7

Masonry and Earthquakes: Material Properties, Experimental Testing and Design Approaches

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1. Introduction

Earthquakes are natural disasters which can occur without warning and they can affect large areas. Earthquakes are often accompanied by aftershocks which may cause additional damage to an already damaged structure or lead to failure. Consequences of earthquakes, such as rock falls, fires, explosions etc., can be very large in the affected areas. An example is the 1906 earthquake in San Francisco. Thereby many lives were killed by the ensuing fire. But not only past events have had fatal consequences, also events of recent years caused countless deaths and consequential damages e.g. the 2010 earthquake in Haiti with a death toll of more than 250,000 people (Eberhard et al., 2011) or the 2011 earthquake in Fukushima, Japan with high consequential damages e.g. to the nuclear power plant (Takewaki et al., 2011) or both earthquakes in Christchurch, New Zealand with high damage on cultural heritage (Ingham & Griffith, 2011; Ingham et al., 2011).

2. Masonry

2.1 Historical overview in masonry construction

Masonry is one of the oldest types of construction. Taking account structural-physical properties and the quite easy construction process, this construction system is used until today. The building material brick was easy to produce. In ancient times, clay was put into a model, the surface was smoothed and then the brick was exposed to air for drying. In later times the raw material was extruded and baked in a kiln. Masonry construction methods were already well known in ancient times, about 5000 BC bricks were used in Mesopotamia. A well-known example for the usage of masonry in these times is the tower of Babel, which was built around 600 BC. Clay bricks were used as building material and bitumen as mortar. Even then masonry offered a faster and cheaper alternative to natural stones in the construction process.
In contrast to natural stone construction methods, manufacturing of regular building stones was a revolution and enables systematic construction methods. Monumental structures, for instance the Pantheon in Rome were built by using masonry. Masonry construction methods do not offer the possibility of build plane top panels and lintels, therefore the construction of arches was developed and enhanced during that period (Maier, 2002).
Masonry always has to be constructed in bond to guarantee an adequate bearing capacity. Depending on the time period, different bond types were used, additionally it has to be

distinguished between pure and mixed masonry. Especially in the Early Middle Ages, mixed masonry was constructed. The facings of the wall were built with bricks but the core was filled up with quarry stones and lower quality bricks. Later on, masonry was used through the whole thickness of a structure, with continuous horizontal joints in each row, independent from the thickness of the wall.
By placing bricks in an adequate way, a cross bonding through the whole thickness of the wall was achieved. As a result of this the decisive bond of the bricks depends on the thickness of the wall. Walls with a thickness of the width of a brick usually were built in a stretching bond and walls with a thickness of the length of a brick in a heading bond. Further masonry was built with different bond types e.g. Markish or Wendish, Gothic or Monastic bond. Thereby in every brick layer one header brick is followed by a few stretcher bricks.
In the 16th century, structured bonds like cross bond and block bond were used, in which a stretcher course is followed by a header course. In the 17th century, the Dutch or Flemish bond has been used. Thereby a header course is followed by a mixed layer of headers and stretchers. The common bond methods are depictured in Fig. 1.
The bearing capacity of a masonry bond is reached by avoiding vertical joints which go through multiple layers of construction stones. An additional capacity can be reached by using anchorages, which set up a force-fit connection of opposite walls and especially of masonry and wooden floor slabs. In the rebuilding period after World War II masonry construction became important again.
Developing of reinforced concrete and enhancing of the corresponding construction methods showed the limits of masonry construction methods. Nevertheless masonry is still used in restoration of historic buildings and in housing construction. It becomes more important again during the last years, because of the increasing requirements to thermal insulation, almost special large-sized honeycomb bricks are used.

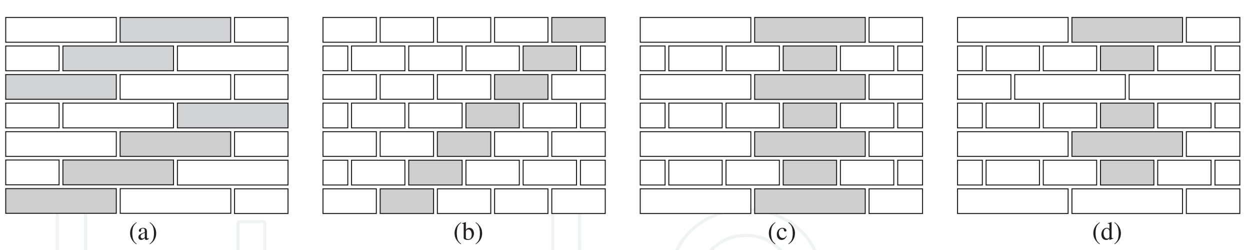


Fig. 1. Various bonds of masonry (a) stretching bond, (b) heading bond, (c) block bond, (d) cross bond; (Bargmann, 1993)

2.1.1 Bricks

In contrast to natural stones, which have to be cut to the appropriate form and size, artificial construction stones as bricks are shaped by the models they are made with. The requirements to construction stones and mortar are defined in various codes. These codes define basic materials, definitions, dimensions, classes of raw density and strength and the assurance of quality.
Nowadays research objectives in brick design should offer the following properties: (a) increasing of efficiency in production process of construction stones; (b) increasing of the bricklayer's capacity; (c) increasing of the bearing capacity; (d) increasing of the building physics.

An important change in the production process was made with the industrial revolution, when the process was automatized. The change from the manual handling to automatically extruding machines yields in constant shapes. Moreover it ensures an alignment of the clay minerals due to the high pressure. This orientation of the clay minerals is the texture. It causes the anisotropic material behaviour depending on the direction of the brick. During the time various shapes and dimensions of bricks for different purposes occurred.

2.1.2 Mortar

Mortar is used to build up compensating layers to connect not accurately fitting bricks to an accurate and usable masonry. The most important characteristics of masonry mortar are its processability, its plasticity, water retention capacity, compressive strength and the bond strength between bricks and mortar. The components of mortar are binding agent, additives and water. Throughout additives and admixtures certain properties can be achieved.

The classification of mortars is based on the type of the binding agent, the production process and the type. Binding agents can be distinguished in non-hydraulic and hydraulic binders.

Non-hydraulic binders become hardened only on air and must not be under permanent moisture and water influence. Non-hydraulic binders are loam, common lime, gypsum, anhydrite, magnesia and fireclay.

Hydraulic binders become hardened both on air and under water and are resistant against permanent moisture. Hydraulic binders are cement, special types of lime and mixed binders. Mortars can be produced on construction side or in plant.

Lime was used as binding agent for masonry verifiable in 1000 BC (Hilsdorf, 1965) and it is up to now the determining binding agent for masonry mortar. Already in ancient times it was well known that admixtures like volcanic ashes or brick powder have a positive influence on strength and moisture resistance of the mortar (Grimm, 1989). Organic components enhance or modify manufacturing and hardening process, especially casein has an improving influence on water demand and water retention capacity (Conrad, 1990).

Extensive analyses of historic mortars have shown information about their composition, mixture and characteristics, which are quite different to modern mortars (Schäfer & Hilsdorf, 1990; Wisser & Knöfel, 1987). Additional components like tuff and puzzolan can be added to lime mortar to make the mortar hardening under water.

Table 1 gives an overview of different mortar types. Nowadays the mortar layer becomes thinner and thinner because the surface of modern bricks is very flat ensured by the production process. Further the thermal properties of mortar are improved continuously.

2.2 Material properties: Bricks, mortar, masonry

2.2.1 Bricks

Older structures often built with bricks, which are not conforming to today’s ones. A comparison of old bricks in terms of material properties and characteristics can be found in (Egermann & Mayer, 1987). The following types were tested: (a) usual common brick (MZ); (b) extruded brick (SM); (c) hand-smoothed brick (HM); (d) historical brick 1796 (QU); (e) historical brick 1884 (BE).

The bricks SM and HM are made from the same raw material as the usual common brick MZ, but they were baked with 800 °C instead of 1000 °C. The historical brick QU from 1796 has been made manually, and the historical brick BE was formed by a screw extruder.

