Seismic Assessment of Historical Brick-Masonry Buildings in Vienna

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SUMMARY:
This paper presents a methodology for the seismic assessment of Viennese brick-masonry buildings, which were built between 1848 and 1918. These buildings are particularly vulnerable to horizontal loads, and thus their seismic resistance cannot be verified with traditional methods of analysis. The presented methodology is based on the capacity spectrum methodology, and it comprises visual inspection, dynamic in-situ tests, and numerical simulation of the non-linear behavior of the brick-masonry walls against horizontal in-plane loading. In an example problem the seismic resistance of a four story historical residential building is verified.

Keywords: Historical brick-masonry building; In-situ measurements; Capacity spectrum methodology; Pushover analysis; Numerical modeling

1. INTRODUCTION

The City Center of Vienna is dominated by historical residential brick-masonry buildings, which were constructed during a major urban expansion in the period between 1848 and 1918 referred to as “Gründerzeit”. At present, one third of the complete building stock in the urban area of Vienna, that is 32,000 objects, consists of these Viennese brick-masonry buildings. They shape the urban image of Vienna, and thus, make the city attractive for visitors. As an example, Fig. 2.1 shows the façade of the street front and a floor plan of a representative Viennese brick-masonry building.

Recently, the behavior of Viennese brick-masonry buildings under seismic loads has become of significant interest, because Eurocode 8 (EC8 2004) imposes additional seismic demands to these structures compared to the previous standards. As a consequence, their seismic resistance cannot be verified anymore with traditional methods of analysis available to the design engineer in practice. This has led to a drastic decline of rehabilitation and reconstruction of this building type, and consequently to huge economical losses. Thus, in 2006 the Austrian national research project SEISMID (Achs et al. 2011) has been launched aiming at providing a methodology for the assessment of the actual load-bearing capacity of these buildings under earthquake excitation. In this paper this methodology is summarized. As an example, the seismic resistance of a residential building from 1895 is assessed.

2. VIENNESE BRICK-MASONRY BUILDINGS

2.1. Building Characteristics

2.1.1. Buildings in their original condition

Viennese brick-masonry buildings have usually four to five stories. The load-bearing walls were built of brick masonry. Thereby, solid bricks of the so-called “Old Austrian” format with dimension $290 \times 140 \times 65$ mm arranged in various patterns are placed in mortar. In historical building regulations it was specified that structural analysis could be skipped if a minimum wall thickness was met. Thus, compared to modern buildings the load-bearing walls are relatively thick. Partition walls were also built of brick-masonry with a thickness of 14 cm. Since, in an untouched building, they are vertically continuous through all floors, they increase the lateral stiffness of the object. Above the basement massive brick vaults were constructed, which provides the basement with a large lateral stiffness. Timber was used for the ceilings and the roof structure. The ceilings were composed of timber beams, over which board flooring was laid. The load-bearing beams were placed perpendicular to the exterior and interior load-bearing walls, as shown in Fig. 2.1b. Usually, they were connected only in direction of the beam axis to the walls.

2.1.2. Remodeled buildings

The conversion of empty attics into apartments requires structural modifications, which are regulated for Viennese brick-masonry buildings in a specific national standard (MA 37 2008). It is regulated that for lightweight constructions up to 7.2 kN/m^2 only minor structural measures are required. The most important required measure is strengthening of the ceiling between the last floor and the attic by a thin reinforced concrete layer or a horizontal steel truss, without increasing significantly the mass.

2.2. Structural Behavior and Seismic Vulnerability

2.2.1. Buildings in their original condition

Typically, the basement is relatively rigid, and thus, only the structure above the brick-vaults is vulnerable against seismic loads. When located in between two buildings, these objects are particular vulnerable in transversal direction (denoted as *x*-direction, Fig. 2.1b), because the resistance against horizontal loads is provided only by the gable walls, the walls of the staircase, and partition walls (these elements are denoted by W1 to W12 in Fig. 2.1b). The timber floors of an untouched building are relatively flexible with questionable floor-wall connection. Therefore, the assumption that they are acting as rigid diaphragms and distributing the inertia forces from the floors onto the walls is less valid compared to, for example, composite timber-reinforced concrete floors (Lang 2002).

Consequently, the earthquake resistance of an untouched building against global collapse is governed predominately by the brick-masonry gable walls, the load-bearing walls of the staircase, and the lateral partition walls, if they are continuous from their support at the brick-vaults to the attic. Since these shear walls are not reliably connected to the load-bearing walls, ceilings, and the roof in a simplified seismic analysis the interaction between these elements cannot be accounted for (MA 37 2008). In the corresponding analysis model each transversal shear wall must be considered separately, as shown in the schematic view of Fig. 2.2a. Clearly, beneficial effects of load-distribution and increased global stiffness through this interaction are not considered, and as a result, the seismic resistance of the object may be underestimated considerably.

