Structural Design for Juwo SmartWall Systems

Simplified calculation methods

Ver 2_2-3-24

Foreword

The Eurocodes for the calculation, design, construction and execution of masonry have now been in use for many years and there is extensive experience in practice which shows that efficient verification is possible with these regulations - especially when using the simplified calculation methods according to DIN EN 1996-3/NA. For the application of the European regulations in Germany, the corresponding national annexes must also be observed, in which determine supplementary specifications and boundary conditions for the use of the regulations. In December 2019, the National Annexes to EN 1996-1-1 and EN 1996-3 were updated and the application conditions for simplified verification were significantly extended. In addition, the design of basement masonry using an earth pressure coefficient freely selectable by structural designers is now also possible without the need for a more precise verification of the bending and shear force stress. These are only two examples of how the daily work of structural engineers can be facilitated by practice-oriented standardisation work. The design of masonry in conventional building construction will therefore continue to be simple and efficient in the future. This is a goal to which the "standardisers" in masonry construction always feel committed.

This brochure is a valuable aid for engineers working in practice to quickly familiarise themselves with the design of unreinforced brick masonry according to the simplified calculation methods taking into account the latest normative specifications. Examples are used to explain how efficient design of brick masonry can be carried out under a wide variety of load situations. In addition, the verifications required for the design in case of fire according to DIN EN 1996-1-2/NA are presented and supplementary information on the execution of masonry according to DIN EN 1996-2/NA is given. This publication takes into account the current state of the art for the calculation and design of unreinforced masonry for the majority of applications occurring in practice.

Darmstadt, January 2021

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Calculation of strength of JUWO SmartWall masonry for use in the UK

Calculation of strength of JUWO SmartWall masonry for use in the UK – A supplementary preface:

Summary:

Although hollow clay brick masonry has been in use in Europe for many decades, it has not been traditional in the UK and is only recently making market headway, partly due to more demanding regulations for thermal performance and certain economies.

However, the prevailing design standards in the UK and in the relevant parts of Europe are the identical (with their respective National Application Documents) and the norms, principals and recommendations in the prevailing standards can be used in the UK, as well as anywhere else.

The designer may then choose to comply strictly with the British BS EN 1996-1-1 (or BS EN 1996-3, the simplified version) or may use the German DIN EN 1996-1-1 (or DIN EN 1996-3, the simplified version) for the calculation of masonry strengths.

Use of the UK versions leads to greater conservatism (erring on the side of safety) than does use of the German versions.

Scope:

This document summarises the methods for determining the strength or resistance of JUWO SmartWall masonry for design of masonry structures in the UK.

This assumes the following:

- The designer is a competent person, such as an appropriately trained and experienced technician or engineer.
- That the responsibility for the design rests with the designer who must be competent to construe and appropriately apply this document and all relevant standards and best practices.
- That the designer will base their design on BS EN 1996-1-1 or BS EN 1996-1-3 and its national annexes.

The recommendations herein are for thin-joint masonry using JUWO SmartWall clay bricks or blocks, with unfilled perpends, but perpends closely abutting. Some recommendations may apply to localised use of other mortar applications. The designer is responsible for choice of relevant application.

All JUWO SmartWall masonry units are grouped according to Table 3.1 of BS EN 1996-1-1, and all units currently supplied are Group 2 or Group 3.

This is intended as a summary guide, and supplement to the German documents, for reconciliation of various national approaches to the determination of some key strength properties of JUWO SmartWall masonry. It must be read in conjunction with the appropriate design standards.

Calculation of **compressive** strength of JUWO SmartWall masonry for use in the UK

Symbols:

K	Coefficient applied in Eq. 3.1 of BS EN 1996-1-1.
f_b	Mean compressive strength (stress) of masonry unit.
f_d	Design compressive resistance (stress) of masonry element.
f_k	Characteristic compressive resistance (stress) of masonry element.
γ_M	Partial factor on material resistance.
α	Exponent applied in Eq. 3.1 of BS EN 1996-1-1.
ζ	Duration factor (used in German application only).
σ_0	Obsolete. Used in earlier German standards and may appear in some declarations.

History:

The JUWO SmartWall products were developed in Germany and have been used for many decades. The applicable standards, when the products were relatively new, preceded the widespread introduction of limit-state standards in Europe (for masonry) and the German building laws approvals were based on them. During development of the Eurocode program, the German industry foresaw that the older standards did not answer to modern developments so well as they wished, yet it was supposed that the new Eurocodes would not be available for some time to come. Therefore, the industry unilaterally developed interim standards, and masonry design (including for Poroton type masonry). The building law approvals were formulated to take this into account. Eventually EN 1996 was published, and now the German industry may use this, as do other countries, so that DIN EN 1996 became an approved standard. A broadly similar pattern occurred in the UK.

However, national choice is allowed, to reflect local practice and building customs, and to avoid abrupt changes in practice and resulting design. In Germany and the UK, the relevant National Application documents (NA) were published. The German NA prescribes the calculation of characteristic compressive strength, f_k, based on compressive strength tests on ***wall samples***, in accordance with EN 1052-1. The UK NA allows (and it is practically the universal UK method) the calculation of f_k from compressive strength tests on ***masonry units***, in accordance with EN 772-1.

<u>The results obtained from each national approach are not the same, in respect of the characteristic strength.</u>

Comparison:

Note that the products comply with BS EN 771-1 and are suitable for use in structural and non-structural masonry, type (P), in the UK, according to BS EN 1996-1-1.

Although partial factors on the loading side are the same in Germany and the UK (among others), the partial factors on materials differ markedly. The UK has much higher partial material factors, leading to more pessimistic (i.e. conservative) design stresses. By contrast, the German material factors are lower, but there is also a duration factor, which applies a further and small degree of conservatism for normal loading duration.

The various partial factors vary in each country according to different criteria. The national choice of factors is largely influenced by history and tradition, and a sense of continuity from previous practice. There is understandable reluctance for a nation to suddenly experience heavier construction or, on the other hand, unease about perceived safety margins, if design strengths were to change abruptly and significantly when new standards are published.

In the UK, these factors are differentiated between two quality control categories, so far as design and execution is concerned. In Germany they are differentiated between normal and special structural usage. The following is a comparison of the material factors:

	Germany	UK
Control Category 1		2.3
Control Category 2		2.7
Normal Construction	1.5	
Special Construction	1.3	

Under the most usual conditions, one would be comparing partial factors of 2.3 with 1.5. The implication, on the face of it, is that UK design stresses would be some third more pessimistic (conservative) than the German ones. If the effect of the German duration factor is included, the discrepancy reduces to about a quarter more pessimistic.

One might imagine, from this, that the UK NA is not yet well adapted to this type of masonry, but that would be an oversimplification as there are many influences on these factors. The most significant factors may include the differences in how the characteristic strength is derived.

In the case of all products, the characteristic strength derived from wall tests is lower than that calculated based on the mean strength. So, if the compressive strengths are derived from wall tests, and the partial material factors for the UK are applied, then absurdly conservative design stresses are obtained.

There are three approaches, among others perhaps, that may be used to reconcile this situation:

1. One may use the German standard, DIN EN 1996-1-1 (or DIN EN 1996-1-3) and the declared values for f_k to obtain the design strength. To avoid over-conservatism, one may use the German partial factors. An objection to this might be that the German standard may not technically and legally meet the 'deemed to satisfy' requirements of the UK Building Regulations. So, one must apply the concept of 'reasonableness' and, arguably NCCI.

2. One may choose to adhere strictly to UK standards and apply BS EN 1996-1-1 and the UK NA thereto, in their entirety, together with the masonry unit strengths. This would then comply in the sense that UK 'deemed to satisfy' documents are used throughout. However, it would overlook the effects captured in the wall tests – an effect compensated by the higher UK partial factors.

3. One may choose to follow BS EN 1996-1-1 as in '2' above but use the f_k values from the wall tests. This would be compliant in the UK, but very conservative design stresses would be obtained, because the partial factors are so high. We do not advocate the building of walls perhaps approaching 50% thicker than their German counterparts.

To evaluate the effect of one choice over another, a comparison was made of the design strength resulting from the first two of the methods mentioned above.

In the case of all products the design stress obtained from method 2 was more conservative than that obtained from method 1. The range of conservatism was from approximately 1% to 47% - that is, always on the ‘safe’ side (expressed as a proportion of the ‘German’ design stress). Very few margins were less than 5% (still on the safe side). Most engineers would consider anything less than 5% as marginal, in such materials and their applications.

This observation is based on all the products being of Group 2 or 3 (see the relevant product declaration). The grouping was based on consideration of the voided area criteria and the web and shell thickness criteria, according to Table 3.1 of BS EN 1996-1-1.

Recommendation:

Therefore, we recommend that, always subject to the designer's discretion, the following approach is adopted (See also the simple procedures below):

- **Calculate the design strength using the UK standard, using mean masonry unit strengths, thence the derived f_k value, and UK partial factors.**

  Then, if additional confidence is desired…

- **Calculate the design strength based on the German standard, using the declared f_k value and the German partial factors.**
- **If the second verification was carried out, choose the worst from the two.**

Do not use the so-called 'shape factor' or the normalised strength enhancement. It is considered inappropriate to apply this concept to hollow blocks/bricks, which are profoundly isotropic compared to solid masonry units. The effects of stress distribution in solid materials, compared to voided material, and the effects of platen restraint in the testing apparatus must disqualify this procedure.

Beware of cases where manufacturers and suppliers do not draw attention to these issues. It is known, for example, that some manufacturers recommend the use of shape factor and normalised strength, and the characteristic strength taken from mean unit strength.

Procedures in detail:

Procedure 1 – To obtain the design compressive resistance, f_d, from the British standard:

i. Obtain the mean masonry unit strength f_b, either:
   - from the manufacturer's declaration
     …or…
   - from the declared strength class, using the following table:

Class	Mean (min.)
2	2.5
4	5
6	7.5
8	10
10	12.5
12	15
16	20
20	25
28	35
36	45
48	60
60	75

ii. Obtain the unit group from the manufacturer's declaration or geometric calculation.

iii. Select appropriate parameters K, α (For thin bed masonry, β = 0 since the mortar is irrelevant).
- For group 2 units: K = 0.7, α = 0.7.
- For group 3 units: K = 0.5, α = 0.7. Note that this is taken from the non-national document as NCCI because the UK NA does not give a value for Group 3.

iv. Obtain f_k from from Eq. 3.1 to 3.4 of BS EN 1996-1-1:

$$f_k = K.f_b^{\alpha}$$

Obtain f_d from Cl. 2.4.1(1) of BS EN 1996-1-1, using $\gamma_M = 2.3$, for Category I control, or $\gamma_M = 2.7$, for Category II control:

$$f_d = f_k/\gamma_M$$

NOTE BS EN 1996-1-1 Cl.NA.2.4 :

"When the perpend joints are unfilled, equation 3.1 may be used, with consideration of any horizontal actions that might be applied to, or be transmitted by, the masonry. See also 3.6.2(4)."

Procedure 2 – Optional – To obtain design compressive resistance, f_d, from the German standard and DIBt approvals:

i. Obtain f_k from the manufacturer's declaration.
ii. Multiply f_k by $\zeta = 0.85$ and divide by $\gamma_M = 1.5$ (for normal structural use) to obtain f_d.

$$f_d = \zeta.f_k/\gamma_M$$

Calculation of **shear** strength of JUWO SmartWall masonry for use in the UK

Symbols:

f_b	Mean compressive strength of the masonry units, as described in 3.1.2.1, for the direction of application of the load on the test specimens being perpendicular to the bed face. It is not recommended to use the normalised strength.
f_{vko}	Characteristic initial shear strength, under zero compressive stress in the direction considered.
f_{vlt}	Limit to f_{vk}.
f_{vk}	Characteristic shear strength in the direction considered.
f_{vd}	Design shear strength in the direction considered.
σ_d	Design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination based on the average vertical stress over the compressed part of the wall that is providing shear resistance.