Loam-mortar	made out of moistured loam with added chaffed components, hardening through drying; application: indoor rooms, well protected outdoor walls, clay floor in agricultures
Gypsum-mortar	made out of gypsum with sand and lime; application: gypsum, sand, lime as wall and ceiling plaster
Lime-mortar	lime-sand mortar is the usual mortar in structural engineering, made out of slaked-lime, sand and water
Lime-cement-mortar	extended cement-mortar is made by add cement to lime-mortar; application: if lime-mortar cannot be used as a result of the type of the bricks, the strength of the mortar and the expected moisture (lime-sand bricks, floating bricks, loaded piles and arches, weather sides and outside wall plaster
Cement-mortar	made out of cement and additional sand; application: for heavily used constructions and for structural members (piles, arches) which are exposed to moisture (foundations)
Raw-cement-mortar	cement, sand and additional components: fly-ash, gravel, stones
Floor-mortar	terrazzo-mortar, magnesia-mortar, xylolite, pavement

Table 1. Mortar types (Bargmann, 1993)

Specimen	Density [g/cm³]		Compressive strength in load direction [MPa]		Splitting tensile strength [MPa]		Elastic modulus in load direction [MPa]		Poisson's ratio in load direction [-]	
	mean	cov	mean	cov	mean	cov	mean	cov	mean	cov
MZ	1.83	1.6	43.0	18.1	3.94	32.4	22669	5.5	0.19	6.4
SM	1.90	1.2	31.3	16.3	3.76	18.0	11867	18.7	0.13	12.5
HM	1.82	1.4	15.6	22.2	1.82	21.7	5716	21.2	0.10	33.5
QU	1.65	4.3	9.5	56.2	0.52	37.4	2726	11.7	0.14	19.1
BE	1.49	72.3	13.9	38.5	2.42	40.1	8379	35.2	0.21	21.9

Table 2. Properties of historical bricks (Egermann & Mayer 1987)

As it can be seen in Table 2, the strength properties of hand-smoothed (HM) and historical bricks (QU, BE) are by trend lower than from machine-made bricks (MZ, SM). The scattering increases due to the manual production process. The consequent enhancement of the oxidation technique mainly led to a reduction of the spreading in the mechanic parameters. On the other hand the shaping methods influence other important parameters which are decisive for research of the structures. As discussed above, the orientation of clay minerals have a significant importance on the compressive strength and can be influenced by the production process. Taking account the compressive strengths of different brick types the results of the experimental study shows that in reference to common bricks (MZ) compressive strength of the extruded bricks (SM) is 73 % and the hand-smoothed (HM) is just 37 %. The compressive strengths of historical bricks (QU, BE) is below the strength of the common bricks.

The bearing capacity of masonry is essentially influenced by the splitting tensile strength of bricks. Common (MZ) and extruded bricks (SM) have approximately the same values for splitting tensile strength, whereas the value of the hand-smoothed bricks is just at 50 %.
If a masonry element is loaded by a centric compressive load, the failure results from cracks caused by tensile stresses. This tensile stress state develops due to different lateral Poisson ratios of bricks and mortar. Additionally, the production process has an important influence on stiffness. In reference to common bricks (MZ) the elastic modulus of the extruded bricks (SM) is 50 % and the hand-smoothed bricks (HM) have a value of about 25 %. As written above, the orientation of the clay minerals causes a relationship of the strength and deformation parameters to the considered direction of loading. Particularly common bricks (MZ) and extruded bricks (SM) show this behaviour. Hand-smoothed and historical bricks (HM, QU, BE) do not show this behaviour according to the considered direction. Therefore it can be concluded, that shaping under high pressure induces an anisotropic material behaviour. In addition to the strength values of construction stones given in Table 3, research results on historical Viennese bricks from the 19th century are given in (Furthmüller & Adam, 2009; Pech, 2010; Zimmermann & Strauss, 2010a). Properties of contemporary brick types, mortar and masonry can be found in (Schubert & Brameshuber, 2011).

Reference	Compressive strength [MPa]		Tensile strength [MPa]		Elastic modulus [MPa]		Fracture energy [Nmm/mm²]		Density kg/m³	
	mean	cov	mean	cov	mean	cov	mean	cov	mean	cov
(a)	29.5	34.6	2.1	33.3	12055	25.6	0.056	3.6	1510	4.0
(b)	22.5	26.6	-	-	-	-	-	-	-	-
(c)	19.3	39.7	-	-	13489	52.6	-	-	1467	6.6

Table 3. Material parameters from historical Viennese bricks from the 19th century
(a): (Furthmüller & Adam, 2009), (b): (Pech, 2010), (c): (Zimmermann & Strauss, 2010a)

2.2.2 Mortar

The percentage of binding agent in the hardened state in comparison to the additives is 1:2 up to 1:3 in reference to weight. The whole hydraulic part of the binding agent, including puzzolanic additives is 10-25 %. The configuration of the mortar has a high influence on the material characteristics. For compressive bearing capacity of masonry the influence factors are the compressive and tensile strength of the mortar and additionally its deformation behaviour. For shear and flexural capacity, initial shear strength and tensile bond strength are the decisive parameters. It is quite difficult to get adequate test specimens from masonry. Therefore these characteristics mostly have to be estimated (Schubert & Brameshuber, 2011).
If masonry is under compressive load, (see Sec. 2.2.3), tensile strength of bricks is decisive for the capacity. The modulus of lateral elongation of the mortar is crucial for the tensile stress state inside the bricks. Compressive strength of mortar is influenced especially by binding agent, the percentage of the components and porosity. Pure lime-mortars have compressive strength in a range from 1.0 to 2.0 MPa. If the hydraulic fraction increases, then also the compressive strength increases, which can be up to 5.0 MPa and more for lime-cement-mortar. If the hydraulic fraction increases, the elastic modulus increases and ductility decreases. Bonding characteristics between mortar and bricks can be specified

with the following parameters: (a) shear strength f_{vk}; (b) tensile bond strength f_{hz}; (c) coefficient of friction μ. Bonding characteristics are mainly influenced by the mortar type and its components. The type of bricks and further the moisture characteristics has an influence.

2.2.3 Masonry

Masonry is defined as a composite material. The bearing behaviour under compressive, tensile, flexural and shear load is different to homogenous materials like concrete or steel. The composite material itself consists of the singular bricks, horizontal and vertical joints. Depending on the scale, masonry can be seen as (a) inhomogenous or (b) homogenous construction material. In case (a) the characterisation has to be made separately for bricks, for mortar and their interaction, in case (b) the characterisation can be done with global parameters on the smeared masonry element; see Fig. 2.

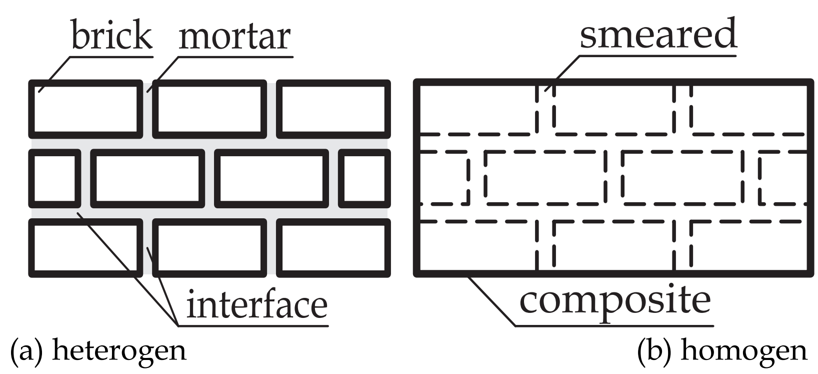


Fig. 2. Characterisation of masonry depending on the scale

2.2.3.1 Compressive strength

Compressive strength of masonry is much higher than its tensile or flexural strength. As a result masonry is mainly used for structures under compressive load and the bearing capacity of masonry is described generally by the parameter compressive strength f_k. Taking into account the different components of masonry, the strengths are different. The compressive strength of the bricks f_b is much higher than the compressive strength of the mortar f_m, therefore the failure mechanism of masonry under compressive load can be characterised.
Compressive strength of masonry can be seen as a function of the strength of the bricks and the strength of the mortar. The formula was detected empirically with the influences of the singular strengths on the overall strength of masonry and shows a nonlinear behaviour between the strengths of bricks and mortar. According to EC 6 the characteristic compressive strength of masonry f_k, which is defined as the 5 %-fractile value, can be determined from the mean values of the compressive strength of bricks and mortar. These mean values can be determined by execute tests according to EN 1052-1 or from the following equation:

$$f_k = K \cdot f_b^{\alpha} \cdot f_m^{\beta} \tag{1}$$

The factor K and the exponents α and β have to be taken from EC 6, e.g. for old bricks K = 0.60 the exponents α = 0.65 and β = 0.25. The relation of eq (1) is just valid for compressive loads perpendicular to the horizontal joints. If the compressive load is applied in the direction of horizontal joints, the strength has to be reduced as a result of the influence of the vertical joints. Researches from (Glitzka, 1988) quantify this reduction as follows:

$$f_{k||} = 0.75 f_k \tag{2}$$

If masonry is under compressive loads, deformations occur in parallel as well as in perpendicular direction in accordance to the load direction. As a result from the differences in material behaviour of bricks and mortar, horizontal stress occurs and causes failure of masonry. There are also differences in the lateral deformations of the masonry components, which can be observed, if masonry is loaded until the uniaxial compressive strength of mortar is almost reached. In this stress state the lateral elongation of mortar is much larger than those of the bricks, but the lateral deformations of the mortar are restricted by the bricks. Hence in the mortar occurs a spatial compressive stress state, whereas the brick is loaded by compression and tension.

The compressive strength of masonry under compressive loads is mostly determined by lateral tensile strength of bricks and the elastic modulus of mortar. However the elastic modulus can be identified in the tests quite difficulty. Therefore, in most applications the compressive strength of the bricks f_b is taken as the initial parameter for defining the overall compressive strength of masonry. Some correlation factors between lateral tensile strength f_{bt} and compressive strength of bricks are listed in the literature (Schubert & Brameshuber, 2011). For common bricks, this correlation is:

$$f_{bt} = 0.026 f_b \tag{3}$$

Tensile stresses inside of bricks cause cracks and fracture of bricks. To follow the progression of the cracks up to failure, the interaction phases between bricks and mortar have to be considered.