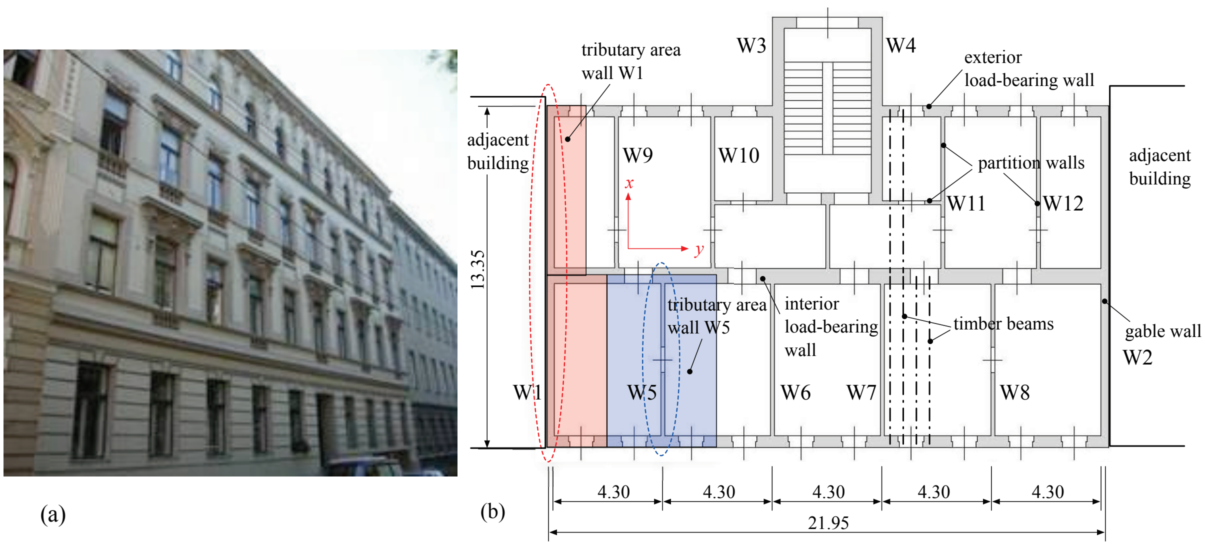


Figure 2.1. Representative Viennese brick-masonry building. (a) Façade. (b) Floor plan (Bauer et al. 2010)

2.2.2. Remodeled buildings
In a remodeled building the strengthened ceiling may be considered as a more or less rigid diaphragm, and thus, at the roof level the lateral displacements of the transversal shear walls are compatible when subjected to seismic loading, as it is shown schematically in Fig. 2.2b. The inertia forces are distributed from this floor to the lateral walls according to their stiffness. Additionally, the strengthened ceiling is connected to the gable walls, and, as a result, also the vertical load distribution is changed, increasing the axial loads in the gable walls. Thus, the strength of the gable walls is increased against horizontal loads.

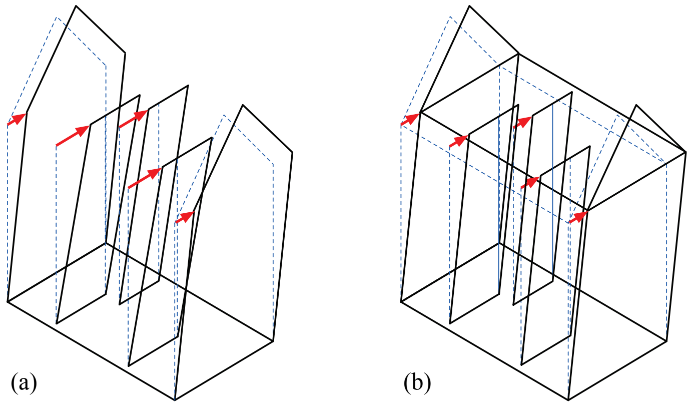


Figure 2.2. Horizontal displacements of the transversal walls subjected to seismic loads. Schematic scetch. (a) Untouched object. (b) Modified object with strengthened ceiling at the roof level (Krakora 2008)

3. ASSESSMENT OF THE SEISMIC RESISTANCE

In the most general approach the resistance of a structure against earthquakes is assessed through repeated non-linear time history analyses using a set of recorded ground motion. However, for the considered building type non-linear time-history analyses are not feasible, because the hysteretic constitutive behavior of the structural elements and the cyclic connection behavior between walls and ceilings has not been investigated adequately, and the computations are much too time-expensive.