Procedure:

The procedure adopted is that of BS EN 1996-1-1 and UK NA in their entirety. All procedures are to be followed at the designer's discretion.

The characteristic initial shear strength f_{vko} should be taken from Table NA.5. This results in:

$$f_{vko} = 0.3\text{MPa}$$

Use Cl. 3.6.2 (4) to obtain the characteristic shear resistance. The characteristic shear resistance is then the lesser of:

$$f_{vk} = 0.5\ f_{vk0} + 0.4\ \sigma_d \text{ (Eq. 3.6)}$$

…and…

$$f_{vk} = f_{vlt} = 0.045\ f_b$$

The design shear resistance is given by:

$$f_{vd} = f_{vk}/\gamma_{MV}$$

$$\text{Where } \gamma_{Mv} = 2.5$$

Calculation of **flexural** strength of JUWO SmartWall masonry for use in the UK

Symbols:

f_b	Mean compressive strength of the masonry units, as described in 3.1.2.1, for the direction of application of the load on the test specimens being perpendicular to the bed face. It is not recommended to use the normalised strength.
f_m	Compressive strength of the mortar.
f_{xk1}	Characteristic flexural resistance (stress) where the plane of failure is parallel to the bed joints.
f_{xk2}	Characteristic flexural resistance (stress) where the plane of failure is perpendicular to the bed joints.
f_{xd1}	Design flexural resistance (stress) where the plane of failure is parallel to the bed joints, in the absence of vertical stress.
f_{xd2}	Design flexural resistance (stress) where the plane of failure is perpendicular to the bed joints.
$f_{xd1,app}$	Design flexural resistance (stress) where the plane of failure is parallel to the bed joints, enhanced by the presence of vertical stress.
σ_d	Design compressive stress on the wall, not taken to be greater than 0.15 N_{Rd} in the middle of the wall according to 6.1.2.1(2).

History:

The British Standard BS EN 1996-1-1 does not recognise the use of unfilled perpends when considering flexure. This is because the use of unfilled perpends is not traditional in the UK. Inquiries of the UK drafting committee confirmed this to be an oversight that may be addressed in future editions.

The use of unfilled perpends elsewhere in Europe is widespread and traditional. It is considered to be more thermally efficient and labour efficient.

For this reason, the British version of EN 1996 can be used, but judiciously. Alternatively, the German version or the non-national version of EN 1996 could be used as NCCI.

Use of the British Standard:

It is always preferable to use data from testing of walls similar to that intended for use in the project.

In cases where the designer considers that this is not possible or appropriate to use direct testing, then BS EN 1996-1-1 may be used, but it provides data based on biaxial bending of wall panels – ***with filled perpends***. The unfilled perpends will affect the flexural strength in a non-conservative (unsafe side) way.

To resolve this issue, in a conservative (safe) manner, it is suggested that one of two approaches is used:

1. The flexural strength for failure perpendicular to the bed joints is ignored so that, in effect, the masonry element becomes uniaxial in flexure.

f_{kx1} = 0.4MPa, f_{kx2} = 0MPa

2. The masonry element may be treated as a gravity structure and the equilibrium verified under flexure and vertical load, assuming no tension, and using the compressive strength only. This method is also the only method that is recommended for any type of masonry retaining structures.

In the above, the NA to BS EN 1996-1-1 recommends that thin joint masonry is treated as though the mortar strength is M12.

Use of the non-national Standard:

It is always preferable to use data from testing of walls similar to that intended for use in the project.

In cases where the designer considers that this is not possible or appropriate to use direct testing, then BS EN 1996-1-1 may be used. The non-national version of EN 1996 encompasses general European use and is deemed to include for unfilled perpends.

1. f_{kx1} = 0.15MPa, f_{kx2} = 0.15MPa

1. The masonry element may be treated as a gravity structure as described in '2' for the British Standard, above.

In all versions of the standard:

$$f_{xd1} = f_{kX1}/\gamma_M$$

$$f_{xd2} = f_{kX2}/\gamma_M$$

$$f_{xd1,app} = f_{kX1}/\gamma_M + \sigma_d$$

Where γ_M = 2.3 for Category I control or 2.7 for Category II control, assuming removal of the panel **would affect** the stability of the building

…OR..

γ_M = 2.0 for Category I control or 2.4 for Category II control, assuming removal of the panel **would not affect** the stability of the building.

Beware of cases where manufacturers and suppliers do not clearly specify or define the conditions and format of the tests used to determine flexural strength. Some may test the product with plaster or parging, which will enhance the strength. The designer may adopt such strengths at their discretion.

1 The essentials at a glance

DIN EN 1996 (Eurocode 6) is the design standard for masonry in Germany introduced by the building authorities. For the national annexes of parts EN 1996-1-1 and EN 1996-3, two amendment sheets A1 and A2 have been published since 2012. Amendment A3 was processed in 2018 / 2019 and the national annexes were published as consolidated versions in December 2019.

This brochure provides a compact summary of the most important regulations of DIN EN 1996-3 including the corresponding National Annex DIN EN 1996-3/NA.

The following essential points must be observed when applying DIN EN 1996:

- DIN EN 1996-3 contains simplified calculation methods with which the vast majority of tasks relevant to building practice can be successfully dealt with.
- The design is carried out according to the partial safety factor concept with differentiated safety factors for effects and resistance.
- The verification is carried out in the ultimate limit state at the design value level with characteristic strength values.
- In the compressive strength verification, an explicit distinction is made with regard to the performance of brick types (e.g. solid brick, perforated brick) and brick types (e.g. brick, lightweight concrete).
- With the consolidated version December 2019 (amendment A3), interior wall plane bricks according to DIN EN 771-1 in conjunction with DIN 20000-401 were included in DIN EN 1996-3. Heat-insulating plane bricks continue to be regulated in general building inspectorate approvals (abZ) or general type approvals (aBG). These approvals / type approvals refer in principle to normative regulations, but may also contain specifications that extend or restrict the normative regulations.
- With the amendment A3, the application of DIN EN 1996-3/NA with higher maximum wall heights becomes possible.
- A mathematical verification of the shear force load-bearing capacity is not necessary for obviously sufficiently braced buildings when applying the simplified calculation methods according to DIN EN 1996-3/NA.
- For single-shell exterior walls, the partial support of the slabs (support depth a < wall thickness t) is explicitly taken into account in the verification.
- For exterior walls that serve as end supports for ceilings or roofs and are subjected to wind loads, a verification of the minimum wall load must be carried out. This verification is usually only relevant in wind zones III and IV for building heights of more than 10 m above ground level.
- The simplified verification of masonry basement walls can be carried out up to a fill height of 115 % of the clear basement height, e.g. to enable a barrier-free exit. Boundary conditions are defined for the sealing of the working space.
- If the design is carried out according to parts 1-1 and 2 of DIN EN 1996, the serviceability does not have to be proven separately.
- Fire protection is verified in accordance with DIN EN 1996-1-2/NA, which is dealt with in section 7.

2 Introduction

With the issue date of December 2010, DIN has published the German version of Eurocode 6 "Design of masonry structures" with the following parts

- DIN EN 1996-1-1: General rules for reinforced and unreinforced masonry [1].
- DIN EN 1996-2: Planning, selection of building materials and execution of masonry [2].
- DIN EN 1996-3: Simplified calculation methods for unreinforced masonry structures [3].

published.

Part 1-1 [2] was corrected and published again with the issue date February 2013.

In April 2011, for the fire protection dimensioning

- DIN EN 1996-1-2: General rules - Structural design for fire [4].

published.

The Eurocodes allow a number of safety-related parameters to be defined nationally. These Nationally Determined Parameters (NDP) include alternative verification methods and specifications of individual values as well as the choice of classes from given classification systems. The corresponding parameters as well as supplementary, non-contradictory information on the application of the Eurocodes are contained in the so-called "National Annexes" to the individual parts of the Eurocodes. They must also be taken into account when designing and dimensioning masonry.

The publication of the National Annexes to Parts 1-1, 2 [5] and 3 took place in January 2012. The National Annex to Part 1-2 [6] appeared in June 2013. The National Annexes to Parts 1-1 [7] and 3 [8] were published as a consolidated version in December 2019 following a third amendment.

DIN EN 1996-3 with its "simplified calculation methods" was included in Eurocode 6, particularly at the request of Germany. This is intended to ensure, in accordance with the simplified procedure according to DIN 1053-1 [9] that has proven itself in Germany, that the structural verification of a large part of all problems occurring in masonry construction is possible within a very short time and without great effort, even when applying the Eurocode.

In this brochure, the most important regulations of the "Simplified calculation methods for unreinforced masonry structures" according to DIN EN 1996-3 in connection with the associated National Annex [8] are presented and supplemented with simple numerical examples. The design of brick masonry in case of fire according to DIN EN 1996-1-2 is also briefly presented.

For the sake of clarity, the designation of which regulations are based on the Eurocode, the National Annex or the amendments is omitted and the wording DIN EN 1996-3/NA is always used.

The brochure concludes with a structural analysis of a multi-storey residential building, which shows that such buildings can be easily realised with monolithic brickwork.

3 Safety concept and verification procedure

3.1 General

The design of building structures according to the Eurocode is carried out across all building materials on the basis of the semi-probabilistic partial safety factor concept. Whereas in DIN 1053-1 uncertainties (scatter of the actions and the load-bearing resistance) were covered by a global safety factor, usually on the resistance side, the Eurocodes work with different partial safety factors on the action and resistance side. The size of the individual partial safety factors is determined depending on the scatter of the respective actions and resistances.

The stability is verified in the ultimate limit state (GdT) by comparing the internal forces that act and the internal forces that can be absorbed. The measured values of action (E_d) and resistance (R_d) result from the respective characteristic quantities (E_k) and (R_k), taking into account the corresponding partial safety factors γ.

The rated value of the resistance R_d must be at least as high as the rated value of the actions E_d **:**

$$E_k \cdot \gamma_F = E_d \leq R_d = \frac{R_k}{\gamma_M} \qquad (1)$$

DIN EN 1990:2010, section 6.4.2, eq. (6.8)

with:

E_k Characteristic value of the impacts
γ_F Partial safety factor for the actions, see Table 1
E_d Design value of the action
R_d Rated value of the resistance
R_k Characteristic value of the resistance
γ_M Partial safety factor of the resistance (or of the material), see table 3

The serviceability of components and structures, which must also be ensured in addition to stability, can be considered fulfilled in masonry construction without further verification if the verification in the ultimate limit state has been carried out using the simplified calculation methods according to DIN EN 1996-3/NA and the execution rules have been complied with.