If masonry is put under compressive load and vertical cracks appear, the limit of lateral tensile strength has been exceeded, and on the appearing cracks, lateral tensile stress is reduced. If the load increases, the proximate cross-sections carry the lateral tensile stress up to an exceeding of the next maximum possible tensile stress state. After a few formations of cracks, the structure fails. The function of the critical stress state is depicted as the enveloping line of the fracture and shows the local reachable limit state. If masonry is under a marginal vertical compressive loading, the tensile stress state inside the bricks is below the enveloping curve of fracture. Therefore vertical stress can be increased, until the brick fails in tension or the mortar fails in compression.

The material performance of masonry under compressive load is defined in EC 6 by means of the parable-rectangle-diagram, see Fig. 3. The limit strain is defined with ε_{m1} and ε_{mu}.

Another important parameter for defining the material behaviour of masonry is the elastic modulus. According to EC 6 the short time elastic modulus can be determined as the secant modulus from tests according to EN 1052-1 or calculated directly from compressive strength:

$$E = 1000 f_k \tag{4}$$

From experimental tests, see also Sec. 2.3, for masonry made of solid bricks with the old-Austrian shape type (height 6.5-7.5 cm) independent from the mortar type, a deviation from the recommended values for the elastic modulus was found out.

$$E = 300 f_k \quad (5)$$

The reason for the lower elastic modulus for masonry made from solid bricks can be seen in the higher percentage of horizontal joints per altitude compared to masonry made out of new honeycomb bricks. Commonly used honeycomb bricks have a height of 25 cm and therefore just 4 horizontal joints per meter altitude difference in relation to 13 joints (masonry made of solid bricks), which is a multiplying factor of 3.3 for the joints. The same ratio can be found in the correlation factors for elastic modulus and compressive strength.

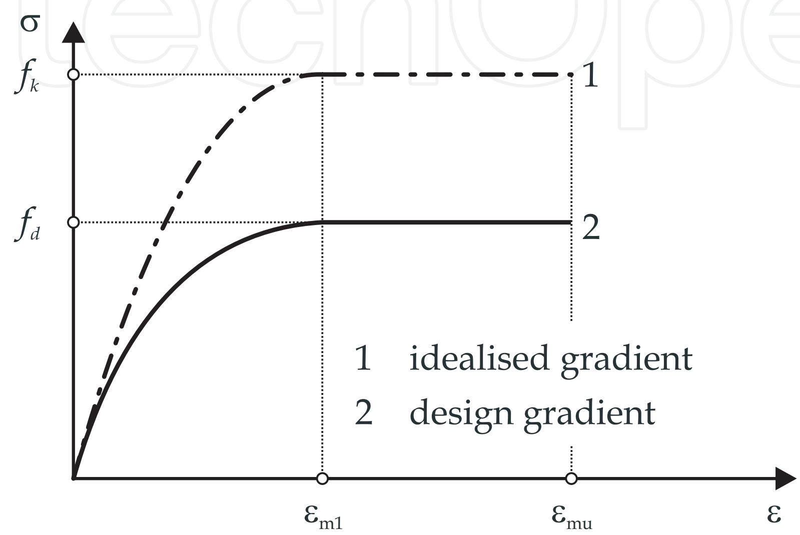


Fig. 3. Stress-Strain-Curve of masonry under compressive load

2.2.3.2 Tensile strength

Generally in masonry walls there is no constant tensile load over the whole cross section perpendicular to the horizontal joint. Besides the dead load, in most cases, masonry has to bear vertical loads. Anyway if there occur a tensile load, two mechanisms of failure can be distinguished: (a) failure of bond between mortar and brick, which is influenced mainly by tensile bond strength f_{hz} between these components, and (b) failure of bricks, if tensile bond strength between mortar and bricks is larger than the tensile strength of bricks.

According to the codes of masonry construction, a planned tensile load perpendicular to the horizontal joints has to be avoided, because the resistance and capacity perpendicular to the horizontal joints has a large scatter.

EC 6 defines boundary conditions, when tensile strength perpendicular to the horizontal joints may be considered. In these cases, the failure of the structural member must not cause a failure of the whole structure. Tensile load in the direction of horizontal joints results from a load in the direction of the wall. To bear these tensile stresses, the walls have to be built in bond, whereat it is convenient to overpressure the occurring tensile stresses with perpendicular compressive stresses.

Tension bearing capacity of masonry in the parallel direction to the horizontal joints is defined mainly by the characteristics of the mortar. To define the deformation behaviour of masonry walls parallel and perpendicular to the horizontal joints extensive research is documented in (Bakes, 1983). However in this analysis the friction between the mortar of the vertical joint and the brick was neglected. Results show until fracture an almost linear material behaviour. Fig. 4 shows different failure modes caused by tension.

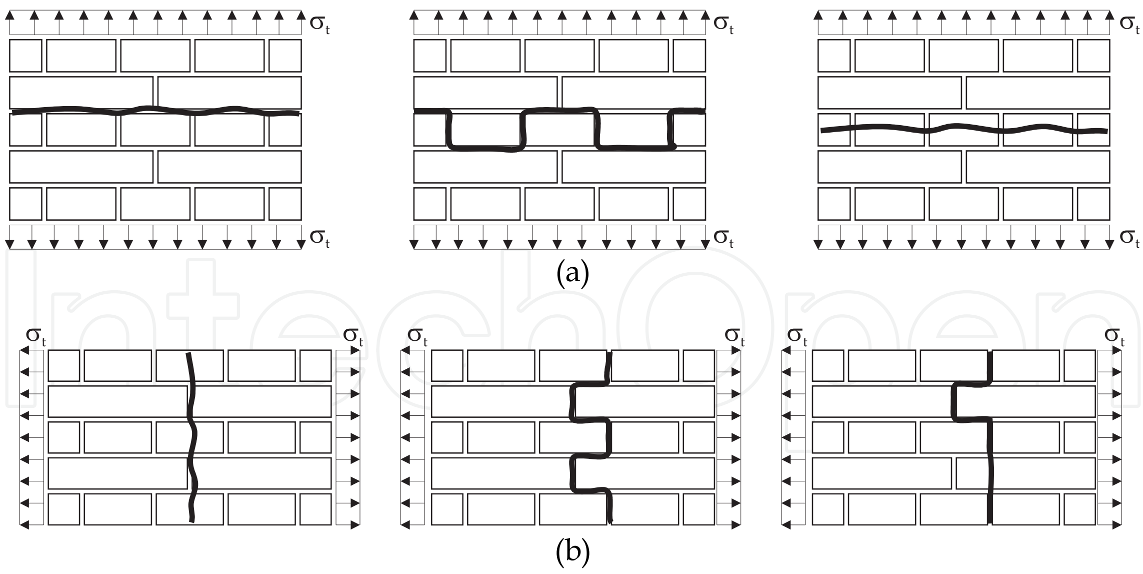


Fig. 4. Masonry under tension load, (a) tension perpendicular to horizontal interface (b) tension parallel to horizontal interface

2.2.3.3 Flexural behaviour

In contrast to the pure tensile load perpendicular to the horizontal joints, which can be excluded in nearly all load cases, flexural loading is a load case which is quite common. If a wall is loaded by wind or earth pressure perpendicular to its surface, then flexural stresses in the perpendicular and the parallel direction of the horizontal joints occur (Fig. 5). Designing principles assume no tensile or flexural stresses perpendicular to the horizontal joints. According to EC 6, the gap in the horizontal joints is just acceptable until to the half of the cross section.

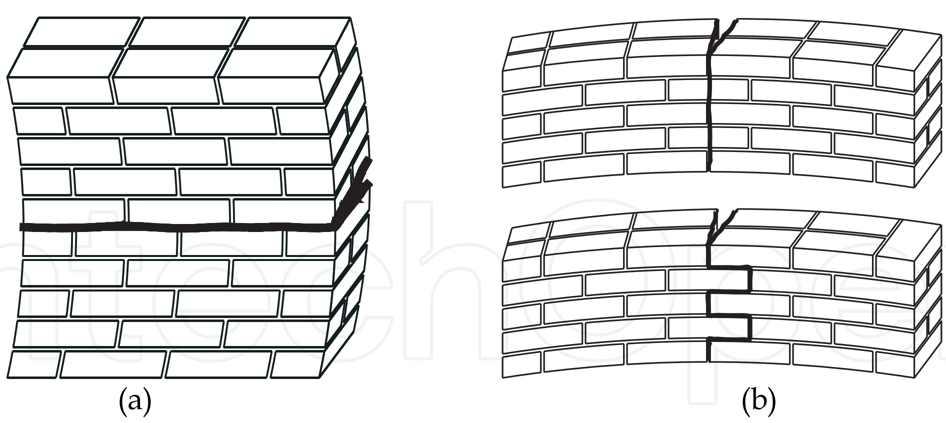


Fig. 5. Flexural load (a) parallel to the horizontal joints f_{xk1} and (b) perpendicular to the horizontal joints f_{xk2}

2.2.3.4 Shear strength

The global bearing behaviour of structures made of masonry is influenced by loads acting in horizontal direction like wind and earthquake loads. For the load transfer vertical shear walls are required, which are loaded with shear forces in the wall direction. Therefore the behaviour of the shear wall is decisive for the bearing capacity of the whole structure. If an element is cut from the shear wall normal stresses are acting in vertical direction and shear stresses are acting at all four edges. This theory assumes that the vertical joints between the bricks nearby the fracture state do not transfer shear stresses, and consequently the vertical joints are neglected because of the low value of the stresses. Additionally the shrinking process in the mortar reduces the bond between mortar interface and brick. Further due to the low compression state in vertical direction no significant friction forces can be developed in the vertical joints.