On the contrary, traditional static methods of seismic analysis, which are based on several simplifications such as pure linear elastic structural behavior, disregarding the effect of timber ceilings, etc., cannot proof the stability of Viennese brick-masonry buildings against collapse even for moderate earthquake events. Thus, a more realistic assessment of the seismic resistance with acceptable expenses requires the consideration of the inelastic path of deformation of the shear walls.

3.1. Capacity Spectrum Methodology

In the last decade the capacity spectrum methodology (Fajfar 1999) became a popular tool for assessing the seismic resistance of regular structures. It represents a compromise between time-history analyses with complex modeling strategies and oversimplified static analysis methods. This method can be applied successfully provided that the dynamic response is dominated by bending vibrations in the fundamental mode, which should be well separated from the higher modes.

In the capacity spectrum approach a global lateral load-displacement relation represents the seismic capacity of a structure, which is the outcome of a non-linear static analysis. In a so-called pushover analysis, the model of the considered structure is loaded by the dead weight, and subsequently a horizontal in-plane load with a pattern close to the fundamental mode shape is applied, compare with Fig. 3.1a. The horizontal loads are incremented in a displacement-controlled procedure up to structural collapse. Plotting the horizontal displacement u_r of a characteristic point, i.e. in general the roof displacement, against the base shear V renders the global pushover curve of the structure.

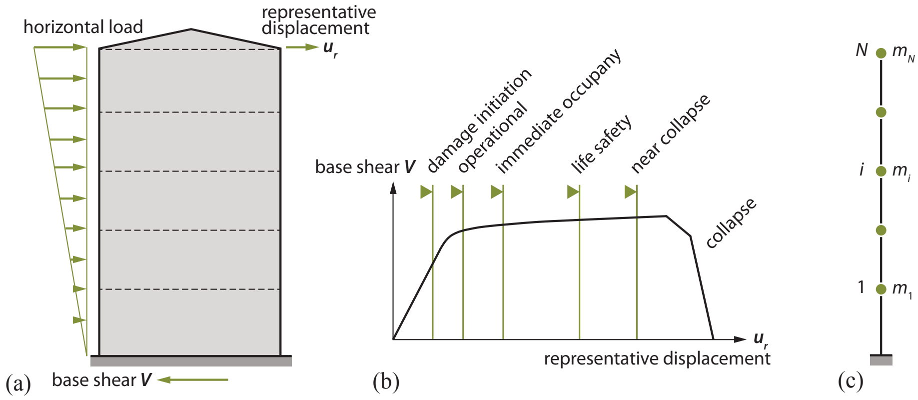


Figure 3.1. (a) Pushover analysis: Horizontal load pattern, representative displacement, base shear. (b) Global pushover curve. (c) Simplified mechanical model

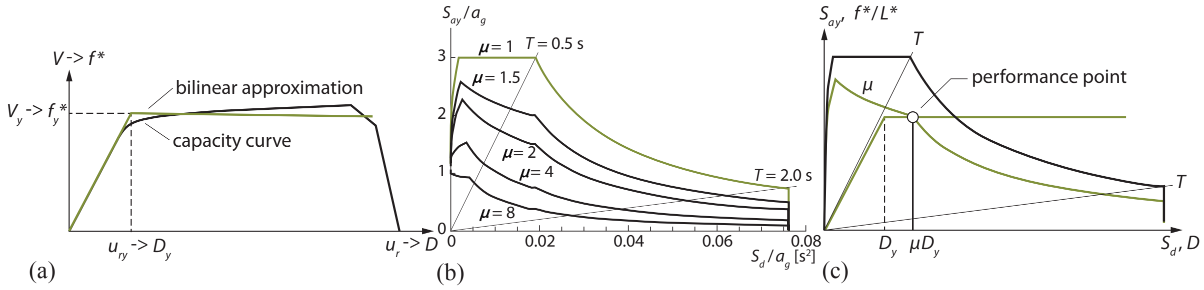


Figure 3.2. (a) Elastic and corresponding inelastic capacity spectra. (b) Capacity curve. (c) Performance point

The global pushover curve of the wall (Fig. 3.1b) is transformed into the domain of the equivalent SDOF (ESDOF) system of the simplified dynamic model. This model consists of lumped masses assigned to the story levels , see Fig. 3.3c. The following relations govern the transformation of the base shear V and displacement u_r into the domain of the ESDOF system (EC8 2004):

$$f^* = V / \Gamma \ , \ \ D = u_r / \Gamma \ , \ \ \Gamma = L^* / m^* \ , \ \ L^* = \sum_{i=1}^{N} m_i \phi_i \ , \ \ m^* = \sum_{i=1}^{N} m_i \phi_i^2 \tag{1}$$