3.2 Design value of the action E_d

When determining the design values of the impacts (E_d), a distinction must be made between two design situations:

Permanent and temporary design situation:

$$E_d = E\left\{\sum_{j\geq1} \gamma G_{,j} \cdot Gk_{,j} \oplus \gamma Q_{,1}\, Qk_{,1} \oplus \sum_{i>1} \gamma Q_{,i} \cdot \Psi 0_{,i} \cdot Qk_{,i}\right\} \qquad (2)$$

DIN EN 1990:2010, clause 6.4.3.2, eq. (6.9b) and eq. (6.10)

Simplified (lying on the safe side with $\psi_{0,i}$ = 1.0):

$$E_d = E\left\{\sum_{j\geq1} \gamma_G \cdot G_{k,j} \oplus \sum_{i\geq1} \gamma_Q \cdot Q_{k,i}\right\} \qquad (3)$$

Exceptional design situation:

$$E_d = E\left\{\sum_{j\geq1} \gamma G_{,j} \cdot Gk_{,j} \oplus A_d \oplus \Psi 1_{,1}\, Qk_{,1} \oplus \sum_{i>1} \Psi 2_{,i} \cdot Qk_{,i}\right\} \qquad (4)$$

DIN EN 1990:2010, clause 6.4.3.3, eq. (6.11a) and eq. (6.11b)

with:

$\gamma_{G,j}$ Partial safety factors of permanent actions according to Table 1
$G_{k,j}$ Characteristic values of permanent actions according to DIN EN 1991/NA
$\gamma_{Q,1}$ Partial safety factors of the variable conductive action according to Table 1
$Q_{k,1}$ Characteristic values of the variable conductivity according to DIN EN 1991/NA
$\gamma_{Q,i}$ Partial safety factors of the other variable actions according to Table 1
$Q_{k,i}$ Characteristic values of the other variable actions according to DIN EN 1991/NA
E_{dA} Design value taking into account the extra-ordinary action
A_d Design value of the extraordinary action according to DIN EN 1991/NA
$\psi_{0,i}$, $\psi_{1,i}$, $\psi_{2,i}$ Combination coefficients according to Table 2
$\oplus$ "to be combined with": the acting loads must be combined in the most unfavourable way; favourably acting, variable loads are to be neglected

Other simplified combination rules are permitted (see section 5.2).

Table	1Important partial safety factors γ_F of the actions for the verification in the ultimate limit state from DIN EN 1990/NA		
Impact	**unfavorable effect**	**beneficial effect**	**exceptional assessment situation**
permanent action (G) z. e.g. dead weight, extension load, earth pressure	γ_G = 1.35	γ_G = 1.0	γ_{GA} = 1.0
variable action (Q) z. e.g. wind, snow, payloads	γ_Q = 1.5	γ_Q = 0	γ_{QA} = 1.0

Table 2	Combination coefficients according to DIN EN 1990/NA (Table NA.A.1.1)		
Impact	ψ_0	ψ_1	ψ_2
Live loads in building construction, categories see DIN EN 1991-1-1 I Living, recreation and office spaces I Assembly rooms, sales rooms I Storage rooms	0,7 0,7 1,0	0,5 0,7 0,9	0,3 0,6 0,8
Snow and ice loads, see DIN EN 1991-1-3 I Locations up to NN + 1000 m I Places above sea level + 1000 m	0,5 0,7	0,2 0,5	0,0 0,2
Wind loads, see DIN EN 1991-1-4	0,6	0,2	0,0
Temperature (not fire), see DIN EN 1991-1-5	0,6	0,5	0,0

Table 3	Partial safety factor γ_M for the material (DIN EN 1996-3/NA, Table NA.1)	
	Permanent and temporary design situations	**Exceptional design situations**
Unreinforced masonry	1,5	1,3

3.3 Rated value of the resistance R_d

The design value of the resistance R_d is determined from the characteristic values of the resistance R_k , the partial safety factors γ_M (according to Table 3) and the factor for taking into account strength-reducing long-term influences ζ:

$$R_d = \zeta \cdot \frac{R_k}{\gamma_M} \qquad (5)$$

DIN EN 1990:2010, section 6.3.5, eq. (6.6c)

with:

ζ Coefficient to take into account strength-reducing long-term influences on the masonry, in general ζ = 0.85; for short-term stresses (e.g. due to wind, earthquake, fire) ζ = 1.0 may be set.

3.4 Detection method

The verification of masonry components can be carried out according to DIN EN 1996 using a more precise procedure (DIN EN 1996-1-1: General rules) or using a simplified procedure (DIN EN 1996-3: Simplified calculation methods).

For common building components made of brick masonry
the simplified calculation methods according to
DIN EN 1996-3/NA is usually completely sufficient.
The increased burden of proof of the general
rules is only in rare cases in more economical
constructions can be implemented. However, there is no
Prohibition of mixing, so that individual components of a
building is perfectly compatible with the general rules of
DIN EN 1996-1-1/NA can be verified.

This brochure mainly deals with the simplified calculation methods. If a building is obviously braced (see section 6.1), a shear force check in the slab or panel direction is not necessary if the associated boundary conditions are observed (see section 4). Therefore, DIN EN 1996-3/NA does not contain any regulations in this regard. If a mathematical verification of the building bracing is required, reference is made to the general rules according to DIN EN 1996-1-1/NA.

3.5 Prohibition of mixing with DIN 1053-1

The design rules of DIN EN 1996/NA may not be combined with the design rules of DIN 1053-1 within a structure. The masonry design must therefore be carried out for all components within a structure either according to the global safety concept or according to the partial safety concept.

3.6 Brick masonry according to general building inspectorate approvals (abZ) / general building type approvals (aBG)

The vast majority of brick structures continue to be designed and constructed in accordance with general building inspectorate approvals (abZ) or general building type approvals (aBG). These approvals refer in principle to normative regulations, but may also contain specifications that extend or restrict the normative regulations.

The validity of approvals or their contents is independent of the introduction or withdrawal of the standards mentioned in the approvals by the building authorities.

4 Prerequisites for the application of the simplified calculation methods of DIN EN 1996-3/NA

When using the simplified calculation methods, certain stresses, e.g.:

- Bending moments from slab restraint or support
- Unintentional outcentres during buckling detection
- Wind on load-bearing walls

do not have to be verified, as they are taken into account in the safety margin on which the verification procedure is based or by constructive rules. In principle, it is assumed that only bending moments from the slab restraint or support and from wind loads occur in the wall.

Due to the simplifications mentioned, the application of the simplified calculation methods is only permissible under certain boundary conditions. If one of these requirements is not met, a more precise calculation using the rules of Part 1-1 is mandatory. The necessary boundary conditions are shown in Table 4.

In addition, it should be noted:

- Building height above ground $h \leq 20$ m (for pitched roofs the average of ridge and eaves height)
- Support span of the overlying slabs $l \leq 6.0$ m, unless the bending moments from the slab rotation angle are limited by constructive measures, e.g. centring (for two-axis spanned slabs, the shorter of the two support spans is to be used for l). With regard to the extension of the application limits for brick masonry for long-span slabs ($l > 6.0$ m), see [14].
- If the wall axes are offset due to a change in wall thickness, the cross-section of the thicker load-bearing wall circumscribes the cross-section of the thinner wall.
- The influence of the wind load perpendicular to the wall plane of load-bearing walls may be neglected if the conditions for applying the unified calculation methods are met and sufficient horizontal supports are available. Such supports include, for example, ceilings with a pane effect or statically verified ring beams at intervals of the permissible wall heights.
- The ceiling support depth a must be $a \geq t/2$, but at least 100 mm. For wall thickness t = 365 mm, the minimum support depth $a \geq 0.45 \cdot t$ is different.
- The overbinding dimension l_{ol} must be at least $0.4 \cdot h_u$ (h_u brick height) and be at least 45 mm.

Table 4 Application limits of the simplified procedure according to DIN EN 1996-3/NA (Table NA.2) for common brickwork

Component	Wall thickness t mm	Clear wall height h m	Payload $_{qk}$ [1] kN/m²	Wind load $_{wk}$ kN/m²
Load bearing interior walls	≥ 115 < 240	≤ 2,75 [2]	≤ 5,0	-
		≤ 3,60 [3]		-
	≥ 240	No limitation		-
Load-bearing exterior walls and double-shell house partition walls	≥ 115	≤ 2,75 [4]	≤ 3,0	No limitation
	≥ 175	≤ 2,75 [2]	≤ 5,0	No limitation
		≤ 3,00 [3]		≤ 1,25
		≤ 3,30 [5]		≤ 1,25
	≥ 240	≤ 12 t [2]		No limitation
		≤ 3,60 [3]		≤ 1,25
	≥ 300	≤ 12 t [2]		No limitation

[1] **including surcharge for non-load-bearing internal partition walls**

[2] **general**

[3] **applies to brick masonry with $_{fk}$ ≥ 3.5 N/mm2**

[4] **As a single-shell exterior wall only for single-storey garages and comparable structures that are not intended for the permanent residence of people. As load-bearing shell of double-shell exterior walls and for double-shell house partition walls up to a maximum of two full storeys plus developed attic storey; bracing transverse walls at a distance *b* ≤ 4.50 m or edge distance from an opening *b*' ≤ 2.0 m (compare DIN EN 1996-3/NA Table NA.2, footnote a and Figure NA.2).**

[5] **Applies to brick masonry with $_{fk}$ ≥ 4.7 N/mm2 (compare DIN EN 1996-3/NA Table NA.2, footnote g).**

General technical approvals (abZ) or general type approvals (aBG) can contain further regulations.

5 Verification of predominantly vertically loaded walls

5.1 General

The stability of walls under predominant (vertical) normal force loading is determined in accordance with DIN EN 1996-3/NA by comparing the existing normal force N_{Ed} with the maximum normal force that can be absorbed N_{Rd} :

$$N_{Ed} \leq N_{Rd} \tag{6}$$

DIN EN 1996-3:2010, clause 4.2.2.1, eq. (4.3)

with:

N_{Ed} Design value of the acting normal force
N_{Rd} Design value of the absorbable normal force

5.2 Design value of the acting normal force N_{Ed}

For standard residential and office buildings, the design value of the acting normal force may be determined in an even more simplified way than in equation (3):

$$N_{Ed} = 1.35 \cdot \sum N_{Gk} + 1.5 \cdot \sum N_{Qk} \tag{7}$$

DIN EN 1996-1-1/NA:2019, NCI to 2.4.2, eq. (NA.1)

with:

N_{Gk} Characteristic value of the acting normal force due to permanent loads (e.g. dead weight).
N_{Qk} Characteristic value of the acting normal force as a result of variable loads (e.g. payload).

In buildings with reinforced concrete ceilings and characteristic live loads (including partition wall surcharge) $q_k \leq 3.0$ kN/m², a simplified approach may be used:

$$N_{Ed} = 1.4 \cdot \sum (N_{Gk} + N\,)_{Qk} \tag{8}$$

DIN EN 1996-1-1/NA:2019, NCI to 2.4.2, eq. (NA.2)

For larger bending moments around the strong axis (e.g. windscreens), the load combination max M ⊕ min N must also be analysed:

$$\min N_{Ed} = 1.0 \cdot \sum N_{Gk} \tag{9}$$

DIN EN 1996-1-1/NA:2019, NCI to 2.4.2, eq. (NA.3)

$$\max M_{Ed} = 1.0 \cdot \sum M_{Gk} + 1.5 \cdot \sum M_{Qk} \tag{10}$$

5.3 Design value of the absorbable normalkrah N_{Rd}

The design value of the absorbable normal force N_{Rd} is determined under the assumption of rigid-plastic material behaviour with the help of a rectangular stress block whose centre of gravity coincides with the point of application of the load resultants. The reduction of the bearing load due to buckling and/or load eccentricities is carried out via the reduction coefficient Φ:

$$N_{Rd} = \Phi \cdot A \cdot f_d \tag{11}$$

DIN EN 1996-3:2010, clause 4.2.2.2, eq. (4.4)

with:

Φ Reduction coefficient $\Phi = \min(\Phi_1, \Phi_2)$, see section 5.4
A $= l \cdot t$ (gross cross-sectional area of the wall section to be verified)
f_d Design value of the compressive strength of masonry

$$f_d = \zeta \cdot \frac{f_k}{\gamma_M} \qquad (12)$$

DIN EN 1996-3/NA:2019, NCI to 4.2.2.2, (NA.2)

with:

f_k characteristic value of the compressive strength of masonry see table 5

γ_M Partial safety factor for material properties, see Table 3

ζ Coefficient to take into account strength-reducing long-term influences on the masonry, in general ζ = 0.85; for short-term stresses (e.g. due to wind, earthquake, fire) ζ = 1.0 may be set

For wall cross-sections < 0.1 m², the design compressive strength of the masonry f_d must be reduced by multiplying by a factor of 0.8.