Through the combination of normal and shear forces in the direction of the wall, a two-axial loading is induced. A plane stress state develops in the direction of the shear wall. The theory of Mann & Müller is defined for a stretcher bond with an overlapping of the stretcher of a half length of the bricks and a ratio of width and length of the bricks of 1:2. The shear stresses inside the horizontal joints induce a torque, which is compensated by the equilibrium on every single brick by a pair of forces. Assuming a linear distribution of stresses over the half length of the brick the stress state can be calculated as follows:

$$\sigma_{x1,2} = \sigma_x \pm \tau \cdot \frac{Q_x}{\Delta y} \quad \text{with } Q_x = 2 \cdot \tau \cdot \Delta x \tag{6}$$

The fracture depends on the ratio of the different loads and the material parameters and can be distinguished into four different failure modes (Mann & Müller, 1978): (a) failure of masonry due to compression; (b) rocking (gap in the horizontal joints at the bottom part of the wall); (c) friction failure of the horizontal joints; (d) tension failure of the bricks.

Failure of masonry due to compression (line *a* in Fig. 6) appears, if the maximal compressive stress σ_{x1} becomes higher than the compressive strength f_k of masonry.

$$\tau = \left(f_k - \sigma_x\right) \cdot \frac{\Delta y}{2\Delta x} \tag{7}$$

Rocking (b) appears, if the minimal compressive stress σ_{x2} becomes zero. It is assumed that the horizontal joints cannot bear tensile stresses (eq. (8), b_1 Fig. 6). If a tensile strength is considered (f_{hz} = tensile bond strength), the fracture condition can be formulated as stated in eq. (9) (b_2 Fig. 6).

$$\tau = \sigma_x \cdot \frac{\Delta y}{2\Delta x} \tag{8}$$

$$\tau = \left(\sigma_x + f_{hz}\right) \cdot \frac{\Delta y}{2\Delta x} \tag{9}$$

Friction failure (line *c* in Fig. 6) appears, if in the area of the horizontal joints with minimal compressive stress the friction resistance is exceeded. The fracture condition can be defined with Mohr-Coulomb's law.

$$\tau = f_{vko} + \mu \cdot \sigma_x \tag{10}$$

The compressive stresses $\sigma_{x1,2}$ and the shear loads as shown in eq. (6) induce a principal stress state inside the bricks, which results in the fourth failure mode, tensile failure of bricks (*d* Fig. 6). The principal stress state induces the fracture of the bricks, and therefore the tensile strength of bricks f_{bt} becomes decisive.

$$\tau = \frac{f_{bt}}{2.3} \cdot \sqrt{1 + \frac{\sigma_x}{f_{bt}}} \tag{11}$$

The discussed failure modes can be pictured as one curve in the στ-diagram (Fig. 6). The curve encloses the area, in which no fracture and failure occurs. Stress states which are outside of the curve, lead to one of the four failure modes due to the discussed criteria.

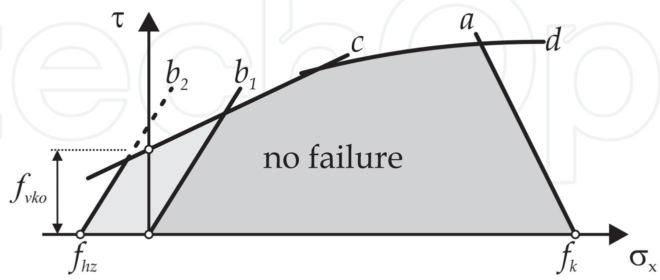


Fig. 6. Curve of the failure mode in the στ-diagram

2.3 Experimental testing

From the 1950ies up to now, extensive research has been done with respect to the correlation of the strengths of used materials and the bearing capacity of structures, whereas most of interest was spent on uniaxial compressive strength. The research led to different calculation models and during the last centuries to a harmonisation of the European codes and standardised design concepts.

For definition of the design parameters of bricks and mortar, testing values are required. Therefore it was a need to standardise the testing methods and applications. On the other hand, efforts were put on resistance against horizontal loads, i.e. shear or dynamic loads and mostly also in combination with bending. As a result of a higher sensitivity in respect to earthquake it was necessary to adapt the codes. This adaption causes quite complex calculation methods and the need of additional material parameters. Various testing methods were discussed for determining the needed material parameters. A simple approach, which has been included into the European codes, is due to basic parameters, i.e. the compressive strengths of bricks and mortar. For defining the horizontal resistance, this approach needs more information, which can be given by the coefficient of friction μ, the initial shear strength f_{vko} and the tensile strength of the bricks f_{bt}. Therefore standardised values are available, which can be taken for the calculation and design or derived from the basic parameters, correlation factors are listed in (Schubert & Brameshuber, 2011).

For existing structures, it is required to get information about the basic material parameters in a non-destructive way as possible. There are a few non-destructive testing methods available, but they do not offer a direct conclusion on the existing strengths. One possibility is the sampling of wall-like specimens. This gives a deep insight into the strength of the tested component. However, by sampling test specimens of the building the structural integrity may get disturbed and in the extreme case the stability against collapse decreases.

Moreover, there are remarkable costs and therefore in most of the cases the number of test samples is not sufficient enough for a detailed static assessment of the existing structure. Especially in older structures the scatter of material parameters are considerably high and then a large number of samples is required to make serious assessments.

2.3.1 Testing methods of static parameters

In-situ testing methods of mortar and bricks have been enhanced to avoid disturbing existing structures. Additionally, the results were calibrated on wall specimens and on model structures and there are lots of standardised testing methods for determining parameters like brick size, shape, density etc.

- testing of masonry compressive strength by direct methods:
  - testing in laboratory of adequate, new constructed masonry specimens according to EN 1052-1 – Determination of compressive strength.
  - in-situ testing on structures, e.g. Flat-Jack-Test (Pech & Zach, 2009).
  - sampling of test specimens and testing in laboratory, according to EN 1052-1 – Determination of compressive strength.
  - sampling of representative test specimens for laboratory testing
- testing of shear strength of masonry:
  - adequate testings on masonry specimens, no standardised methods available
  - definitions and specifications of storage, size of test specimens, load application, boundary conditions can be found in the Mauerwerk Kalender
- testing of flexural strength of masonry:
  - general:
    - different test setups in literature, no valid results, many influencing parameters cannot be evaluated yet
    - values are not necessarily required for the assessment of existing structures
  - test method according to EN 1052-2 – Determination of flexural strength:
    - flexural in the plane of the wall-panel
    - test setup for determining the flexural strength parallel and perpendicular to the horizontal joints
  - test method according to EN 1052-5 – Determination of bond strength by the bondwrench method:
    - for torque application e.g. at the top of the wall, pure torque load
    - there cannot be estimated a common relationship between the test results of tensile bond strength and shear strength
- testing of tension strength of masonry:
  - There is no standardised test method available. In the ESECMaSE-Projekt the "Direct tension Test" was proposed as test method.
- testing of compressive strength of the components (indirect testing of masonry) and calculation of the overall compressive strength of masonry:
  - testing of strength of the bricks according to EN 772-1 – Determination of compressive strength. Compressive strength of the bricks is determined perpendicular to the horizontal joints, which is sufficient precise. The possibility of a reduced bearing capacity due to diagonal principal compressive stresses is neglected.
  - testing of strength on small sized test specimens
  - testing of strength by rebound hammer (Pech & Zach, 2009).
  - testing of mortar strength on a standardised prism, according to EN 1015-11 – Determination of flexural and compressive strength of hardened mortar
  - testing of mortar strength by stamp compression test (Pech & Zach, 2009).

    - testing of mortar strength by determination of penetration resistance and the related deformation
- testing of splitting tensile strength of bricks:
    - There is no standardised test method available. Testing can be performed analogous to concrete, where load application is done with stripes of a width of 10 mm. The test method can also be applied on drill cores.
- testing of flexural strength of bricks:
    - testing method according to RILEM-Recommendations TC 76 – flexural strength of units (de Vekey, 1988)
- testing of centric brick tensile strength:
    - There is no standardised test method available.
- testing of coefficient of friction and initial shear strength between brick and mortar:
    - method according to EN 1052-3 – Determination of initial shear strength.
- additional physical analysis on bricks:
    - EN 772-13 – Determination of net and gross dry density of masonry units.
    - EN 772-16 – Determination of dimensions.

