ermittlung der inelastischen Last-Verformungskurve (Kapazitätskurve) eines Bauwerks unter konstanten Gewichtslasten und monoton wachsenden Horizontalkräften (**4.3.3.4.2.2**). Die Kapazitätskurve (**4.3.3.4.2.3**) gibt die Beziehung zwischen der Relativverschiebung eines maßgebenden Bezugspunktes (Kontrollknoten nach Anhang B.1) und der Oberkante des Fundaments oder eines starren Kellergeschosses sowie der einwirkenden Gesamterdbebenkraft an. Für die Ermittlung der Kapazitätskurve und den Ansatz der Festigkeiten sind **4.3.3.4.1(1)** bis **(4)** zu beachten. Der Schnittpunkt der Kapazitätskurve mit dem elastischen Antwortspektrum (**3.2.2.2**) im (Spektralbeschleunigungs-Spektralverschiebungsdiagramm (S_a-S_d-Diagramm) liefert die Zielverschiebung (**4.3.3.4.2.6**), die Grundlage des Nachweises ist. Der Nachweis ist erfüllt, wenn die 1,5-fache Zielverschiebung kleiner oder gleich der auf den Bezugspunkt (Kontrollknoten) bezogenen Verformungskapazität ist. Die Verformungskapazität ist diejenige Verformung, bei der gemäß der Kapazitätskurve nach Erreichen der maximalen Tragfähigkeit noch mindestens 80 % des maximalen Tragwerkswiderstands vorhanden ist.*

Hinweis: Falls die kinematische Boden-Bauwerk-Wechselwirkung (DIN EN 1998-5) von Bedeutung ist, sind deren Auswirkungen zu beachten. ***(*1)***

NCI zu 4.3.3.4.2.1(2) und (3) Nichtlineare statische (pushover) Berechnung

Folgende Bedingungen sollten bei der Berechnung und der Wahl der Rechenmodelle berücksichtigt werden:

- *Das gewählte Rechenmodell muss in der Lage sein das Schwingungsverhalten und die nichtlinearen Lastumverteilungseffekte im Bauwerk zu erfassen.*
- *Die Decken verfügen über eine ausreichende Steifigkeit in ihrer Ebene und sind in der Lage die Horizontallasten über ausreichend dimensionierte Verbindungen auf die Aussteifungselemente zu verteilen.*
- *Wenn das dynamische Verhalten im Wesentlichen durch die Grundeigenform bestimmt wird, ist es ausreichend nur die modale Verteilung der Horizontalkräfte zu berücksichtigen. Höhere Schwingformen können vernachlässigt werden, wenn die Bedingungen in **4.3.3.2.1(2)** erfüllt sind.*
- *Wird das dynamische Verhalten auch durch höhere Eigenformen beeinflusst, so sind die modalen Horizontalkraftverteilungen der wesentlichen Eigenformen mit wechselnden Vorzeichen zu kombinieren. Für jede Kombination der Eigenformen ist eine eigene Pushover-Analyse durchzuführen.*
- *Dem Nachweis der Zielverschiebung ist die ungünstigste Erdbebenrichtung zugrunde zu legen.*
- *Die Horizontalkomponenten der Erdbebeneinwirkung sind als gleichzeitig anzunehmen. Die Kombination der Richtungen kann nach **4.3.3.5.1(6)** erfolgen.*

NCI zu 4.3.3.4.2.3(2) Kapazitätskurve

*Bei der Ermittlung der Kapazitätskurve sind insbesondere **4.3.3.4.1(2)** bis **(6)** von Bedeutung. Die Aufstellung der Kapazitätskurve setzt voraus, dass alle am Lastabtrag beteiligten primären seismischen Bauteile in der Lage sind, die auftretenden Beanspruchungen entlang des gesamten Belastungspfads aufnehmen zu können (z. B. zusätzliche Deckenmomente infolge Rahmentragwirkung).*

*Für unbewehrte Mauerwerksbauten können zur Ermittlung der Kapazitätskurve des Bauwerks für die Einzelwände die Verformungskapazitäten nach **NCI zu 9.4(6)** unter Ansatz der Biege- und Schubkapazitäten nach DIN EN 1996-1-1* ***(*2)*** *angesetzt werden. Zur realitätsnahen Modellierung sollten die Wandsteifigkeiten für Schub und Biegung für gerissene Querschnitte angesetzt werden.*

Sind keine genaueren Werte verfügbar, können diese aus den um 50 % abgeminderten Werten der Wandsteifigkeiten im ungerissenen Zustand erhalten werden.