Table Characteristic values of compressive strength f_k for brickwork 5From vertically perforated bricks HLzA, HLzB, HLzE[1)] and masonry units T1 according to DIN EN 771-1 [11] in connection with DIN 20000-401 [12] as well as PHLzB and PHLzE vertically perforated flat bricks in N/mm²

<table>
<tr><th rowspan="2">Brick strength class</th><th colspan="4">Standard masonry mortar [2)]</th><th colspan="2">Light masonry mortar [3)]</th><th rowspan="2">Thin bed mortar [4)]</th></tr>
<tr><th>M 2,5</th><th>M 5</th><th>M 10</th><th>M 20</th><th>LM 21</th><th>LM 36</th></tr>
<tr><td>4</td><td>2,1</td><td>2,4</td><td>2,9</td><td>-</td><td>1,6</td><td>2,2</td><td>-</td></tr>
<tr><td>6</td><td>2,7</td><td>3,1</td><td>3,7</td><td>-</td><td>2,2</td><td>2,9</td><td>3,1</td></tr>
<tr><td>8</td><td>3,1</td><td>3,9</td><td>4,4</td><td>-</td><td>2,5</td><td rowspan="7">3,3</td><td>3,7</td></tr>
<tr><td>10</td><td>3,5</td><td>4,5</td><td>5,0</td><td>5,6</td><td>2,8</td><td>4,2</td></tr>
<tr><td>12</td><td>3,9</td><td>5,0</td><td>5,6</td><td>6,3</td><td rowspan="5">3,0</td><td>4,7</td></tr>
<tr><td>16</td><td>4,6</td><td>5,9</td><td>6,6</td><td>7,4</td><td>5,5</td></tr>
<tr><td>20</td><td rowspan="3">5,3</td><td rowspan="3">6,7</td><td>7,5</td><td>8,4</td><td rowspan="3">6,3</td></tr>
<tr><td>28</td><td>9,2</td><td>10,3</td></tr>
<tr><td>36</td><td>10,6</td><td>11,9</td></tr>
</table>

1) Vertically perforated brick with perforation E (HLzE) only for compressive strength classes 8 to 20 and mortar classes M 5 and M 10

2) Compare DIN EN 1996-3/NA:2019; Table NA.D.1

3) Compare DIN EN 1996-3/NA:2019; Table NA.D.5

4) Compare DIN EN 1996-3/NA:2019; Table NA.D.10

5.4 Reduction coefficient Φ

5.4.1 Φ1 for load reduction at the wall head and wall foot due to the ceiling rotation angle at end supports

For ceilings between floors: for f_k < 1.8 N/mm2

$$\Phi_1 = \left(1.6 - \frac{l_f}{5}\right) \cdot \frac{a}{t} \le 0.9 \cdot \frac{a}{t} \qquad (13)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3; equation (NA.2).

for f_k ≥ 1.8 N/mm2

$$\Phi_1 = \left(1.6 - \frac{l_f}{6}\right) \cdot \frac{a}{t} \le 0.9 \cdot \frac{a}{t} \qquad (14)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3; equation (NA.1)

Thereby

f_k the characteristic value of the compressive strength of masonry

l_f the span of the adjacent storey slab in m, for biaxial spanned slabs with $0.5 \le l_1 / l_2 \le 2.0$ 0.85 times the shorter span may be used for l_f

a the ceiling support depth

t the thickness of the wall

For ceilings above the top floor, especially for roof ceilings with low superimposed loads, the following applies:

For single-axis tensioned ceilings

$$\Phi_1 = 0.333 \cdot \frac{a}{t} \qquad (15)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3; equation (NA.3)

for biaxially tensioned ceilings with $0.5 \le l_1 / l_2 \le 2.0$

$$\Phi_1 = 0.4 \cdot \frac{a}{t} \qquad (16)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3; addition to equation (NA.3)

If the load reduction due to the ceiling rotation angle is avoided by constructive measures, e.g. centring, the following applies irrespective of the ceiling support width:

With partially overlying ceiling tile

$$\Phi_1 = 0.9 \cdot \frac{a}{t} \qquad (17)$$

With full contact ceiling tile

$$\Phi_1 = 0.9 \qquad (18)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3; clause (NA.4)

As a constructive measure to limit the bending moments from the ceiling rotation angle in case of wide-span ceilings and low to medium superimposed load levels, it is recommended to insert a soft insert (so-called load-free strips, e.g. felt strips) at the wall head on the inner wall edge (compare also chapter 10.2).

Ceiling supports for monolithic brickwork are often designed with a support depth a of 2/3 to 4/5 of the masonry thickness t. According to DIN 4108 Supplement 2:2019-06 [20], greater ceiling support depths are permissible. For example, a solid floor with a bearing depth of a = 285 mm can rest on an exterior wall of thickness t = 365 mm. This corresponds to a ratio of a / t = 78 %. With this ceiling support depth, the value of the reduction coefficient Φ_1 increases to take into account the load reduction at the wall head and wall foot due to the ceiling rotation angle for end supports. This usually leads to an increase in the loads that can be absorbed N_{Rd} .

5.4.2 Φ2 for load reduction due to danger of buckling at half wall height

$$\Phi_2 = 0{,}85 \cdot \frac{a}{t} - 0{,}0011 \cdot \left(\frac{h_{ef}}{t}\right)^2 \qquad (19)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.3, (NA.5), eq. (NA.4)

with:

h_{ef} Kink length

In the case of solid slab ceilings or ribbed ceilings according to DIN EN 1992-1/NA with load-bearing beams supported on two sides, the restraint of the wall in the slabs may be taken into account by reducing the buckling length:

$$h_{ef} = \rho_2 \cdot h \qquad (20)$$

DIN EN 1996-3/NA:2019 clause NCI to 4.2.2.4; eq. (NA.5)

with:

ρ_2 Buckling length coefficient according to DIN EN 1996-3/NA:2019 section NCI to 4.2.2.4 (NA.8)
= 0.75 for wall thicknesses $t \leq 175$ mm
= 0.90 for wall thicknesses 175 mm $< t \leq 250$ mm
= 1.00 for wall thicknesses $t > 250$ mm

h Clear storey height

The slenderness $\frac{h_{ef}}{t}$ must not be greater than 27 (compare DIN EN 1996-3:2006 section 4.2.2.5).

If no more detailed considerations are made, the smaller value of Φ_1 and Φ_2 can be used for the design in accordance with DIN EN 1996-3/NA:2019, section NCI to 4.2.2.3 (NA.6) (see tables 6 to 8).

The following tables 6, 7 and 8 show the dimensional factors for the slenderness h_{ef} /t. Common slendernesses for monolithic exterior walls are between h_{ef}/t = 5 (2.50 m / 0.49 m) and 10 (3.60 m). / 0,365 m).

Table 6 Decisive reduction factor $\Phi = \min(\Phi_1, \Phi_2)$ for **single-axis** tensioned ceilings

<table>
<tr><th rowspan="3"></th><th colspan="8">$f_k \geq 1.8$ N/mm²</th><th colspan="4">$f_k < 1.8$ N/mm²</th></tr>
<tr><th colspan="4">a/t = 1.0</th><th colspan="8">a/t = 67 % (e.g. 245 mm / 365 mm)</th></tr>
<tr><th colspan="12">No reduction of slab support width permitted, as slabs are uniaxially tensioned</th></tr>
<tr><th>h_{ef}/t</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th></tr>
<tr><td>5,0</td><td rowspan="2">0,82</td><td rowspan="7">0,77</td><td rowspan="15">0,68</td><td rowspan="19">0,60</td><td>0,54</td><td rowspan="4">0,51</td><td rowspan="10">0,46</td><td rowspan="15">0,40</td><td rowspan="10">0,47</td><td rowspan="15">0,40</td><td rowspan="15">0,33</td><td rowspan="15">0,27</td></tr>
<tr><td>5,5</td><td>0,53</td></tr>
<tr><td>6,0</td><td>0,81</td><td>0,53</td></tr>
<tr><td>6,5</td><td>0,80</td><td>0,52</td></tr>
<tr><td>7,0</td><td>0,80</td><td colspan="2">0,51</td></tr>
<tr><td>7,5</td><td>0,79</td><td colspan="2" rowspan="2">0,50</td></tr>
<tr><td>8,0</td><td>0,78</td></tr>
<tr><td>8,5</td><td colspan="2">0,77</td><td colspan="2">0,49</td></tr>
<tr><td>9,0</td><td colspan="2">0,76</td><td colspan="2">0,48</td></tr>
<tr><td>9,5</td><td colspan="2">0,75</td><td colspan="2">0,47</td></tr>
<tr><td>10,0</td><td colspan="2">0,74</td><td colspan="3">0,46</td><td>0,46</td></tr>
<tr><td>10,5</td><td colspan="2">0,73</td><td colspan="3">0,45</td><td>0,45</td></tr>
<tr><td>11,0</td><td colspan="2">0,72</td><td colspan="3">0,43</td><td>0,43</td></tr>
<tr><td>11,5</td><td colspan="2">0,70</td><td colspan="3">0,42</td><td>0,42</td></tr>
<tr><td>12,0</td><td colspan="2">0,69</td><td colspan="3">0,41</td><td>0,41</td></tr>
<tr><td>12,5</td><td colspan="3">0,68</td><td colspan="8" rowspan="4">The verification of monolithic exterior walls with $h_{ef}/t > 12$ is carried out according to the general rules of DIN EN 1996-1-1/NA.</td></tr>
<tr><td>13,0</td><td colspan="3">0,66</td></tr>
<tr><td>13,5</td><td colspan="3">0,65</td></tr>
<tr><td>14,0</td><td colspan="3">0,63</td></tr>
</table>

For exterior walls on the top floor, especially under roof ceilings, $\Phi = 0.333 \cdot a/t$ always applies.

[1] Examples in each case taking into account the reduction of the buckling length ρ_2 according to DIN EN 1996-3/NA:2019-12, NCI to 4.2.2.4; clause (NA.8).

Table 7 Decisive reduction factor Φ = min (Φ_1 , Φ_2) for **biaxially** tensioned ceilings

<table>
<tr><th rowspan="3"></th><th colspan="8">$f_k \geq 1.8$ N/mm²</th><th colspan="4">$f_k < 1.8$ N/mm²</th></tr>
<tr><th colspan="4">a/t = 1.0</th><th colspan="8">a/t = 67 % (e.g. 245 mm / 365 mm)</th></tr>
<tr><th colspan="12">Ceiling support width min (l_1 , l_2) [m] 1)2)</th></tr>
<tr><th>h_{ef} /t</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th><th>4,5</th><th>5,0</th><th>5,5</th><th>6,0</th></tr>
<tr><td>**5,0**</td><td colspan="3" rowspan="2">0,82</td><td rowspan="9">0,75</td><td colspan="3">0,54</td><td rowspan="5">0,50</td><td>0,54</td><td rowspan="5">0,50</td><td rowspan="12">0,44</td><td rowspan="15">0,39</td></tr>
<tr><td>**5,5**</td><td colspan="3">0,53</td><td>0,53</td></tr>
<tr><td>**6,0**</td><td colspan="3">0,81</td><td colspan="3">0,53</td><td>0,53</td></tr>
<tr><td>**6,5**</td><td colspan="3">0,80</td><td colspan="3">0,52</td><td>0,52</td></tr>
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<tr><td>**7,5**</td><td colspan="3">0,79</td><td colspan="4" rowspan="2">0,50</td><td colspan="2" rowspan="2">0,50</td></tr>
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<tr><td>**8,5**</td><td colspan="3">0,77</td><td colspan="6">0,49</td></tr>
<tr><td>**9,0**</td><td colspan="3">0,76</td><td colspan="6">0,48</td></tr>
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<tr><td>**13,0**</td><td colspan="4">0,66</td></tr>
<tr><td>**13,5**</td><td colspan="4">0,65</td></tr>
<tr><td>**14,0**</td><td colspan="4">0,63</td></tr>
</table>

For external walls in the uppermost storey, especially under roof ceilings, the following applies for biaxially tensioned ceilings with 0.5 ≤ l_1 / l_2 ≤ 2.0: Φ_1 = 0.4 · a/t (compare DIN EN 1996-3/NA:2019-12, section NCI to 4.2.2.3; equation (NA.3)).