For estimating the strength of the bricks and as basic value for the calculative determination of the masonry compressive strength from the values from the components (e.g. according to national part of EC 6) the bricks have to be tested according to EN 772. As an alternative for existing structures, the compressive strength can be determined in a non-destructive way by rebound and penetration methods.

Estimating of compressive strength of the bricks on existing structures has to follow EN 1998-3 for the required minimum number of testing samples. Existing structures are defined as those objects, which were built before the actual standards for masonry were valid, therefore these objects cannot ensure the required quality. The required amount of tests for a sufficient result of an existing structure with homogenous material and a knowledge class KL3 it is necessary to perform one test series per 1000 m² total floor area or two test serials per structure. For a knowledge class KL2 50 % of the required tests for KL3 have to be done. The definition of the knowledge classes is standardised in EN 1998-3 and depends on the geometry, constructional details and the materials. According to EN 1998-3 a test series is defined by the following parameters: (a) at least three test specimens (masonry) or (b) at least three test locations for testing strength of the components by taking specimens of the bricks and mortar for compressive strength tests or (c) at least six test locations for testing strength of the components by rebound and penetration methods.

After determining the test locations, the bricks usually are taken from masonry by means of a masonry saw or they are chiselled out. The test locations have to be documented. In order to minimize the size of the disturbed masonry, instead of the five to six full bricks according to EN 772-1, four to six half bricks can be taken instead. Although the size of the test specimens generally should not be decisive for compressive strength, especially the inhomogeneities in historical bricks can be the reason for high scatter and unsafe test results. For average determination of a test location there have to be taken five values, divergent from EN 772-1. In case of non-destructive testing with the rebound hammer there have to be taken 10 individual test results for each test location for determining the compressive strength.

In addition to performing the tests, they should be documented. The documentation should include the following parts: (a) object/structure; (b) date of testing; (c) situation of the test locations (identification on building plan); (d) testing method and standardisation to normative values; (e) characteristic masonry strength for each test location, test serial, type of masonry; (f) compressive strengths of bricks and mortar for each test location, test serial, type of masonry if the components were tested individually; (g) type of construction stones according to EN 771-1.

2.3.2 Testing methods of dynamic parameters

By investigation of the behaviour of masonry walls under cyclic load, essential information of the load-displacement-curve can be determined. The load-displacement-behaviour depends on the decisive failure mode. By means of pseudo-dynamic experiments, the dynamic behaviour of structures can be determined depending on time.

The experimental research of the cyclic behaviour of masonry walls has been investigated during the last years in lots of research projects, e.g. ESECMaSE and SEISMID. Usually, shear wall tests were performed, in which under constant vertical loads a cyclic horizontal load is applied in a quasi-static way. The defined boundary condition on the top of the wall is either a fixed support or a cantilever arm, which has the possibility of free rotation. This experimental setup allows the determination of special dynamic parameters, e.g. energy dissipation and hysteretic damping. The obtained test data can be used for further relevant parameters for seismic design concepts, like behaviour factor, stiffness and stiffness degradation. This topic is discussed in (Knox & Ingham, 2011; Tomazevic et al., 1996a; Zimmermann et al., 2010a; Zimmermann et al., 2010b). A further possibility for experimental testing is the performance of shake table tests on a vibrating table. In contrast to the shear walls discussed above, precise acceleration spectra can be taken for loading. The direct analysis of whole structures (walls, slabs, openings and floors) allows a better consideration of the load bearing behaviour. Experimental work is reported e.g. in (Benedetti et al., 1998; Tomazevic et al., 1996b; Tomazevic, 2007).

3. Seismic loads

Seismic loads are considered in EC 8. Part 1 specifies the basics, the loads from seismic impacts and structural design concepts in seismic influenced regions. The code specifications cover the design concepts by requirements on geometry, design by verification of the load bearing capacity and considerations for construction details. The other parts of EC 8 include specifications for bridges (2), existing structures (3), silos, tanks and pipelines (4), foundations and retaining walls (5) and towers, pylons and chimneys (6). Each structure has to be designed considering the unfavourable limit states, including and not including seismic loads. Therefore, depending on the used material, the corresponding EC is the basis for design, considering the specifications of EC 8.

3.1 Seismic zones

The exposure to seismic loads is specified by one single parameter, which is the reference-top level ground acceleration a_{gR} for foundation class A. In Austria, the value of this ground acceleration is from 0.18 up to 1.34 m/s². This reference-top level ground acceleration has to be multiplied with the coefficient of importance γ_I to obtain the ground acceleration a_g.

$$a_g = \gamma_I \cdot a_{gR} \tag{12}$$

The reference-top level ground acceleration at a coefficient of importance of $\gamma_I = 1.0$ corresponds to a reference exceeding probability of P_{NCR} = 10 % within 50 years or a reference recurrence period of T_{NCR} = 475 years.

3.2 Categories of importance

Failure of a structure, its impacts on human life, public safety and social an economic effects are defined by means of the categories of importance and the related coefficients of importance γ_I. In case of structural design, structures with a low importance for public safety are category I (e.g. structures with an agricultural using, γ_I = 0.8), common structures are category II, structures where gatherings have to be considered (e.g. school buildings) are category III and the most important structures are classified in category IV (e.g. hospitals) both categories have a range of γ_I = 1.0 to 1.4.

3.3 Foundation classes

Foundation has an applicable influence on the exposure of a structure to seismic loads. Foundation can be divided in classes A – E, S_1 and S_2, according to EC 8. The classification of the local foundation should be done considering the shear wave velocity v_{s30}, if the value is known; otherwise the classification should be done with the number of blows of the Standard-Penetration-Test, N_{SPT}. Additional investigations to the required static analysis are necessary, if the local foundation is classified to S_1 or S_2, or if the structure has the category of importance III or IV.

3.4 Response spectrum

The dynamic impact of an earthquake on a structure is generally characterised by a horizontal response spectrum. Thereby on the abscissa it is plotted the natural oscillation time T and on the ordinate the maximal amplitude of the response acceleration of a planar single degree of freedom, which has a constant natural oscillation time over the duration of the seismic impacts. The response spectrum is timely independent and depicts the smoothing and enclosing distribution of many earthquakes.

3.4.1 Horizontal-elastic response spectrum

The horizontal seismic impact can be described by means of two orthogonal components, which are independent from each other and can be characterised by the same response spectrum. The horizontal component of the elastic response spectrum $S_a(T)$ is defined at 5 % viscous damping by four groups. In general there are two modes of spectra, type 1 and type 2. Type 1 is assumed for larger magnitudes of surface waves, $M_S > 5.5$ and type 2 for smaller magnitudes, $M_S \leq 5.5$. The parameters of the elastic response spectrum are listed in EC 8. In Fig. 7 the characteristics of the response spectrum are depicted for the different foundation classes.

The elastic acceleration spectrum $S_a(T)$, depending on the settling time T, can directly be transformed into an elastic displacement response spectrum $S_{De}(T)$ by following condition:

$$S_{De}(T) = S_a(T) \cdot \left[\frac{T}{2\pi}\right]^2 \tag{13}$$

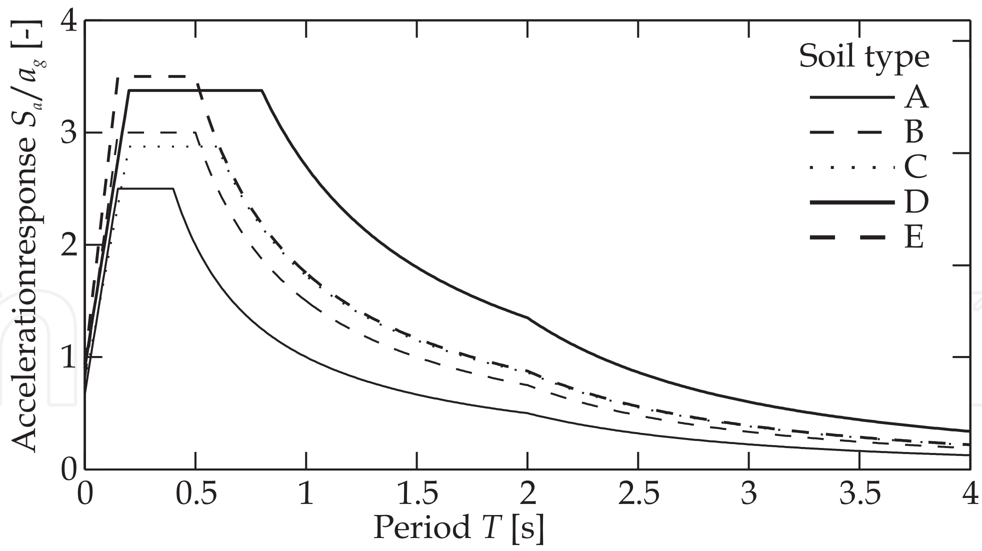


Fig. 7. Elastic response spectra of type 1 for foundation classes A – E with 5 % viscous damping

The correlation of spectral acceleration and displacement for a single degree of freedom with a predefined natural period is described by the ADR-spectrum (Acceleration Displacement Response), whereby the abscissa shows the time of displacement response $S_{De}(T)$ and the ordinate the acceleration response $S_a(T)$. For periods exceeding 4.0 s, the elastic acceleration response spectrum of type 1 can be obtained from the elastic displacement response spectrum by inverting eq. (13), according to EC 8, Annex A.