Thereby, f^* is the equivalent spring force, and D the displacement of the ESDOF system. m_i denotes the lumped mass assigned to the ith story, and ϕ_i is the ith element of the shape vector, which approximates the fundamental mode shape. Note that ϕ_N is one, N denoting the total number of stories. The transformed global pushover curve $f^* - D$ (Fig. 3.2a) represents the capacity of the building to resist horizontal loads. Thus, it is referred to as capacity curve. Dividing f^* by the equivalent mass L^* renders its ordinate in the dimension “acceleration”. This transformation permits the comparison of the capacity curve with the seismic demand represented by a capacity spectrum of the actual site. A capacity spectrum represents the maximum spectral peak acceleration S_a of an earthquake excited SDOF system plotted against the corresponding peak spectral displacement S_d for a series of target ductilities μ (Fajfar 1999), as shown in Fig. 3.2b. Note that the ductility defines the ratio of the maximum imposed (inelastic) deformation to the deformation at onset of yield. In the subsequent step, the intersection point between a bilinear approximation of the capacity curve and the capacity spectrum is searched. If this point is in the inelastic branch of deformation, the elastic spectrum must be reduced so that the ductility of the capacity curve at this point and of the corresponding inelastic spectrum coincide, Fig. 3.2c. This intersection point, which is referred to as performance point, is found iteratively. If no performance point can be found, because the ductility of the structure is too low, the resistance of the structure against the imposed earthquake cannot be verified. For details of this methodology the reader is referred to Fajfar (1999).

3.2. Proposed Seismic Assessment Methodology for Viennese Brick-Masonry Buildings

In the following a methodology for assessing the seismic resistance is presented, which was developed particularly for Viennese brick-masonry buildings. It is based on the capacity spectrum methodology, and it comprises visual inspection, dynamic in-situ tests, and numerical simulation of the non-linear behavior of the brick-masonry walls against horizontal in-plane loading. Subsequently, the individual steps of the methodology are described in more detail.

3.2.1. Visual inspection

A Rapid-Visual-Screening method, which has been developed for Viennese brick-masonry buildings, is applied to assess the actual condition of the considered object and its vulnerability to earthquake loads. It is based on two parameters, i.e. the damage relevance and the structural parameter. The damage relevance includes parameters to evaluate the social and economic impact of seismic damages on a certain building. One of the main parameters of the damage relevance is the number of exposed persons within the inspected object. The structural parameter mainly consists of individual indicators to describe certain structural parts of the building itself. The most important parameters, which can be directly related to earthquake-induced damage, are the regularity of the building in elevation and the state of preservation. The combination of damage relevance and structural parameter is used to categorize the building into a vulnerability class. For details it is referred to Achs and Adam (2011).

3.2.2 Dynamic in-situ tests

Dynamic in-situ tests are conducted to identify the fundamental natural frequencies and mode shapes. In an occupied building only ambient vibration tests are feasible. They should be conducted in the night, when no disturbing signals are to be expected. Alternatively, recording the free vibration response induced by an impact leads to more reliable outcomes, because the vibration amplitudes are much larger. However, the building should be unoccupied. It is proposed to use triaxial capacitive accelerometers to record the dynamic response. In an occupied building the sensors can only be placed in the public domain of the building, i.e. in the staircase, corridors, and attic. It is proposed to distribute the sensors in vertical direction story-wise on top of each other in the staircase, and in horizontal direction in the attic to capture both bending and torsional modes. A direct connection of the sensors to the structural elements is essential to capture only the global dynamic behavior of the building, and to avoid interfering signals from local dynamic effects. Thus, the accelerometers must be directly mounted to the load-bearing walls in windowsills and alcoves. The recorded acceleration time histories are transformed into the frequency domain by Fast-Fourier-Transformation yielding the spectral acceleration response. From these data natural frequencies and mode shapes are identified. For details see Achs et al. (2011).

3.2.3. Calibration of structural parameters

A full elastic three-dimensional finite element model of the building including partition walls and ceilings is developed. It is important to consider the stiffness of the adjacent buildings by distributed springs attached to the gable walls. Numerical modal analysis is performed, and numerically derived natural frequencies and mode shapes are compared with the outcomes from the dynamic in-situ tests in an effort to verify the assumptions for the mass distribution within the building, the initial elastic material properties of the structural elements and distributed springs. Note that Furtmüller (2010) has derived a set of orthotropic-elastic parameters for the considered Viennese brick-masonry made of running bonds, which is proposed to utilize. Subsequently, these properties are adjusted iteratively so that the dynamic parameters are sufficiently close to those obtained in the experiments.

The outcomes of the dynamic in-situ tests and the corresponding numerical modal analysis deliver information, whether the capacity spectrum methodology can be utilized for assessing the seismic resistance. The capacity spectrum methodology requires that the fundamental mode is well separated from the higher modes, and describes predominantly bending vibrations in a distinct direction. In general, for the considered building type these requirements are met for objects that are not located at corners of a building block.