1) In the case of two-axis spanned slabs, 0.85 times the shorter span may be used for the slab support width lf if: 0.5 ≤ l_1 / l_2 ≤ 2.0 (compare DIN EN 1996-3/NA:2019-12, NCI to 4.2.2.3, section (NA.2)).

2) The relevant reduction factors Φ in the table were determined with l_f = 0.85 · min (l_1 , l_2).

Table 8 Decisive reduction factor Φ = min (Φ_1 , Φ_2) for **biaxially** tensioned ceilings

<table>
<tr><td rowspan="3"></td><td colspan="4">$f_k \geq 1.8$ N/mm²</td><td colspan="4">$f_k < 1.8$ N/mm²</td></tr>
<tr><td colspan="8">a/t = 78 % (e.g. 285 mm / 365 mm)</td></tr>
<tr><td colspan="8">Ceiling support width min (l_1 , l_2) [m] 1)2)</td></tr>
<tr><td>h_{ef}/t</td><td>4,5</td><td>5,0</td><td>5,5</td><td>6,0</td><td>4,5</td><td>5,0</td><td>5,5</td><td>6,0</td></tr>
<tr><td>5,0</td><td colspan="3">0,64</td><td rowspan="6">0,59</td><td>0,64</td><td rowspan="6">0,59</td><td rowspan="13">0,52</td><td rowspan="15">0,45</td></tr>
<tr><td>5,5</td><td colspan="3">0,63</td><td>0,63</td></tr>
<tr><td>6,0</td><td colspan="3">0,62</td><td>0,62</td></tr>
<tr><td>6,5</td><td colspan="3">0,62</td><td>0,62</td></tr>
<tr><td>7,0</td><td colspan="3">0,61</td><td>0,61</td></tr>
<tr><td>7,5</td><td colspan="3">0,60</td><td>0,60</td></tr>
<tr><td>8,0</td><td colspan="4">0,59</td><td colspan="2">0,59</td></tr>
<tr><td>8,5</td><td colspan="6">0,58</td></tr>
<tr><td>9,0</td><td colspan="6">0,57</td></tr>
<tr><td>9,5</td><td colspan="6">0,56</td></tr>
<tr><td>10,0</td><td colspan="6">0,55</td></tr>
<tr><td>10,5</td><td colspan="6">0,54</td></tr>
<tr><td>11,0</td><td colspan="6">0,53</td></tr>
<tr><td>11,5</td><td colspan="7">0,52</td></tr>
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<tr><td>12,5</td><td colspan="8" rowspan="4">The verification of monolithic exterior walls with $h_{ef}/t > 12$ is carried out according to the general rules of DIN EN 1996-1-1/NA.</td></tr>
<tr><td>13,0</td></tr>
<tr><td>13,5</td></tr>
<tr><td>14,0</td></tr>
</table>

For external walls in the uppermost storey, especially under roof ceilings, the following applies for biaxially tensioned ceilings with 0.5 ≤ l_1 / l_2 ≤ 2.0: Φ_1 = 0.4 · a/t (compare DIN EN 1996-3/NA:2019-12, section NCI to 4.2.2.3; equation (NA.3)).

1) In the case of two-axis spanned slabs, 0.85 times the shorter span may be used for the slab support width lf if: 0.5 ≤ l_1 , l_2 ≤ 2.0 (compare DIN EN 1996-3/NA:2019-12, NCI to 4.2.2.3, clause (NA.2)).

2) The relevant reduction factors Φ in the table were determined with l_f = 0.85 · min (l_1 , l)$_{2}$

5.5 Highly simplified verification according to DIN EN 1996-3/NA, Annex A

Alternatively, DIN EN 1996-3/NA, Annex A offers an even further simplified possibility to determine the design value of the absorbable normal force N_{Rd} of buildings made of unreinforced masonry with a maximum of three storeys.

The additional or more stringent application requirements for this type of determination of N_{Rd} compared to those listed in section 4 are as follows:

- Maximum three storeys above ground
- smallest building dimension in the ground plan is at least 1/3 of the building height
- Slenderness $h_{ef}/t \leq 21$
- Clear storey height $h \leq 3.0$ m
- Wall thickness $t \geq 365$ mm, if $a/t < 1$
- Ceiling support depth $a \geq 2/3 \cdot t$

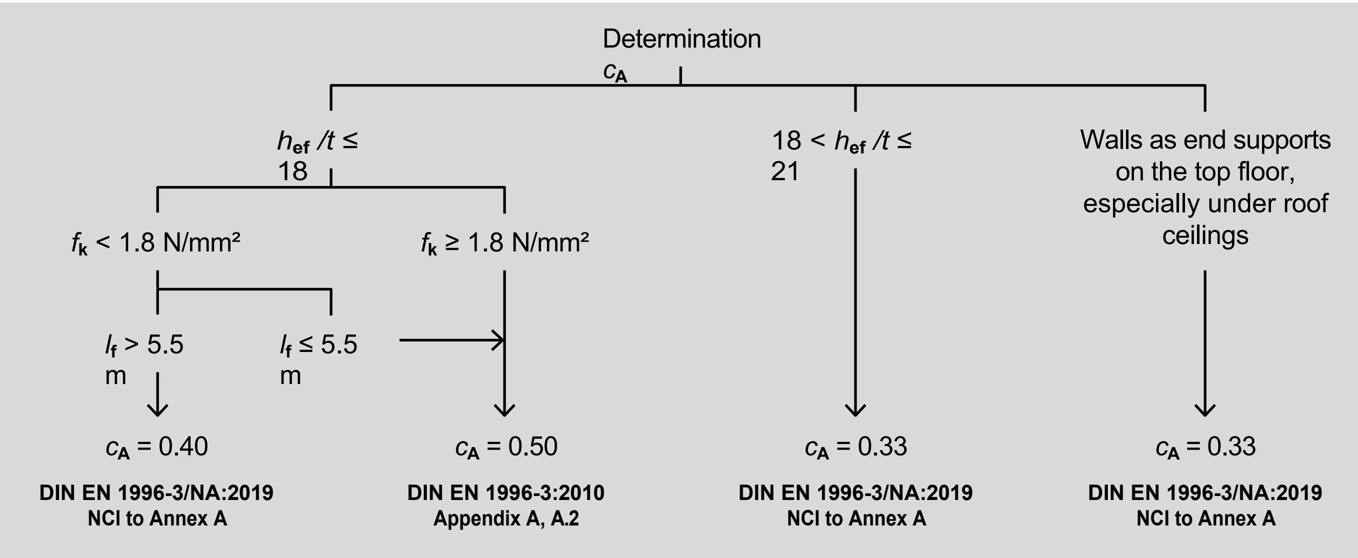
$$N_{Rd} = c_A \cdot f_d \cdot A \qquad (21)$$

DIN EN 1996-3:2010 Annex A, section A.2; Eq. (A.1)

with:

N_{Rd} Design value of the absorbable normal force
c_A Reduction coefficient
= 0.50 for walls with a slenderness $h_{ef}/t \leq 18$ generally at $f_k \geq 1.8$ N/mm² and at $f_k < 1.8$ N/mm² and simultaneous ceiling span $l \leq 5.5$ m
= 0.40 for walls with a slenderness $h_{ef}/t \leq 18$ in conjunction with a characteristic compressive strength of the masonry of $f_k < 1.8$ N/mm² and at the same time ceiling spans $l > 5.5$ m
= 0.33 for walls with slenderness $18 < h_{ef}/t \leq 21$ and generally for walls as end supports on the top floor, especially under roof ceilings
A $= l \cdot t$ Gross cross-sectional area of the wall section to be verified
f_d Design value of the compressive strength of the masonry work

Flow chart for determining the constant c_A :



For partially suspended ceilings on walls with

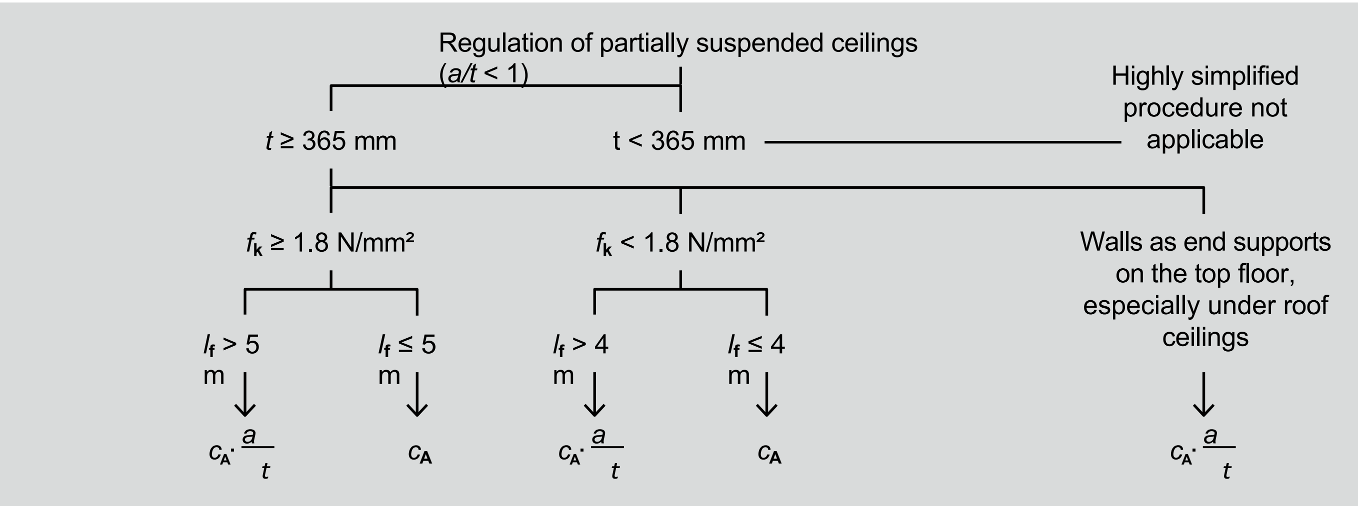
- $f_k \geq 1.8$ N/mm² and a ceiling support width > 5 m

or

- $f_k < 1.8$ N/mm² and a ceiling support width > 4 m

and generally for walls as end supports on the top floor, especially under roof ceilings, the values for c_A *must be* multiplied by a/t.

The cases for which c_A is to be multiplied by a/t can be taken from the following flow chart:



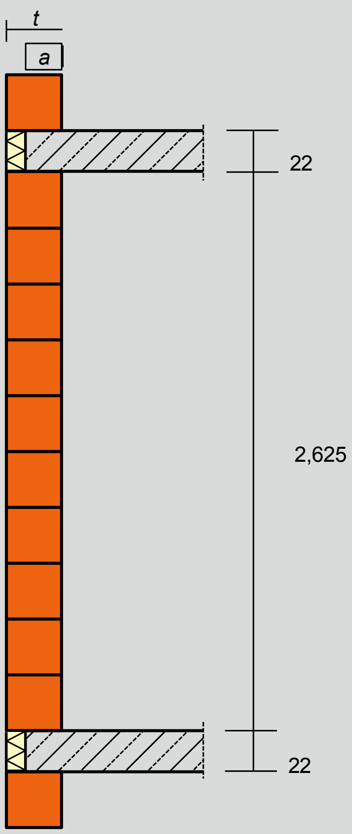
5.6 Design examples

Example 1 Monolithic exterior wall

Highly insulating plane brick with thin-bed mortar according to general building approval (abZ) with f_k = 3.0 N/mm²

Span	$l = l_f$ = 5.50 m < 6.00 m
Wall thickness	t = 0,365 m
Clear storey height	h = 2.625 m < 12 · t = 4.38 m
Support depth	a = 0,245 m a/t = 0.67 > 0.45
Payload on ceiling	q_k = 2.3 kN/m² < 5 kN/m²

The boundary conditions according to DIN EN 1996-3/NA:2019, NCI to 4.2.1.1 for the application of the simplified calculation methods are fulfilled.