3.4.2 Vertical-elastic response spectrum

The vertical component of seismic impact can be neglected in Austria and is therefore listed only for ensuring completeness. It can be described by the vertical elastic response spectrum $S_{ve}(T)$. Just as the horizontal spectrum, the vertical spectrum can be divided in four groups, whereby the maximal values are considerably smaller. In general the dominant impact is the horizontal response spectrum.

3.4.3 Design value of ground displacement

The design value of ground displacement d_g can be estimated in accordance with the design value of the ground acceleration as follows:

$$d_g = 0.025 \cdot a_g \cdot S \cdot T_C \cdot T_D \tag{14}$$

3.4.4 Design spectrum

Observations of earthquake impacts have shown that structures can reduce seismic impacts by nonlinear reactions. To use the advantages of a linear calculation, the energy dissipation is considered by a reduced response spectrum (= design spectrum). The ratio of the estimated maximal exposure to the real appearing lower exposure is defined by the behaviour factor q. This coefficient is an approximation of the ratio of the seismic forces which would act on the structure if the response at 5 % viscous damping would be completely elastic and of those forces which can be used for a barely satisfactory linear design. Table 4 gives an overview of the behaviour factor q.

Material	Range	Relevant code
concrete	1.50 – 5.85	EC 8 Table 5.1
steel	≤ 1.50 – 6.50	EC 8 Table 6.1 resp. Table 6.2
bond between concrete and steel	≤ 1.50 – 6.50	EC 8 Table 6.2 resp. Table 7.2
wood	1.50 – 5.00	EC 8 Table 8.1
masonry	1.50 – 3.00	EC 8 Table 9.1

Table 4. Behaviour factor q according to EC 8

The four groups of the horizontal design spectrum can be defined as follows:

$$\begin{aligned}
&0 \le T \le T_B : S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right] \\
&T_B \le T \le T_C : S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \\
&T_C \le T \le T_D : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C}{T}\right] \\ \ge \beta \cdot a_g \quad with \ \beta = 0.2 \end{cases} \\
&T_D \le T \le 4s : S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[\frac{T_C T_D}{T^2}\right] \\ \ge \beta \cdot a_g \quad with \ \beta = 0.2 \end{cases}
\end{aligned} \tag{15}$$

in which $S_d(T)$ = elastic design response spectrum a_g = design ground acceleration according eq. (12), T = settling time, S = soil parameter, T_B, T_C, T_D = period, q = behaviour factor and β = lower bound of spectrum, recommended value is 0.2.
If in eq. (15) the vertical component a_{vg} is used instead of the design ground acceleration a_g and S = 1, all four groups of the vertical design spectrum can be defined. The coefficient of behaviour should not exceed 1.5 for masonry.

3.4.5 Alternative Interpretation of seismic impacts

Alternatively, seismic impacts can be considered by means of natural or simulated time dependent distributions of acceleration. In spatial models of a structure there have to be considered three simultaneous time dependent distributions of acceleration, whereby the same distribution may not be used for both horizontal directions. Simulated distributions of acceleration have to be generated in a way, that the response spectra describe the elastic response spectra of Sec. 3.4.1 and Sec. 3.4.2 for 5 % viscous damping. The duration of the distributions of acceleration has to be consistent to the characteristics of the earthquake, which refers to a_g and to the earthquake's magnitude. If there is no further information available, the minimum duration of the stationary part of the distribution of acceleration should be considered with 10 s. Moreover it should be used a minimum of three distributions of acceleration, whose average for the zero period yields at least to a value of $a_g{\cdot}S$ for the considered location. No ordinal value in the range of 0.2 T_1 up to 2.0 T_1 shall be smaller than 90 % of the corresponding value of the design spectrum, whereby T_1 is the natural period of the structure in the related direction.

3.5 Seismic design

Seismic loads are inertial loadings, which act on the mass points and they are the result of multiplying mass with acceleration. Hence the loading is not applied external on the structure, but is produced by ground movements and deformations in the structure itself. Therefore seismic loading depends on both the place of location and on the structure itself. In earthquakes prone areas, the seismic aspects have to be considered already in the conceptual state of designing structures, therewith both the requirements on structural safety and minimizing failure effects can be fulfilled with bearable costs, compare with EC 8 Part 2.1, (Bachmann, 2002a, 2002b).

3.5.1 General principles

Generally, structures have to be designed from the constructive aspect as easy as possible for ensuring a definite and direct load transfer, avoiding uncertainties in modelling and increasing the safety of the structure. In buildings, the floor slabs have a decisive significance. In the plane of the slab the inertial forces are bounded and transferred to the vertical members. Therefore the slabs should have a sufficient stiffness in their plane and the behaviour of horizontal panels. Particularly in case of mixed structures, large openings and changes of the stiffness, the behaviour of the panel and the interaction between horizontal and vertical structural members should be ensured.

For avoiding non-uniform torsional loading on bearing structural members it has to be ensured that stiffness is distributed preferably constant around the structure and a sufficient torsional stiffness is warranted. In addition to constructive aspects, an adequate foundation and connective elements to the superstructure are required for ensuring a homogenous seismic loading of the structure and the load transfer to the ground. Structures with load-bearing walls with diverging values of length and stiffness should have a box-shaped or cellular foundation; separate parts of the foundation should be connected with a ground slab or a flexible foundation beam.

3.5.2 Criteria of regularity

For purposes of seismic design of construction it has to be distinguished between regular and non-regular structures. This differentiation influences the calculation model, the calculation method and the behaviour factor, compare Table 5.

Regularity		Acceptable simplifications		Behaviour factor
ground plan	vertical section	model	linear-elastic calculation	
Yes	Yes	plane	simplified	reference value
Yes	No	plane	modal	reduced value
No	Yes	spatial*	simplified	reference value
No	No	spatial	modal	reduced value

* according EC 8 Part 4.3.3.1(8) under special circumstances an planar model can be used in each of the both directions

Table 5. Seismic design according to constructive regularity

A structure has to fulfil the following requirements that it can be classified as regularly in respect to the **ground plan**: The distribution of the horizontal stiffness and mass in accordance to two perpendicular axes should be symmetric. The shape of the ground plan has to be compact and should be enhanced by a polygon line, offsets and niches must not contain more than 5 % of the floor area. Comparing to the stiffness of the horizontal members, the slabs have to assure a sufficient stiffness in their plane to ensure the load transfer. Slenderness ratio λ has to fulfil $L_{max}/L_{min} \leq 4$, in which L_{max} and L_{min} are the maximum and minimum perpendicular dimensions of the structure.

For each floor and in each direction of calculation the effective excentricity has to be in *x*-direction $e_{0x} \leq 0.30\, r_x$ and $r_x \geq l_s$ and in *y*-direction $e_{0y} \leq 0.30\, r_y$ and $r_y \geq l_s$ respectively, in which e_o is the distance between centre of stiffness and centre of mass, *r* is the radius of torsion and l_s the radius of inertia of the floor mass. In case of one floor the radius of torsion is defined as the square root from the torsion stiffness in reference to the horizontal stiffness. The radius of inertia is defined as the square root of the polar moment of inertia in reference to the centre of mass. In case of several floors, the centre of stiffness and the radius of torsion can be determined just approximately. Simplifying it can be regarded as regular, if load-bearing structural members range from the foundation up to the top edge of the structure and if the bending lines of the stiffening systems under horizontal loading are not different. An approximate approach for the calculation is defined in the national part of EC 8, Annex B.

That a structure can be determined as regularly in respect to the **vertical section**, it has to conform to the requirements below. All horizontally operating stiffening systems have to range from the foundation up to the top edge of the structure, respectively up to the adequate height of structural members. Horizontal stiffness and the mass of the respective floors have to be constant over their height or decrease steadily from the bottom to the top. In case of frame structures, the ratio of the real strength of proximate floors to the required strength from the calculation should not diverge too much. If there are offsets, the conditions from Fig. 8 must be considered.

The offsets have to be designed symmetrically and may not exceed more than 20 % of the previous dimensions. In case of a single offset within the lower 15 % of the total height, the offset may not exceed 50 % of the ground plan dimension. Then the continuous part below should be able to bear at least 75 % of the total horizontal shear load. In the event of asymmetric offsets, in each vertical section the sum of offsets may not exceed 30 % of the dimensions of the ground plan and each offset may not be larger than 10 % of the previous dimension, see Fig. 8.

In general, the requirements of EC 8 with regard to regularity in ground plan and vertical section can be summarized, that a compact construction type with symmetric distributed mass and stiffness has a positive influence on the seismic loadbearing capacity, compare Fig. 9.