3.2.4. Numerical modeling of the shear walls for pushover analysis

In the presented methodology the transversal brick-masonry walls are subjected to a pushover analysis individually. To capture the complex non-linear structural behavior of each biaxially loaded shear wall a numerical finite element model based on plane-stress finite elements is established. For the present pushover analysis on a larger scale it is proposed to assign nonlinear homogenized material properties in the framework of multi-surface plasticity theory to the finite element model, which captures the fundamental failure modes of in-plane loaded Viennese brick-masonry. The underlying macro material model is a refined multi-surface plasticity model (Lu and Heuer 2007). Thereby, five yield functions f_1 to f_5 confine the elastic domain, as depicted in Fig. 3.3a. Each yield function can be related to a particular failure mode. Function f_1 describes tensile failure of the bricks, f_2 compressive failure of the bricks, f_3 shear failure of the bricks, f_4 shear failure of the bed joints, and f_5 tensile failure of the joints (Lu and Heuer 2007). Modified yield functions f_1 to f_5 as described in Furtmüller and Adam (2011) are formulated for a plane stress state, i.e. normal stresses σ_x and σ_y, and shear stress τ_{xy}. The underlying xy-coordinate system is oriented in the directions of the mortar joints, the x-axis being horizontally and the y-axis vertically aligned. Hardening and softening is considered by scaling the strength parameters with scalar multipliers $\Omega_i(\kappa)$ ranging from 0 to 1 according to a formulation of Schlegel (2004). As an example, Fig. 3.3b shows multiplier Ω_{fc} for the compressive strengths f_{cx} and f_{cy} as a function of the corresponding internal variable κ_{fc}. For details see Furtmüller (2010).

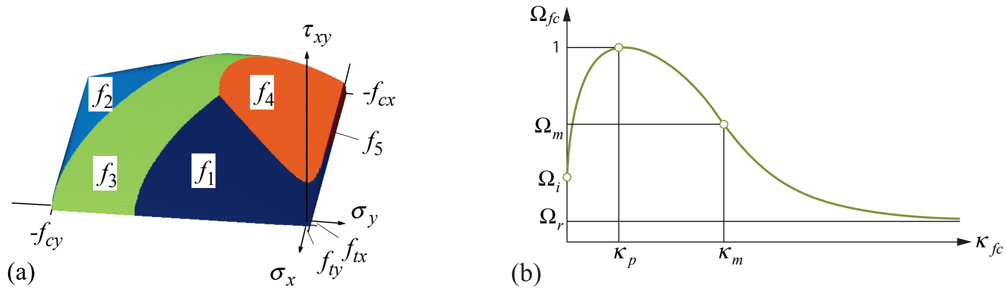


Figure 3.3. (a) Composite yield surface of the macro-model. (b) Hardening/softening in the compressive regime

Based on comprehensive experiments on specimens of historical bricks, mortar, and small-scale brick-masonry elements under various loading conditions, a corresponding set of material parameters for Viennese brick-masonry in the pattern of a running bond has been derived. Thereby, a numerical homogenization procedure was performed yielding the transition from the meso-scale, where bricks and mortar are treated separately, to the macro-scale. For details see again Furtmüller (2010).

3.2.5. Capacity curve

The considerations for the capacity curve are different whether an untouched building in its original condition or a modified building with strengthened floor above the last story is assessed.

As it has been outlined previously in an untouched building it is assumed that no loads are distributed from the timber ceilings to lateral shear walls, and thus, the shear walls must be considered independently. However, only the “weakest” shear wall against seismic loads must be analyzed, because this wall governs the seismic resistance of the entire building. This wall must resist the horizontal seismic loads acting on the wall and on tributary area of the adjacent building elements. In the pushover analysis, the self-weight of the wall and the dead weight of adjacent building elements (only of those acting vertically on this wall) is imposed to the top of the numerical model, and subsequently a horizontal in-plane load with a *linear* horizontal load pattern is applied. The pushover curve of this wall is transformed into the domain of the corresponding ESDOF system, which is based on a *linear* shape vector, compare with Eqn. 1. Thereby, at each story level a lumped mass is assigned to the wall. The surface area of each wall (and not the stiffness) determines the horizontal tributary area of the lumped mass. As an example, in the floor plan of Fig. 2.1b the tributary area for gable wall W1 (shaded in red), and for partition wall W5 (shaded in blue) is plotted. It is emphasized that the tributary area for the lumped masses and for the vertical loads is in general different. The natural frequencies of the ESDOF system and the building do not coincide, because only one individual shear wall is considered neglecting the impact of interaction to the other building elements.