Calculation with uniaxial tensioned ceiling:

N_{Gk} = 130 kN/m
N_{Qk} = 55 kN/m
$N_{Ed} = 1.4 \cdot (N_{Gk} + N_{Qk}) = 1.4 \cdot (130 + 55)$ = 259 kN/m

$h_{ef} = \rho_2 \cdot h$ **NCI to 4.2.2.4; Eq. (NA.5)**
$= 1,0 \cdot 2,625 = 2,625$ m

for wall thicknesses t > 250 mm according to NCI to 4.2.2.4, (NA.8)

$$\Phi_1 = \left(1.6 - \frac{l_f}{6}\right) \cdot \frac{a}{t} \le 0.9 \cdot \frac{a}{t}$$

NCI at 4.2.2.3, (NA.2), Glg. (NA.1), da $f_k \ge$ 1.8 [N/mm²]

$$= \left(1,6 - \frac{5,5}{6}\right) \cdot \frac{0,245}{0,365} \le 0,9 \cdot \frac{0,245}{0,365}$$

$$= 0,459$$

$$\Phi_2 = 0.85 \cdot \frac{a}{t} - 0.0011 \left(\frac{h_{ef}}{t}\right)^2$$

NCI at 4.2.2.3, (NA.5), Glg. (NA.4)

$$= 0,85 \cdot \frac{0,245}{0,365} - 0,0011 \left(\frac{2,625}{0,365}\right)^2$$

$$= 0,514$$

$\Phi = \min(\Phi_1; \Phi_2) \Rightarrow \Phi_1 = 0.459$ **NCI to 4.2.2.3, (NA.6)**

$$f_d = \zeta \cdot \frac{f_k}{\gamma_M} = 0.85 \cdot \frac{3.0}{1,5} = 1.70 \text{ N/mm}^2$$

NCI to 4.2.2.2, (NA.2)

$N_{Rd} = A \cdot f_d \cdot \Phi$
$= 1,0 \cdot 0,365 \cdot 1,70 \cdot 0,459$
$= 0.285$ MN/m

DIN EN 1996-3:2010; Section 4.2.2.1 (1)P, Glg. (4.3)

Proof:
N_{Ed} = 259 kN/m ≤ 285 kN/m = N_{Rd}
Proof fulfilled!

DIN EN 1996-3:2010; Section 4.2.2.2 (1), Glg. (4.4)

Highly simplified procedure:
h_{ef}/t = 2.625/0.365 = 7.19 **DIN EN 1996-3:2010; Annex A, A.2**
$N_{Rd} = c_A \cdot a/t \cdot A \cdot f_d$
$= 0.50 \cdot 0.67 \cdot 1.0 \cdot 0.365 \cdot 1.70$ = 208 kN/m
DIN EN 1996-3:2010; Annex A, A.2 (1), Eq.

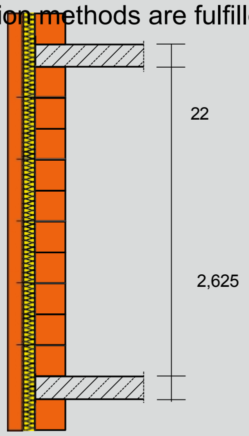
Proof (highly simplified procedure):
N_{Ed} = 259 kN/m > 208 kN/m = N_{Rd}
Proof according to highly simplified procedure **not** fulfilled!

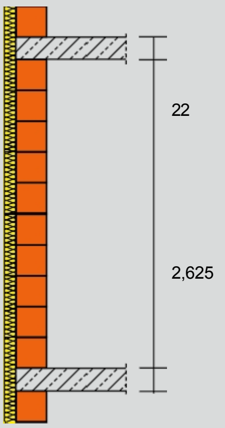
Example 2 Double-skin exterior wall or additionally insulated wall

HLzB 12 with standard masonry mortar M 5 with f_k = 5.0 N/mm²

Span	$l = l_f$ = 5.50 m < 6.00 m
Wall thickness	t = 0,24 m
Clear storey height	h = 2.625 m < 3.6 m
Support depth	a = 0.24 m
	a/t = 1.0 > 0.5
Payload on ceiling	q_k = 2.3 kN/m² < 5 kN/m²

The boundary conditions according to DIN EN 1996-3/NA:2019, NCI to 4.2.1.1 for the application of the simplified calculation methods are fulfilled.





Calculation with uniaxial tensioned ceiling:

N_{Gk} = 130 kN/m

N_{Qk} = 55 kN/m

$N_{Ed} = 1.4 \cdot (N_{Gk} + N_{Qk}) = 1.4 \cdot (130 + 55) = 259$ kN/m

$h_{ef} = \rho_2 \cdot h$ — NCI to 4.2.2.4; Eq. (NA.5)

$= 0,9 \cdot 2,625 = 2,36$ m — for wall thicknesses 175 mm < $t \leq$ 250 mm according to NCI to 4.2.2.4, (NA.8)

$\Phi_1 = 1.6 - \frac{l_f}{6} \cdot \frac{a}{t} \leq 0.9 \cdot \frac{a}{t}$ — NCI at 4.2.2.3, (NA.2), Glg. (NA.1), da $f_k \geq 1.8$ [N/mm²]

$= 1,6 - \frac{5,5}{6} \cdot \frac{0,24}{0,24} \leq 0,9 \cdot \frac{0,24}{0,24}$

$= 0,683$

$\Phi_2 = 0.85 \cdot \frac{a}{t} - 0.0011 \cdot \left(\frac{h_{ef}}{t}\right)^2$ — NCI at 4.2.2.3, (NA.5), Glg. (NA.4)

$= 0,85 \cdot \frac{0,24}{0,24} - 0,0011 \cdot \left(\frac{2,36}{0,24}\right)^2$

$= 0,743$

$\Phi = \min(\Phi_1; \Phi_2) \Rightarrow \Phi_1 = 0.683$ — NCI to 4.2.2.3, (NA.6)

$f_d = \zeta \cdot \frac{f_k}{\gamma_M} = 0.85 \cdot \frac{5.0}{1,5} = 2.83$ N/mm² — NCI to 4.2.2.2, (NA.2)

$N_{Rd} = A \cdot f_d \cdot \Phi$ — DIN EN 1996-3:2010; Section 4.2.2.1 (1)P, Glg. (4.3)

$= 1,0 \cdot 0,24 \cdot 2,83 \cdot 0,683$

$= 0.465$ MN/m

Proof: — DIN EN 1996-3:2010; Section 4.2.2.2 (1), Glg. (4.4)

$N_{Ed} = 259$ kN/m ≤ 465 kN/m $= N_{Rd}$

Proof fulfilled!

Highly simplified procedure:

$h_{ef}/t = 2.36/0.24 = 9.84$

$c_A = 0,50$ — DIN EN 1996-3:2010; Annex A, A.2

$N_{Rd} = c_A \cdot A \cdot f_d = 0.5 \cdot 1.0 \cdot 0.24 \cdot 2.83 = 340$ kN/m — DIN EN 1996-3:2010; Annex A, A.2 (1), Eq.

Proof (highly simplified procedure):

$N_{Ed} = 259$ kN/m < 340 kN/m $= N_{Rd}$

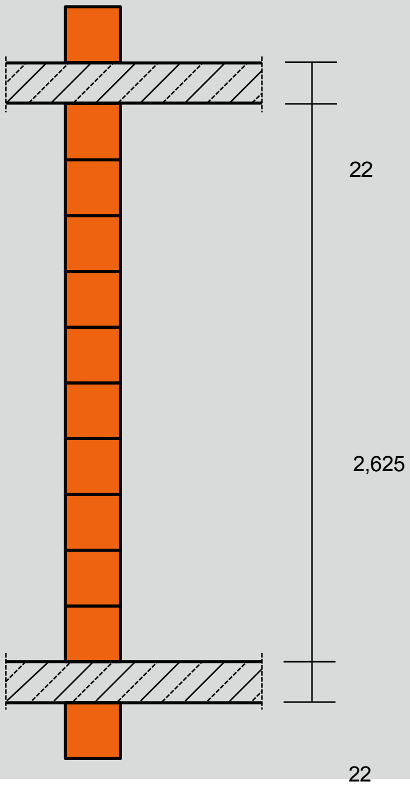
Proof fulfilled!

Example 3 Interior wall

Planziegel PHLzB 12 with thin-bed mortar with f_k = 4.7 N/mm² (see table 5)

Span $l = l_f$ = 5.50 m < 6.00 m
Wall thickness t = 0,24 m
Clear storey height h = 2.625 m < 3.6 m
Support depth a = 0.24 m
a/t = 1.0 > 0.5
Payload on ceiling q_k = 2.3 kN/m² < 5 kN/m²

The boundary conditions according to DIN EN 1996-3/NA:2019, NCI to 4.2.1.1 for the application of the simplified calcul-
The requirements for the use of the methods are fulfilled.



Calculation with uniaxial tensioned ceiling:

N_{Gk} = 210 kN/m
N_{Qk} = 90 kN/m
$N_{Ed} = 1.4 \cdot (N_{Gk} + N_{Qk}) = 1.4 \cdot (210 + 90)$ = 420 kN/m

$h_{ef} = \rho_2 \cdot h$ NCI to 4.2.2.4; Eq. (NA.5)

$= 0{,}9 \cdot 2{,}625 = 2{,}36$ m for wall thicknesses 175 mm < t ≤ 250 mm according to NCI to 4.2.2.4, (NA.8)

Φ_1 Not decisive, as there is no end support.

$$\Phi_2 = 0.85 \cdot \frac{a}{t} - 0{,}0011 \cdot \left(\frac{h_{ef}}{t}\right)^2$$ NCI at 4.2.2.3, (NA.5), Glg. (NA.4)

$$= 0{,}85 \cdot \frac{0{,}24}{0{,}24} - 0{,}0011 \cdot \left(\frac{2{,}36}{0{,}24}\right)^2$$

$= 0{,}743$

$\Phi = \Phi_2 = 0{,}743$ NCI to 4.2.2.3, (NA.6)

$$f_d = \zeta \cdot \frac{f_k}{\gamma_M} = 0.85 \cdot \frac{4.7}{1{,}5} = 2.66 \text{ N/mm²}$$ NCI to 4.2.2.2, (NA.2)

$N_{Rd} = A \cdot f_d \cdot \Phi$ DIN EN 1996-3:2010; Section 4.2.2.1 (1)P, Glg. (4.3)
$= 1{,}0 \cdot 0{,}24 \cdot 2{,}66 \cdot 0{,}743$
$= 0.475$ MN/m

Proof: DIN EN 1996-3:2010; Section 4.2.2.2 (1), Glg. (4.4)
N_{Ed} = 420 kN/m ≤ 475 kN/m = N_{Rd}
Proof fulfilled!

Highly simplified procedure:
$h_{ef}/t = 2.36/0.24 = 9.84$
$c_A = 0{,}50$ DIN EN 1996-3:2010; Annex A, A.2
$N_{Rd} = c_A \cdot A \cdot f_d = 0.50 \cdot 1.0 \cdot 0.24 \cdot 2.66$ = 320 kN/m
DIN EN 1996-3:2010; Annex A, A.2 (1), Eq.

Proof (highly simplified procedure):
N_{Ed} = 420 kN/m > 320 kN/m = N_{Rd}
Proof according to highly simplified procedure **not** fulfilled!