NCI zu 4.3.3.4.2.3(3) Kapazitätskurve

*Der Bezugspunkt darf am Massenmittelpunkt des Gebäudedaches angenommen werden, sofern es sich beim obersten Geschoss um ein Vollgeschoss handelt. Ansonsten ist der Bezugspunkt am Massenmittelpunkt der Decke des obersten Vollgeschosses geeignet. Es gilt die Definition des Vollgeschosses nach **NA.D.8(2)**.*

NCI zu 4.3.3.4.2.6 Zielverschiebung

*Das in Anhang B angegebene N2-Verfahren zur Ermittlung der Zielverschiebung ist in der Anwendung auf Stahlbetonrahmentragwerke begrenzt. Alternativ können allgemein die Kapazitätsspektrum-Methode nach ATC 40 oder die Verschiebungs-Koeffizienten-Methode nach FEMA 356 angewendet werden. Alternativ kann der Nachweis mit gedämpften oder inelastischen Spektren erfolgen. (*3)*

Das in Anhang B angegebene N2-Verfahren zur Ermittlung der Zielverschiebung basiert auf inelastischen Spektren für Stahlbetonrahmentragwerke und kann nicht auf andere Tragwerkstypen übertragen werden. Für andere Tragwerkstypen ist es erforderlich abgesicherte inelastische Spektren auf Grundlage von nichtlinearen dynamischen Zeitverlaufsberechnungen zu ermitteln. Alternativ kann der Nachweis mit gedämpften Spektren auf Grundlage der Kapazitätsspektrum-Methode erfolgen. In diesem Fall ist der erforderliche Dämpfungsansatz durch rechnerische oder experimentelle Untersuchungen abzuleiten.

NCI zu 4.3.3.4.2.7 Verfahren zur Abschätzung der Torsionswirkungen

*Zufällige Torsionswirkungen sind nach **4.3.2** beim Ansatz der Horizontalkräfte in den Massenpunkten des Rechenmodells durch Ansatz von Exzentrizitäten oder Zusatzmomenten zu berücksichtigen.*

NCI zu 9.4(6) Tragwerksberechnung

Alternativ zu den Regeln in 9.4(6) kann eine Umverteilung der durch lineare Berechnung nach Abschnitt 4 ermittelten Gesamterdbebenlast auf die einzelnen Wände auch vorgenommen werden, wenn:

*d) die Lastumlagerung unter Verwendung nichtlinearer oder vereinfachter elastisch-ideal plastischer Lastverformungskurven (*4) der Einzelwände unter Einhaltung des globalen Gleichgewichts erfolgt. Hierbei darf hinsichtlich der Tragwerksmodellierung nach DIN EN 1996-1-1 verfahren werden.*

Dabei dürfen die maximalen Verformungen zwischen Wandfuß und Wandkopf für alle Mörtel und Steinarten von in Deutschland üblichen Bauweisen für unbewehrtes Mauerwerk wie folgt angesetzt werden:

- *Biegeversagen: $0{,}006 \cdot H_0/L \cdot H$.*
- *Schubversagen: $0{,}004 \cdot H$, wenn bei der Bemessungssituation infolge Erdbeben die mittlere Normalspannung 15 % der charakteristischen Mauerwerkdruckfestigkeit f_k nach DIN EN 1996-1-1 oder allgemeiner bauaufsichtlicher Zulassung oder Bauartgenehmigung nicht überschreitet. In allen anderen Fällen ist die Verformung für Schubversagen mit höchstens $0{,}003 \cdot H$ anzusetzen.*

Hierbei sind H die Stockwerkshöhe, L die Wandlänge und H_0 der Abstand zwischen dem Querschnitt, in dem die Biegekapazität erreicht wird, und dem Wendepunkt, jeweils in m.