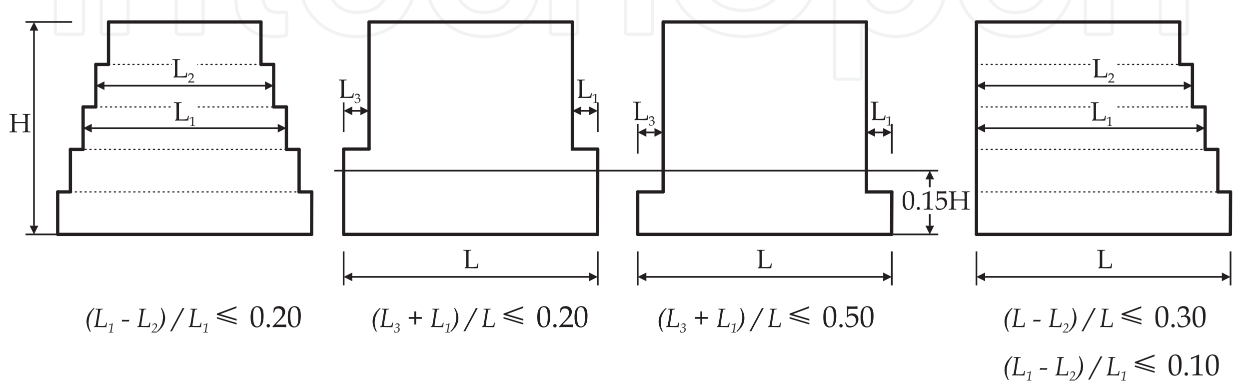
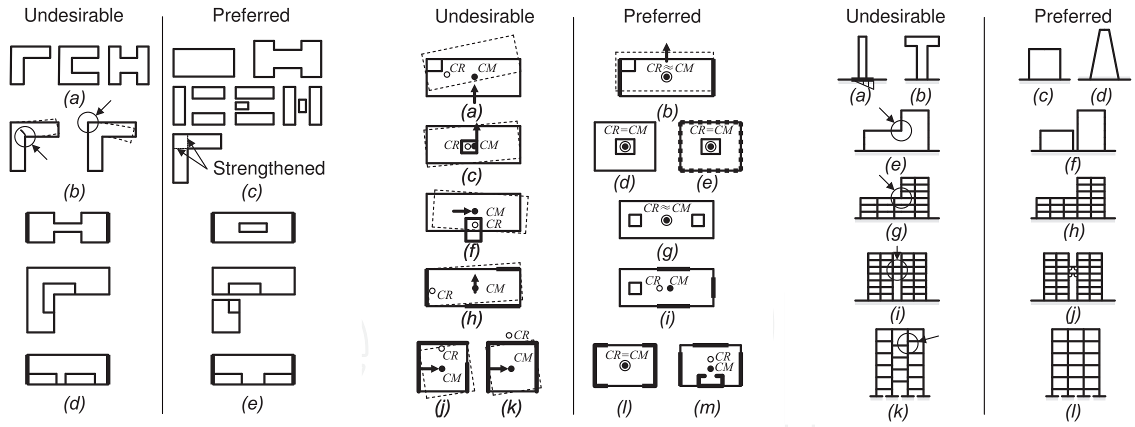


Fig. 8. Criteria of regularity of structures in reference to the vertical section, according to EC 8



CM=centre of mass, CR=centre of stiffness

(a) horizontal setup (b) distribution of mass and horizontal stiffness, (c) vertical setup

Fig. 9. Comparison of disadvantageous and favourable structural characteristics, from (Paulay & Priestley, 1996)

3.5.3 Coefficients of combination

In the seismic load case, variable loads have to be considered in a reduced way. Their value is defined by the factors of combination $\psi_{E,\iota} = \varphi.\psi_{2,\iota}$. Thereby $\psi_{2,i}$ is the value, which takes account of the quasi constant part, as recommended in Annex A1, EC 0. The factor φ defines the probability of a simultaneous occurrence of the variable loads in each floor. In Austria it is recommended to use φ = 1.0, or the values from Table 6.

The masses, which are required for calculating the inertial forces, which have to be applied in case of seismic loads, result from the dead loads from the permanent loads and the variable loads, which are reduced by the factor $\psi_{E,i}$.

$$\sum G_{k,i} + \sum \psi_{E,i} \cdot Q_{k,i} \tag{16}$$

Type of loading	Floor	φ
Category A – C according EC 1	housetop	1.0
	floors with an related usage	0.8
	independent usage of floors	0.5
Category D – F according EC 1 und archives	-	1.0

Table 6. Values for the calculation of $\psi_{E,i}$

3.5.4 Computation of structures

Modelling has to ensure, that the distribution of masses and stiffness is described correctly, in case of nonlinear analysis the distribution of strength, too. The structural model should also take account of the connecting zones to the deformation of the structure, e.g. ending zones of beams or columns. In case of structures made of concrete, reinforced concrete,

composite constructions or masonry, the stiffness of the load-bearing structural members should be determined by considering the formation of cracks. If no calculative analysis of the cracked members is available, approximately the half of the stiffness of the uncracked members can be assumed.
Bracing elements in masonry, which have an essential impact on the horizontal stiffness, should be considered. The ductility of the foundation has to be regarded, if it influences the model in a negative way and can optionally be regarded if there is a positive impact. The definitions of EC 8 recommend that the stiffness and the masses of the structure should be summarized to a substitute beam. Then the seismic loads are calculated for each floor and then redivided to the load-bearing structural members. This procedure presumes that the load distribution takes place by floor slabs with an adequate stiffness.
Alternatively, the seismic loading can also be calculated directly at a spatial model, if the recommendations from EC 8 are used analogously. This model even allows the calculation of structures, which do not have a sufficient panel effect of the floor slabs. In Table 7 the calculation methods for the load case seismic loads are summarized.

	Force method	Response spectrum	Pushover	Timehistory
Method	static	static	static	dynamic
Model	linear plane	linear plane, spatial	nonlinear plane, spatial	nonlinear plane, spatial
Torsion	simplified approach	plane: simplified approach, spatial: in model	plane: simplified approach, spatial: in model	plane: simplified approach, spatial: in model
Considering nonlinearity	global by coefficient of behaviour	global by coefficient of behaviour	in model	in model
Load	response spectrum	response spectrum	response spectrum	time response
Calculation	analysis with static resultant forces	modal analysis with quadratic superposition of state variables	Pushover-calculation, continuous increasing external forces	at least three time response calculations with static analysis
Uncertainties	modelling, dynamic of the structure, material behaviour	modelling, dynamic of the structure, material behaviour	modelling, dynamic of the structure	modelling
Regularity	very high	plane: high spatial: none	plane: high spatial: none	plane: high spatial: none
Traceability	very easy	easy	easy	difficult
Utilisation of bearing reserves	low	low	good	very good
Computational effort	low	middle	high	very high

Table 7. Overview about the calculation methods for seismic loads (Zimmermann & Strauss, 2010b)

3.5.5 Resultant force method

The resultant force method or the simplified response spectrum method can be applied, if the structure complies with the requirements of regularity in vertical section from Sec. 3.5.2 and if the natural period T_1 in each main direction is lower than 4 T_C and 2 s respectively. Then it can be assumed, that the higher mode shapes have no influence on the total seismic load and that they can be neglected. The total seismic force F_b for each main direction is calculated by:

$$F_b = S_d\left(T_1\right) \cdot m \cdot \lambda \tag{17}$$

in which $S_d(T_1)$ = ordinate of the design spectrum, m = total mass of the structure and λ = correction coefficient for the participation of the mass. For more than two floors and $T_1 \leq 2\,T_C$ it is recommended λ = 0.85, otherwise λ = 1.0. The natural period T_1 can be determined by an approximation procedure, e.g. the energy method (Flesch, 1993). For buildings up to a height of 40 m, the following approach can be used:

$$T_1 = C_t \cdot H^{3/4} \tag{18}$$

in which C_t = 0.085 for flexural resistant steel frameworks, C_t = 0.075 for frameworks made of reinforced concrete and C_t = 0.050 for all other structures; H = height of the structure.
Alternatively, in case of structures with shear walls made of concrete or masonry, the value for C_t can be defined as follows:

$$C_t = 0.075 \,/ \left(\sqrt{\sum \left[A_i \cdot \left(0.2 + \left(l_{w,i} \,/\, H \right)^2 \right) \right]} \right) \tag{19}$$

in which A_i = effective cross section of the shear wall i and $l_{w,i}$ = length of the shear wall i, under the condition of $l_{w,i}/H \leq 0.9$.
The distribution of the horizontal seismic loads is based on the mode shapes or can be assumed as a triangular distributed load over the height, if the horizontal displacement of the eigenmode is approximated to be linear over the height.

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \qquad F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \tag{20}$$

Thereby F_i = the applying horizontal load on floor i, s_i, s_j = displacement of the masses m_i, m_j and z_i, z_j = height of the masses. If the horizontal seismic loads are calculated as loads of the floors, the assumption of load transfer by rigid floor panels has to be fulfilled. In case of the separate structural members, the additional loading resulting from accidentally torsion load has to be considered with a coefficient $\delta = 1 + 0.6(x/L_e)$, which is a multiplying factor for the seismic load. Thereby x = distance from the structural member from centre of mass and L_e = distance between the outside structural members perpendicular to the considered direction of seismic impact.