5.7 Verification of the minimum load

For walls that serve as end supports for slabs or roofs and are subjected to wind loads, a verification is required in accordance with DIN EN 1996-3/NA, NCI to 4.2.1.2 (NA.4).
of the minimum superimposed load of the walls. The verification may be carried out at the centre of the wall height, taking into account the proportion of the wall's own weight acting there:

$$N_{Ed} \geq \frac{3 \cdot q_{Ewd} \cdot h2 \cdot b}{16 \cdot \left(a - \frac{h}{300}\right)} \qquad (22)$$

DIN EN 1996-3/NA:2019, NCI to 4.2.1.2, (NA.4), eq. (y)

with:

N_{Ed} Design value of the smallest vertical load in the centre of the wall height in the wall cross-section to be verified.
q_{Ewd} Design value of the wind load per unit area
h Clear storey height
b Width of action of the wind load
a Ceiling support depth

Figure 1 shows the permissible maximum wall height h as a function of the existing design wind load w_d and wall thickness t for a related slab support depth a/t = 2/3.

It can be seen that in inland wind zones 1 and 2 the usual storey heights can be realised without any problems. The verification of the minimum load can usually be omitted in these wind zones.

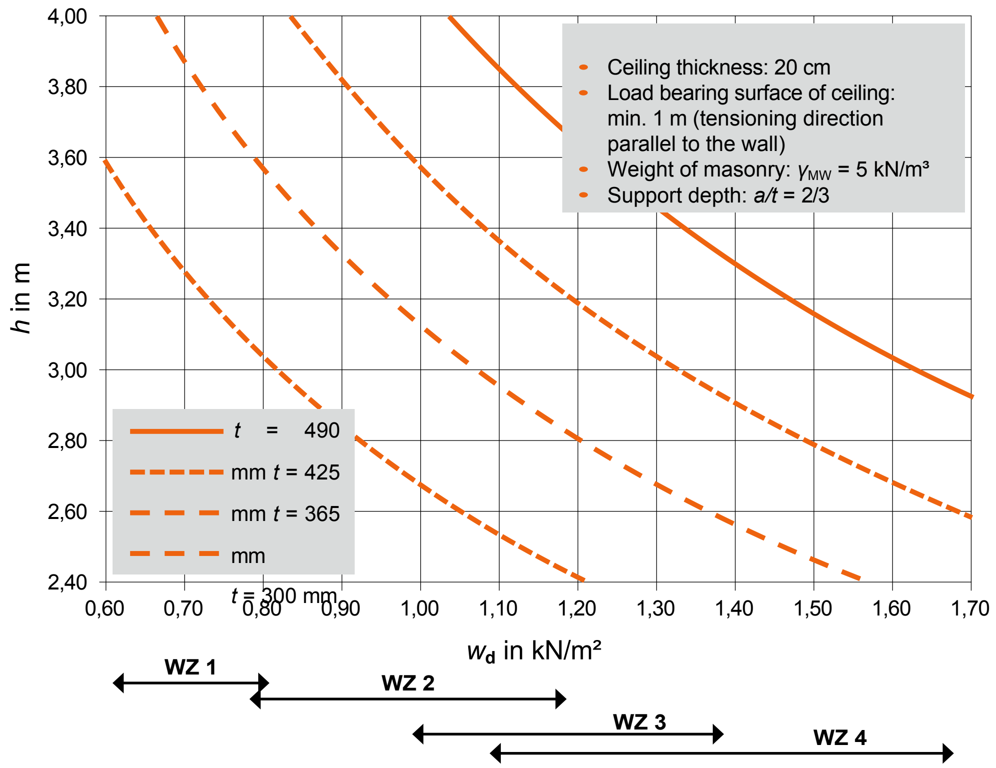


Figure 1: Maximum wall height as a function of the design wind load [13].

It should be noted that the verification according to equation (22) refers to walls supported on two sides (top and bottom), since the transfer of wind loads can be considered given for walls supported on four sides anyway.

If the verification is not fulfilled, the walls or wall sections in question can be designed and constructed as non-load-bearing exterior walls in accordance with Section 9 - on the safe side.

In [14], the verification with equation (22) is further developed on the basis of an extended arch model, taking into account the effects of the second-order theory. In [14], the verification with equation (22) is further developed on the basis of an extended arc model, taking into account the effects of second-order theory, in such a way that more favourable values (lower required minimum superimposed loads) result than according to equation (22).

By considering realistic boundary conditions and design approaches, it can be shown that in wind zones 1 to 3 and wall slendernesses h/t ≤ 15, the verification of the required minimum loads is usually fulfilled and can therefore be omitted.

6 Verification of horizontally loaded wall panels according to DIN EN 1996-1-1/NA

6.1 General

According to DIN EN 1996-3/NA, a mathematical verification of the stiffening may be dispensed with if the storey ceilings are formed as stiff slabs or if statically verified, sufficiently stiff ring beams are available and if there is an obviously sufficient number of sufficiently long stiffening walls in the longitudinal and transverse direction of the building, which are led up to the foundations without major weaknesses and without projections.

If it is not evident from the outset that the stiffening of a structure is sufficient, a mathematical verification of the stiffening must be carried out in accordance with the general rules of DIN EN 1996-1-1/NA, Section 6.2, in accordance with DIN EN 1996-3/NA, NDP to 4.1 (1). This is done by comparing the acting shear force V_{Ed} with the shear force bearing capacity V_{Rdlt} :

$$V_{Ed} \leq V_{Rdlt} \qquad (23)$$

DIN EN 1996-1-1/NA:2019, NCI to 6.2 (NA.8), Glg. (NA.18)

with:

V_{Ed} Design value of the acting shear force

V_{Rdlt} Design value of the shear force bearing capacity

6.2 Rated value of the acting shear force V_{Ed}

The decisive horizontal actions on masonry buildings can, for example, result from

- Wind pressure and wind suction,
- Imperfections (e.g. unintentional crookedness),
- Earth pressure and
- Earthquake

result. They are first introduced via the façade into the stiff floor slabs and from there further into the stiffening wall slabs. The distribution to the wall slabs depends on the static system (determinate or indeterminate).

The modelling of piers as well as short and/or vertically low-loaded exterior walls as pendulum supports is a possibility to be considered in order to verify the horizontal loads via vertically high-loaded interior walls.

6.3 Rated value of the cross wire load capacity V_{Rdlt}

The design value of the shear force bearing capacity is determined as follows:

$$V_{Rdlt} = l_{cal} \cdot f_{vd} \cdot \frac{t}{c} \qquad (24)$$

DIN EN 1996-1-1/NA:2019, NCI to 6.2 (NA.12), Glg. (NA.19)

with:

l_{cal} Calculated wall length, see 6.3.1

f_{vd} Design value of the shear strength

$$f_{vd} = \frac{f_{vk}}{\gamma_M} = \frac{f_{vlt}}{\gamma_M} \qquad (25)$$

DIN EN 1996-1-1:2013, clause 2.4.1 (1)P

with:

f_{vk} $= f_{vlt}$ characteristic shear strength, see 6.3.2

γ_M Partial safety factor for material properties (here: $\gamma_M = 1.5$)

t Wall thickness

c Shear stress distribution factor
$= 1.0$ for $h/l \le 1.0$
$= 1.5$ for $h/l \ge 2.0$
Intermediate values may be interpolated linearly

h Clear wall height

l Length of the wall panel

6.3.1 Calculated wall length l_{cal}

For the verification of wall panels modelled as a cantilever model under wind load, a mathematically increased wall length according to equation (26) may be used. In all other cases, $l_{cal} = l$ or $l_{c,lin}$.

$$l_{cal} = 1.125 \cdot l \le 1.333 \cdot l_{c,lin} \qquad (26)$$

DIN EN 1996-1-1/NA:2019, NCI to 6.2 (NA.12), Glg. (NA.19)

with:

l Length of the wall panel

$l_{c,lin}$ Pressed-over length of the wall panel to be used for the calculation

$$l_{c,lin} = \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{e_w}{l}\right) \cdot l \le l \qquad (27)$$

DIN EN 1996-1-1/NA:2019, NCI to 6.2 (NA.12), Glg. (NA.20)

e_w Eccentricity of the acting normal force in the longitudinal direction of the wall

$$e_w = \frac{M_{Ed}}{N_{Ed}} \qquad (28)$$

DIN EN 1996-1-1/NA:2019, NCI to 6.1.2.2 (NA.3), Eq. (NA.15)

M_{Ed} Design value of the acting moment in the longitudinal direction of the wall

N_{Ed} Design value of the acting normal force

6.3.2 Characteristic Shear strength f_{vk} = f_{vlt}

The characteristic shear strength f_{vk} = f_{vlt} results in Depending on whether frictional or stone tensile failure becomes mathematically decisive:

$$f_{vk} = f_{vlt} = \min(f_{vlt1}, f_{vlt2}) \quad (29)$$

DIN EN 1996-1-1/NA:2019, NDP to 3.6.2 (3) b)

For disc shear, the following applies for non-mortared butt joints:

f_{vlt1} Characteristic shear strength at frictional failure

$$f_{vlt1} = 0.5 \cdot f_{vk0} + 0.4 \cdot \sigma_{Dd} \quad (30)$$

DIN EN 1996-1-1/NA:2019, NDP to 3.6.2 (4) b) i.V.m. NDP to 3.6.2 (3) b) Eq. (NA.4)

f_{vk0} Adhesion shear strength according to table 9 compare DIN EN 1996-1-1/NA:2019, Tab.NA.12 see also 6.3.3

σ_{Dd} Rated value of the associated compressive stress. The following applies to rectangular cross-sections:

$$\sigma_{Dd} = \frac{N_{Ed}}{l_{c,lin} \cdot t} \quad (31)$$

DIN EN 1996-1-1/NA:2019, NDP to 3.6.2 (3) b), eq. (NA.5)

f_{vlt2} Characteristic shear strength at stone tensile failure

$$f_{vlt2} = 0{,}45 \cdot f_{bt,cal} \cdot \sqrt{1 + \frac{\sigma_{Dd}}{f_{bt,cal}}} \quad (32)$$

DIN EN 1996-1-1/NA:2019, NDP to 3.6.2 (3) b), eq. (NA.5)

$fbt_{,cal}$ = 0.020 - f_{st} for hollow blocks
= 0.026 - f_{st} for vertically perforated bricks and bricks with grip holes or grip pockets
= 0.032 - f_{st} for solid stones without finger holes or finger pockets

f_{st} converted average minimum compressive strength of the bricks according to Table 10 (compare DIN EN 1996-1-1/NA:2019, Tab.NA.3)

Table 9 Characteristic values f_{vk0} of the adhesive shear strength in N/mm²

Masonry mortar according to DIN 20000-412 or DIN 18580	M 2,5	M 5	M 10	M 20
Mortar groups according to DIN 1053-1	NM II	NM IIa LM21 LM36	NM III DM	NM IIIa
Mortar compressive strength f_m [N/mm²]	2,5	5,0	10,0	20,0
Adhesive shear strength f_{vk0} [N/mm²]	0,08	0,18	0,22	0,26

Table 10 Converted mean minimum compressive strength Calculated values for f_{St} depending on the compressive strength class

Compressive strength class of masonry bricks (SFK)	4	6	8	10	12	16	20	28	36	48	60
Converted mean minimum compressive strength f_{st} [N/mm²]	5,0	7,5	10,0	12,5	15,0	20,0	25,0	35,0	45,0	60,0	75,0

6.3.3 Edge strain verification

According to DIN EN 1996-1-1/NA, NCI to 7.2, an edge strain check is only required if the adhesive shear strength f_{vk0} is taken into account when determining the shear strength.