Für eingefasstes Mauerwerk dürfen die für unbewehrtes Mauerwerk angegebenen Verformungswerte um den Faktor 2 vergrößert werden.

Fußnoten zu den Änderungsvorschlägen:

*(*1) Im ersten Projektteil wurde nachgewiesen, dass die kinematische Interaktion für übliche mehrgeschossige Wohngebäude keinen wesentlichen Einfluss hat. Der Einfluss ist im Vergleich zu den wesentlichen Parametern (Einspanngrad, Verformungsfähigkeiten, Form des Spektrums) vernachlässigbar. Zudem ist im Teil 5 kein praktikables Berechnungskonzept angegeben, so dass im ersten Projektteil die FEMA-Richtlinien angewendet wurden. Der Hinweis ist zu streichen, da eine praktische Anwendung unrealistisch ist.*

*(*2) Korrektur Normverweis*

*(*3) Der Hinweis die Methoden nach ATC 40 und FEMA 356 anzuwenden ist unzureichend und stellt für unbewehrte Mauerwerksbauten in Deutschland ein Sicherheitsdefizit dar. In den zitierten Normen werden Verformungskapazitäten und Dissipationswerte angesetzt, die auf den amerikanischen Mauerwerksmaterialien basieren. Die Normenwerke sind sehr alt und gehen von Fugen aus Normalmörtel und vermörtelten Stoßfugen aus. Eine Übertragung auf die deutschen Bauweisen ist deshalb nicht möglich. Aus diesem Grunde ist die vorgeschlagene Präzisierung vorzunehmen.*

*(*4) Die Verwendung von genaueren Kurven als die Näherung elastisch-ideal plastischer Lastverformungskurven muss auch erlaubt sein.*

3 ERGEBNISVERGLEICH ALLER NACHWEISANSÄTZE

Der Ergebnisvergleich aller Nachweisansätze ist in Abbildung 3-1 und Abbildung 3-2 dargestellt. Aus den Ergebnissen können folgende Rückschlüsse gezogen werden:

- Das lineare Kragarmmodell liefert unrealistisch kleine aufnehmbare Beschleunigungen und ist in der Praxis nur schwer einsetzbar, da selbst die Nachweise in der niedrigsten Erdbebenzone nur mit Problemen durchführbar sind.
- Mit dem neuen Vorschlag des Verhaltensbeiwerts ergeben sich deutlich höhere aufnehmbare Beschleunigungen. Diese liegen im Vergleich zum Kragarmmodell etwa um den Faktor 2 höher. Die Ergebnisse des neuen Ansatzes liegen aber noch deutlich unter den genaueren nichtlinearen Nachweisen, welche die Reserven besser ausnutzen.
- Die Schubwandtabellen nach DIN EN 1998-1-2010 [6] liefern Ergebnisse zwischen dem Kragarmmodell und dem neuen Vorschlag für den Verhaltensbeiwert. Die Anwendung ist deutlich eingeschränkt für niedrige Festigkeiten.
- Der Vorschlag für die neuen Schubwandtabellen aus Kassel [21], [22] ist optimistischer und liegt deutlich über dem neuen Vorschlag für den Verhaltensbeiwert. Für Spektren mit einem ausgeprägten Plateaubereich ergeben sich aufnehmbare Beschleunigungen im Bereich der nichtlinearen Berechnungsergebnisse. Die Gründe hierfür sind in WP3-RWTH [12] erläutert. Die Anwendung wurde erweitert auf niedrige Festigkeiten und größere Geschossanzahlen.