3.5.6 Multimodal response spectrum

If the criteria of regularity in respect of vertical section are not fulfilled and other modal shapes than the natural eigenmode are decisive, the multimodal response spectrum method has to be applied instead of the simplified response spectrum method, see Sec. 3.5.2. This method can be used for each type of structure. In this dynamic calculation method the whole structure is divided into individual single degree of freedoms and the reaction under applied dynamic load is identified for each single degree of freedom i with a natural period T_i. This reaction can be determined from the design spectrum, the shear force $F_{b,i}$ is then:

$$F_{b,i} = S_d\left(T_i\right) \cdot m_{i,eff} \tag{21}$$

in which $S_d(T_i)$ = ordinate of the design spectrum for the natural period T_i and $m_{i,eff}$ = effective modal mass of the i-th eigenmode.

All decisive mode shapes have to be considered, which have a significant impact on the structural response. An eigenmode is decisive, if the sum of the effective modal mass is at least 90 % of the total mass. In addition, no eigenmode may be neglected, which modal mass has more than 5 % of the total mass. If these requirements cannot be fulfilled, the number k of the modal inputs which have to be taken into account should be at least $k \geq 3\ n^{0.5}$; thereby n = number of floors.

The period of the last eigenmode T_k which should be considered may not exceed 0.2 s. For each eigenmode i the maximal loading values can be determined. As a result, the singular modal parts can be added to the reaction of the whole structure. If all decisive mode shapes can be assumed as independent from each other, the maximal value of the seismic loading follows with the SRSS-formula (Square Root of Sum of Squares):

$$E_E = \sqrt{\sum E_{E,i}} \tag{22}$$

in which E_E = seismic load value and $E_{E,i}$ = seismic load value of the i-th eigenmode. Mode shapes are oscillating independently from each other, if the difference between the singular eigenfrequencies is large enough. This is fulfilled, if for two consecutive periods i and j $T_j \leq 0.9\ T_i$. If this condition is not reached, other methods for combination have to be applied, e.g. the CQC-method (Complete Quadratic Combination). Thereby the load value is:

$$E_E = \sqrt{\sum\sum E_{E,i} \cdot \rho_{ij} \cdot E_{E,j}} \tag{23}$$

in which ρ_{ij} = factor of interaction. The factor of interaction takes account of the modal damping in reference of the mode shapes i and j and the ratio of the circular eigenfrequencies ω_i and ω_j. This method is documented e.g. in (Clough & Penzien, 1995; Flesch, 1993). Torsion loadings at spatial models can be incorporated by additional torsional moments $M_{a,i}$ around the vertical axis of each floor i:

$$M_{a,i} = e_{a,i} \cdot F_i \qquad with\ e_{a,i} = \pm 0.05 \cdot L_i \tag{24}$$

in which F_i = horizontal force in floor i, $e_{a,i}$ = accidental excentricity of floor mass i and L_i = floor dimension perpendicular to the direction of seismic load.

3.5.7 Nonlinear static method (Pushover)

Alternatively to response spectrum methods, nonlinear methods, e.g. the nonlinear static pushover method can be applied (Chopra & Goel, 1995). The characterisation of the material behaviour has to be done with a bi-linear force-deformation-relation. For concrete and masonry, the linear-elastic stiffness of the bi-linear relationship should coincide with that from cracked cross sections. For ductile members the secant stiffness to the yield point should be taken for the bi-linear relation. After the yield point, the tangential stiffness can be appropriated. In case of brittle materials, the tangential stiffness of the force-deformation-relation should be considered. If there are no further specifications, the material characteristics should be based on average values. For new structures, the material parameters can be taken from the codes EC 2 – EC 6 or from other appropriate European standards.

At nonlinear static calculation, the horizontal loads are increased monotonous under constant dead loads and the gained load-deformation-curves of the singular load-bearing structural members are superposed. As a result the capacity curve of the structure is achieved. This calculation method can be used for both determining the bearing capacity of existing and of new structures (Chopra, 2002; Clough & Penzien, 1995). Instead of the behaviour factor q, which incorporates the energy dissipation and the nonlinear effects in a global way, the real nonlinear material behaviour is considered. Depending on the criteria of regularity, in the calculation either two plane models for each of the both horizontal main directions are set up or a spatial model is used. In case of low masonry structures (≤ 3 floors), whose load bearing walls are loaded mainly by shear loads, each floor can be considered singularly. For distributing the horizontal loads two approaches should be applied, on the one hand a modal, and on the other hand a mass-proportional distribution of the horizontal loads, referring to eq (20).

The horizontal loads have to be applied in in the centres of mass, whereby accidental excentricities according eq (24) have to be considered. From the nonlinear static calculation, the curve of capacity of the structure has to be defined in a range of 0 – 150 % of the aimed displacement, whereby the control displacement of the capacity curve can be assumed in the centre of mass of the top of the structure. The aimed displacement is determined by the displacement of an equivalent single degree of freedom. The method for determining the aimed displacement is regulated in EC 8, Annex B.

3.5.8 Nonlinear dynamic calculation (Timehistory)

Seismic loads can also be determined by means of simulated or measured time responses of the ground acceleration. Solving eq. (25) yields to the variation in time of the responded oscillations of the system in the considered degrees of freedom (Chopra, 2002; Clough & Penzien, 1995).

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=\{f(t)\} \tag{25}$$

Thereby $\ddot{x}$ = acceleration, $\dot{x}$ = velocity and x = displacement vector, $f(t)$ = load vector, **M** = mass matrix, **C** = damping matrix and **K** = stiffness matrix. The time responses of all decisive parameters have to be quantified separately, because the maximum values of the displacement parameters $x_j(t)$ do not appear at the same time. The structural response oscillation depends on the characteristics of the applied variation of time and therefore at least three time responses have to be considered. For solving the differential equation system in eq. (25), (a) the modal method for linear systems, or (b) the direct integration method for linear and nonlinear systems can be used.

By means of the **modal method** the linked differential equation system is decoupled by transformation of the variables. As a result for linear elastic systems the displacements can be described as a linear combination of the mode shapes. Solving the decoupled differential equation yields to the time response of the i-th modal response oscillation. An advantage in

contrast to the response spectrum method is that the maximal response can be determined more accurately and in terms of the direct integration the calculation effort decreases. The main disadvantage is that only linear material behaviour can be incorporated.
The method of the **direct integration** solves eq. (25) directly with the aid of a numeric integration. As a result of this, variable constitutive equations and damping mechanisms (apart from Rayleigh-damping) can be regarded. The main disadvantage of this method is the huge calculation effort, and the need of adequate numerical models for the description of the nonlinear material behaviour under cyclic loading.
The scope of the two described time-history methods is primarily in the assessment of existing structures. Further information is given in (Bachmann, 2002b; Chopra, 2002; Clough & Penzien, 1995; Flesch, 1993).

4. Damage quantification

The quantification of damage is an important task for evaluating the condition of the structure and the degradation over time caused by loads and/or environmental impacts. Damage indexes can be used for structural assessment and further such indexes can be used for decision making of repair and of demolition respectively. Additionally different cost factors can be considered for the decision process and life cycle assessment of structures (Frangopol et al., 2009; Strauss et al. 2010; Strauss et al., 2008).
Damage indexes are mathematical models for a quantitative assessment and they are of substantial importance to estimate critical condition states of structures. The calculation of damage indexes and different studies can be found in the literature, e.g. (Fajfar, 1992; Cosenza et al., 1993; Moustafa, 2011).
Damage indexes can also be correlated with experimental test results. If a structure is subjected to repeated pseudo-dynamic load reversals the increasing degree of damage can be described by damage indexes (Tomazevic, 1998).

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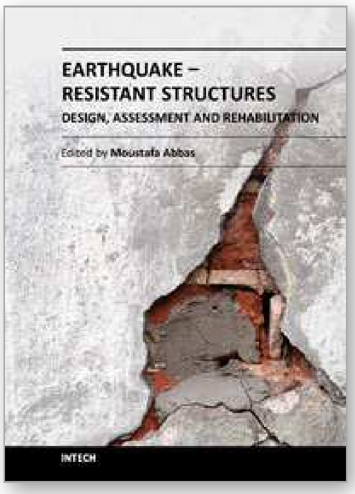

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This book deals with earthquake-resistant structures, such as, buildings, bridges and liquid storage tanks. It contains twenty chapters covering several interesting research topics written by researchers and experts in the field of earthquake engineering. The book covers seismic-resistance design of masonry and reinforced concrete structures to be constructed as well as safety assessment, strengthening and rehabilitation of existing structures against earthquake loads. It also includes three chapters on electromagnetic sensing techniques for health assessment of structures, post earthquake assessment of steel buildings in fire environment and response of underground pipes to blast loads. The book provides the state-of-the-art on recent progress in earthquake-resistant structures. It should be useful to graduate students, researchers and practicing structural engineers.