For a building with strengthened floor the capacity curve of each lateral wall, which transfers seismic loads to the foundation, is determined. Remember that strengthening of the floor changes the vertical load distribution. According to Lang (2002) the individual pushover curves can be superposed: $V = V_1 + V_2 + V_3 + ...$, compare with Fig. 3.4. The resulting curve represents an approximation of the capacity of the entire building in its vulnerable direction. The total mass of the building is assigned to lumped masses located at the story levels. Since the capacity of the complete building is considered, the fundamental natural frequency of the ESDOF system and the object should be closely spaced.

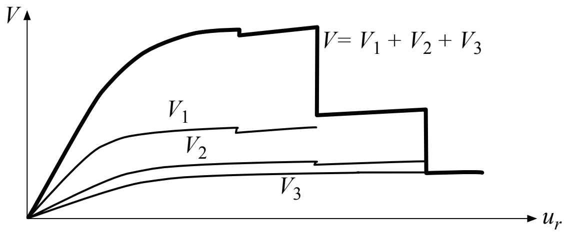


Figure 3.4. Strengthened ceiling: Individual capacity curves of shear walls and capacity curve of the building

3.2.6. Application of the capacity spectrum methodology

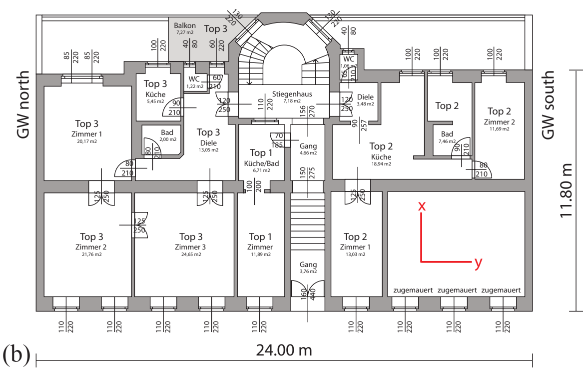
The capacity spectrum methodology is applied to check the earthquake resistance of the building, based on the capacity curve, and on the capacity spectrum describing the seismic demand on structures located in Vienna (EC8 2004). In order to capture the local site condition it is recommended to adapt the capacity spectrum according to the outcomes of a microzonation (Achs et al. 2011).

4. APPLICATION

4.1. Considered Building

Based on the proposed methodology subsequently the earthquake resistance of the object "Riglergasse 10" located in the 18th district of Vienna is assessed. The residential building, which was constructed in 1895, has a basement, a ground floor, and three essentially identical upper floors. The building height is 18.70 m. A vaulted brick-slab separates horizontally the basement and the ground floor. The brick-masonry load-bearing walls have a thickness up to 70 cm, the thickness of the partition walls is 15 cm. In the upper floors the ceilings are built of closely spaced timber beams. In 2006 the attic was converted into apartments using a lightweight construction. Fig. 4.1a shows the front of the building facing the street, in Fig. 4.1b the floor plan of the ground floor is depicted.

(a)



(b)

Figure 4.1. Object "Riglergasse 10" located in the City of Vienna, Austria. (a) Building front facing the street Riglergasse. (b) Floor plan of the ground floor

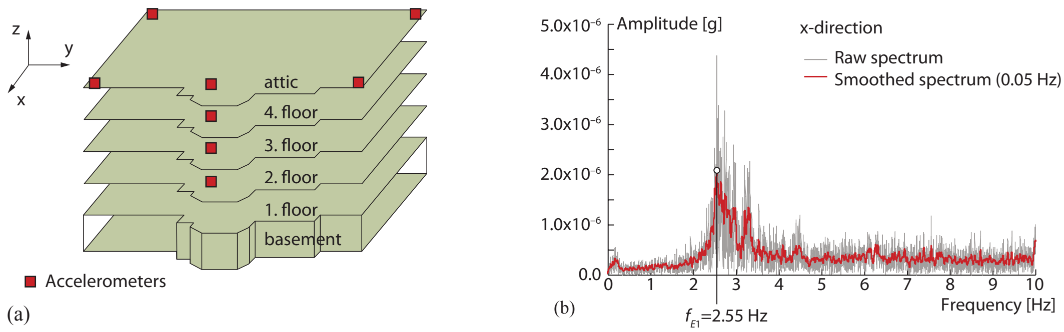


Figure 4.2. (a) Position of the accelerometers. (b) Frequency spectrum of the response in *x*-direction recorded at the top of the staircase