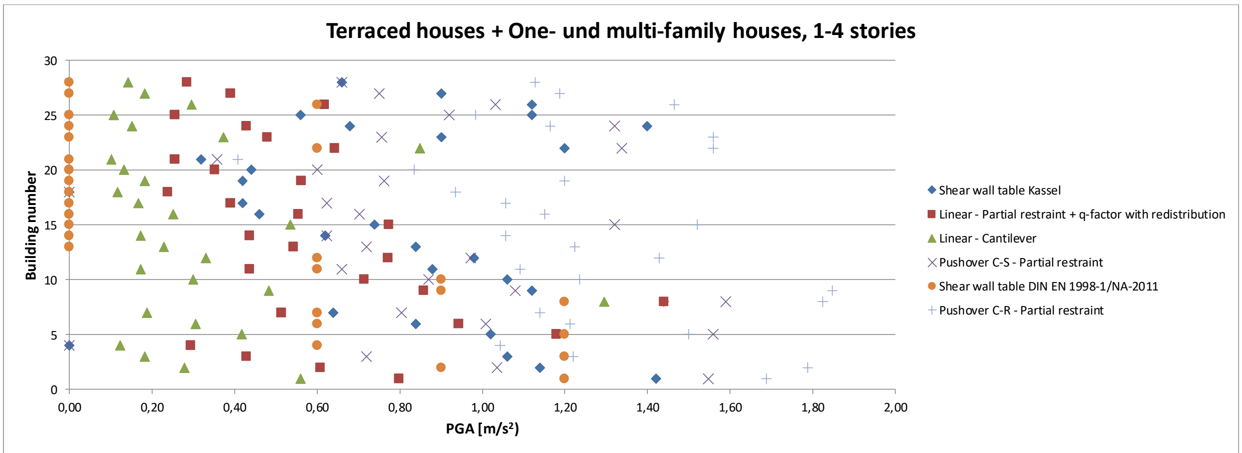

Abbildung 3-1: Ergebnisse der Berechnungsergebnisse für die 28 Gebäude für die verschiedenen Nachweisverfahren

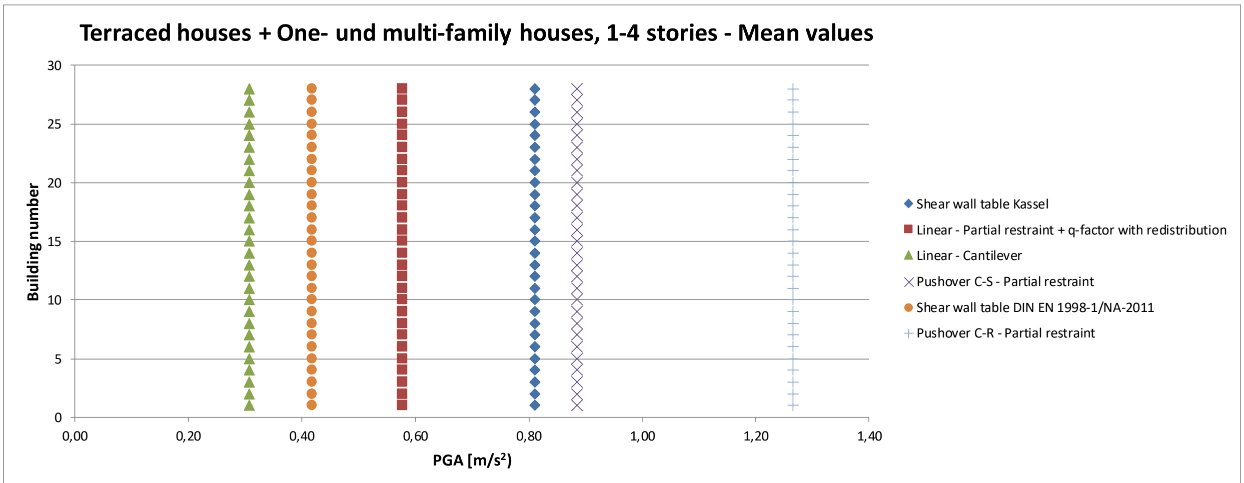

Abbildung 3-2: Mittelwerte Berechnungsergebnisse für die 28 Gebäude für die verschiedenen Nachweisverfahren

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