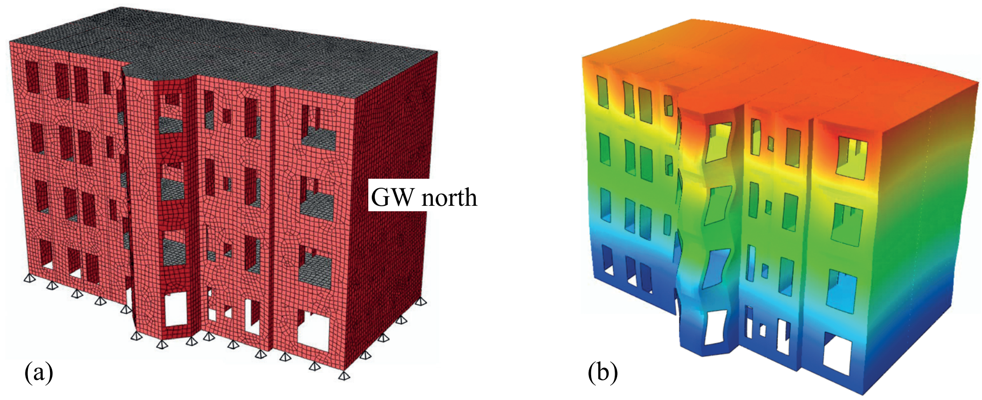


Figure 4.3. (a) Finite element model for numerical evaluation of the natural frequencies and mode shapes. (b) Fundamental mode shape (fundamental frequency f_{N1} = 2.75 Hz)

4.2. Dynamic In-Situ Tests

In 2006 before and after the building modification dynamic measurements were conducted to identify its dynamic properties (Achs 2011). Since the object was occupied, ambient accelerations were recorded by accelerometers, which were vertically distributed in the staircase and horizontally distributed in the attic, compare with Fig. 4.2a. Exemplary, Fig. 4.2b shows the frequency response in *x*-direction recorded with the accelerometer at the top of the staircase. From the response data the first two fundamental natural frequencies of the untouched object were identified: f_{E1} = 2.55 Hz and f_{E2} = 4.35 Hz. The corresponding mode of frequency f_{E1} describes a bending vibration in *x*-direction (coordinates *x* and *y* are depicted in Fig. 4.1b), while the mode of frequency f_{E2} is a coupled bending-torsional mode, which is significantly affected by the adjacent buildings.

4.3. Numerical Evaluation of Natural Frequencies and Mode Shapes

A three-dimensional finite element model of the untouched building was created as shown in Fig. 4.3a. Thereby, the stiff basement embedded in the soil was assumed to be rigid. Triangular and rectangular shell elements were used to discretize walls and ceilings. To the masonry orthotropic-elastic parameters of a running bond were assigned, as specified in Furtmüller (2010). The mass density was assumed to be 1,700 kg/m^3. The timber ceilings were approximated by equivalent isotropic-elastic plates of 10 cm thickness with Young's modulus E = 10,000 N/mm^2, Poisson's ratio ν = 0.2, and mass density ρ = 1,500 kg. Dead loads were considered via uniformly distributed masses. For the ceiling above the ground floor this mass was 300 kg/m^2, above the first and second floor 200 kg/m^2, and for the ceiling (including the roof) above the third floor 500 kg/m^2. The displacements of the nodes at the support were fixed. Numerical modal analysis resulted in the following first two natural frequencies: f_{N1} = 2.75 Hz, f_{N2} = 4.29 Hz. These values coincide sufficiently well with the experimentally determined counterparts f_{E1} and f_{E2}. The mode shape, which corresponds to fundamental frequency f_{N1}, is shown in Fig. 4.3b. It is readily seen that this mode shape describes a bending vibration in x-direction, as it was observed in the experiments. It can be concluded that this mode, which is well separated from the higher modes, dominates the global vibration behavior of the building. Thus, an essential requirement for application of the capacity spectrum methodology is satisfied.

4.4. Global Load-Displacement Relation of the Governing Load-Bearing Wall

In x-direction horizontal loads are mainly transferred via the gable walls at both ends of the building, and the walls of the staircase to the foundation. Since no lateral partition walls run vertically continuous from the ground floor to the roof, their contribution to the resistance of the building against earthquakes is negligible. The shorter gable wall denoted in Fig. 4.1b as "GW north" is assumed to be the wall, which governs the earthquake resistance of the complete building, because its strength is expected to be smallest. Fig. 4.4a shows the numerical model of the governing wall for the pushover analysis. To this model the elastic and inelastic homogenized material parameters of Viennese brick-masonry, as derived in Furtmüller (2010), were assigned. The boundary domains to the perpendicular external walls were modeled elastically. The wall was discretized by means of 74,814 reduced integrated finite elements with linear shape functions and altogether 150,748 degrees-of-freedom. The permanent vertical loads from the self-weight of the wall, and the dead weight of the structural elements resting on the wall were imposed to the top of the wall. Subsequently, in a displacement controlled analysis the horizontal in-plane load was incremented. The pattern of this load is linear. Fig. 4.4b shows the global pushover curve of the wall "GW north", where the base shear V is plotted against the horizontal displacement u_r of the uppermost point of the wall. In these computations the global softening regime was not achieved. It can be observed that this curve tends to a plateau at a base shear of about 950 kN. The discontinuities (e.g. at u_r = 29 mm) of the curves can be led back to locally brittle behavior.

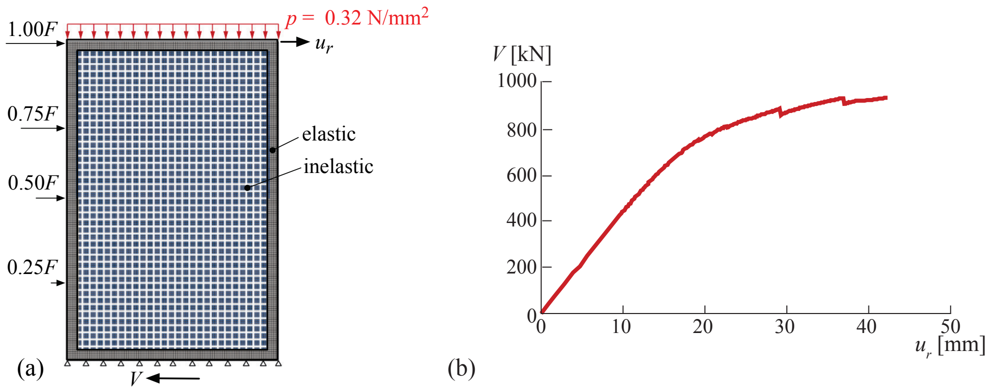


Figure 4.4. (a) Numerical model of the wall "GW north". (b) Global pushover curve of the wall "GW north"

4.5. Assessment of the Earthquake Resistance

The subsoil is classified according to EC8 (2004) as class D (deposits of loose-to-medium non-cohesive soil, or of predominantly soft-to-firm cohesive soil). The reference peak ground acceleration is a_{gR} = 0.8 m/s^2. The 5% damped elastic capacity spectrum of the building site according to EC8 (2004) is shown in Fig. 4.5b by a solid thin green line. The lumped masses of the corresponding dynamic four-degree-of-freedom system include 37.5% of the total building mass, see Fig. 4.5a. For the considered wall the coefficients Γ and L^*, Eqn. 1, which transform the MDOF structure into the corresponding ESDOF system, are obtained as: Γ = 1.211, L^* = 341,000 kg. Then, the capacity curve f^* / L^* against D, depicted as a red line in Fig. 4.5b, is obtained. A black solid line represents its elastic-perfectly plastic idealization. It is readily observed that the capacity curve and the capacity spectrum intersect in the inelastic range of deformation. Iteration leads to the inelastic capacity spectrum for a ductility $\mu = 1.4$ plotted with a solid continuous green line. The corresponding ductility of the capacity curve at the performance point is $\mu = 1.4$ as well (as required). The spectral displacement at the performance point of $D = 21$ corresponds to a roof displacement of $u_{r^*} = 25.4$ mm in an earthquake according to EC8 (2004). It can be concluded that the earthquake resistance of the representative load-bearing wall complies with the regulations of EC8 (2004), if inelastic deformations are considered.

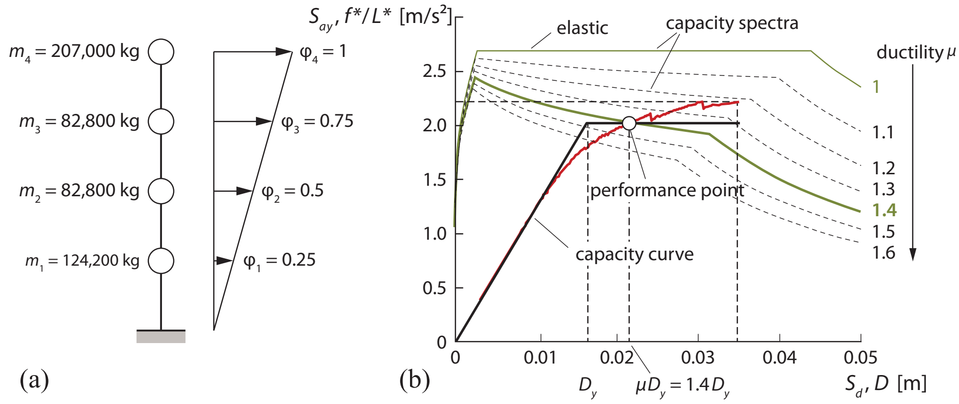


Figure 4.5. (a) Dynamic model of the wall "GW north". (b) Capacity spectra, capacity curve, performance point

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