If the adhesive shear strength f_{vk0} is used to determine the characteristic shear strength for frictional failure according to equation (30), the calculated edge strain $\varepsilon_R \leq 10^{-4}$ must also be verified for wind shears with gaping joints under characteristic loads ($e_{w,k} > l/6$):

$$\varepsilon_R = \frac{1}{E}\left[\frac{l}{l_{c,lin}} - 1\right] \cdot \sigma_D \leq 10^{-4} \qquad (33)$$

DIN EN 1996-1-1/NA:2019, NCI to 7.2 (NA.10)

with:

E Modulus of elasticity
for brick masonry E = 1100 - f_k can be assumed
l Wall length
$l_{c,lin}$ according to equation (27)
σ_D Existing compressive stress

$$\sigma_D = \frac{2 \cdot N_{Ek}}{A_{c,lin}} = \frac{2 \cdot N_{Ek}}{l_{c,lin} \cdot t} \qquad (34)$$

N_{Ek} Affected normal force in the characteristic design situation
t Wall thickness

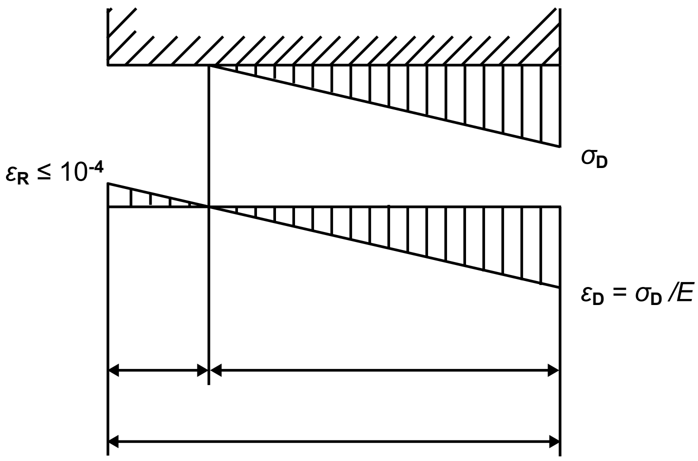


Figure 2: Stress and strain distribution for cross-sections subjected to excentric loads

6.4 Verification of the bending pressure load-bearing capacity

In the case of wall panels subject to shear forces, the bending pressure check about the strong axis must always be carried out taking into account the load combination max M ⊕ min N (usually at the base of the wall):

$$N_{Ed} \leq N_{Rd} \qquad (35)$$

DIN EN 1996-3:2010, clause 4.2.2.1, eq. (4.3)

with:
N_{Ed} Design value of the acting normal force
N_{Rd} Design value of the absorbable normal force

$$N_{Rd} = A \cdot f_d \cdot \Phi_y \qquad (36)$$

DIN EN 1996-3:2010, clause 4.2.2.2, eq. (4.4)

A = l - t Gross cross-sectional area of the wall section to be verified
f_d Design value of the compressive strength of the masonry according to equation (12)
Φ_y Reduction coefficient (around the strong axis):

$$\Phi_y = 1 - 2 \cdot \frac{e_w}{l} \tag{37}$$

DIN EN 1996-1-1/NA:2019, clause NCI to 6.1.2.2, eq. (NA.14)

e_w Eccentricity of the acting normal force in the longitudinal direction of the wall

6.5 Combined stress

In the case of a combined load from bending around the strong and around the weak axis, a bending pressure check (buckling check) must also be carried out at half the wall height. For simplification, the reduction values for both axes may be combined multiplicatively to determine N_{Rd} :

$$N_{Rd,\,centre} = A \cdot f_d \cdot \Phi_x \cdot \Phi_{y,\,centre} \tag{38}$$

with:

Φ_x Reduction coefficient in the centre of the wall for bending about the weak axis ($\Phi_x = \Phi_2$ according to equation (19))

$\Phi_{y,centre}$ Reduction coefficient at wall height centre for bending around the strong axis

$$\Phi_{y,\,middle} = 1 - 2 \cdot \frac{e_{w,\,middle}}{l} \tag{39}$$

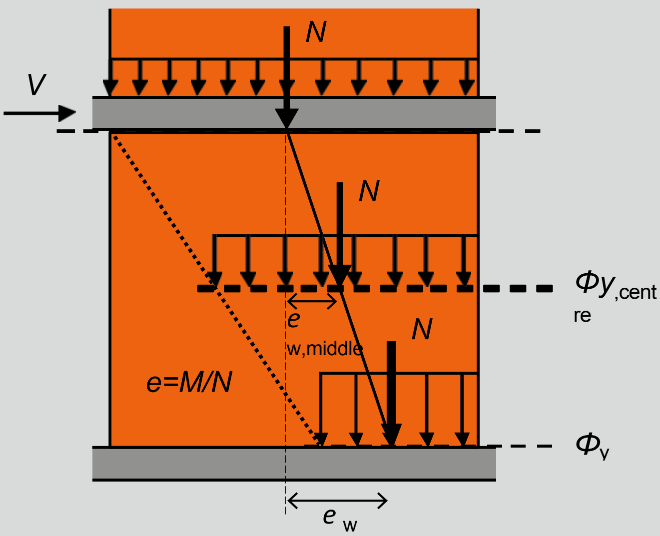
General technical approvals (abZ) / general type approvals (aBG) can contain further regulations.

6.6 Example

Example 4 Internal wall from example 3 as bracing wall

PHLzB 12 with thin-bed mortar
- f_k = 4.7 N/mm² (see table 5)
- f_{vk0} = 0.22 N/mm² (see table 9)
- f_{st} = 15.0 N/mm² (see table 10)

Butt joints not mortared
Wall length l = 3.0 m
Wall thickness t = 0.24 m
clear storey *heighth* = 2.625 m
n_{Gk} = 90 kN/m
V_{Qk} = 60 kN (from wind)



Verification of the shear force bearing capacity

$$e_w = \frac{\max M_{Ed}}{\min N_{Ed}} = \frac{1.5 \cdot V_{Qk} \cdot h}{1.0 \cdot l \cdot n_{Gk}} = \frac{1.5 \cdot 60 \cdot 2.625}{1.0 \cdot 3.0 \cdot 90} = 0.875 \text{ m}$$

$$l_{c,lin} = \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{e_w}{l}\right) \cdot l = \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{0.875}{3,0}\right) \cdot 3.0 = 1.875 \text{ m} < 3.0 = l$$

$$l_{cal} = \min \left\{ \begin{array}{l} 1,125 \cdot l = 1,125 \cdot 3,0 = 3,38 \\ 1.333 \cdot l_{c,lin} = 1.333 \cdot 1.875 = 2.50 \end{array} \right\} = 2,50 \text{ m}$$

$$\sigma_{Dd} = \frac{\min N_{Ed}}{l_{c,lin} \cdot t} = \frac{1.0 \cdot 3 \cdot 90}{1,875 \cdot 0,24} = 600 \text{ kN/m}^2 = 0.60 \text{ N/mm}^2$$

$$f_{vlt} = f_{vk} = \min \left\{ \begin{array}{l} 0,5 \cdot 0,22 + 0,4 \cdot 0,60 = 0,35 \\ 0,45 \cdot 0.026 \cdot 15 \cdot \sqrt{1 + \frac{0,60}{0,026 \cdot 15}} = 0,28 \end{array} \right\} = 0.28 \text{ N/mm}^2$$

$$\frac{h}{l} = \frac{2.625}{3,0} = 0.88 < 1.0 \Rightarrow c = 1.0$$

$$V_{Rdlt} = l_{cal} \cdot \frac{f_{vlt}}{\gamma_M} \cdot \frac{t}{c} = 2.50 \cdot \frac{0.28}{1,5} \cdot \frac{0.24}{1,0} = 0.112 \text{ MN} = 112 \text{ kN}$$

Verification: $V_{Ed} = 1.5 \cdot 60.0 = 90.0 \text{ kN} < 112 \text{ kN} = V_{Rdlt}$

Verification of the edge strain

Since the adhesive shear strength f_{vk0} was used in the calculation of the shear load capacity, the edge strain under characteristic loads must be verified.

$$e_{w,k} = \frac{1.0 \cdot V_{Qk} \cdot h}{1.0 \cdot l \cdot n_{Gk}} = \frac{1.0 \cdot 60 \cdot 2.625}{1,0 \cdot 3,0 \cdot 90} = 0.58 \text{ m} > 0.5 \text{ m} = \frac{l}{6} \text{ (cross-section cracked)}$$

$$l_{c,lin} = \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{e_{w,k}}{l}\right) \cdot l = \frac{3}{2} \cdot \left(1 - 2 \cdot \frac{0,58}{3,0}\right) \cdot 3,0 = 2,76 \text{ m}$$

$$\sigma_{Dd} = \frac{2 \cdot N_{Ek}}{l_{c,lin} \cdot t} = \frac{2 \cdot 1.0 \cdot 3 \cdot 90}{2,76 \cdot 0,24} = 0.815 \text{ N/mm}^2$$

Detection: $\varepsilon_R = \frac{1}{E} \cdot \left(\frac{l}{l_{c,lin}} - 1\right) \cdot \sigma_D = \frac{1}{1100 \cdot 4,7} \cdot \left(\frac{3}{2,76} - 1\right) \cdot 0,815 = 0,000014 \leq 10^{-4}$

Verification of the bending load capacity around the strong axis (at the wall foot)

$$\Phi_y = 1 - 2 \cdot \frac{e_w}{l} = 1 - 2 \cdot \frac{0.875}{3} = 0.42$$

$$N_{Rd} = A \cdot \zeta \cdot \frac{f_k}{\gamma_M} \cdot \Phi_y = 3.0 \cdot 0.24 \cdot 1.0 \cdot \frac{4,7}{1,5} \cdot 0.42 = 948 \text{ kN}$$ (ζ = 1.0, as wind acts briefly).

Verification: min N_{Ed} = 1.0 · 3.0 · 90.0 = 270 kN < 948 kN = N_{Rd}

Verification of the combined stress (in the centre of the wall height)

$$e_{w,middle} = \frac{\max M_{Ed}}{\min N_{Ed}} = \frac{1.5 \cdot V_{Qk} \cdot h}{1.0 \cdot l \cdot n_{Gk}} = \frac{1.5 \cdot 60 \cdot 2.625/2}{1,0 \cdot 3,0 \cdot 90} = 0.44$$

$\Phi_x = \Phi_2$ = 0.74 (cf. example 3)

$$\Phi_{y,middle} = 1 - 2 \cdot \frac{e_{w,middle}}{l} = 1 - 2 \cdot \frac{0.44}{3} = 0.71$$

$$N_{Rd,Middle} = A \cdot \zeta \cdot \frac{f_k}{\gamma_M} \cdot \Phi_x \cdot \Phi_{y,middle} = 3.0 \cdot 0.24 \cdot 1.0 \cdot \frac{4.7}{1,5} \cdot 0.74 \cdot 0.71 = 1.185 \text{ MN} = 1185 \text{ kN}$$

Verification: min N_{Ed} = 1.0 · 3.0 · 90.0 = 270 kN < 1185 kN = $N_{Rd,mitte}$

7 Design in case of fire according to DIN EN 1996-1-2/NA

7.1 General

The resistance of building components to fire is characterised by the fire resistance class. It indicates the minimum duration in minutes that a building component can withstand a fire load. The classification of building materials or components into fire resistance classes is carried out according to DIN EN 1996-1-2/NA or DIN EN 1996-1-2/NA. according to general building inspectorate approvals (abZ) / general building type approvals (aBG) with the help of the tables given there. In addition to other influencing factors, the static utilisation or the existing load is of particular importance for the corresponding classification of a wall.

7.2 Utilisation factors in case of fire

In DIN EN 1996-1-2/NA and in the general building inspectorate approvals (abZ) / general building type approvals (aBG), three different utilisation factors are regulated for masonry, the definitions of which are compiled in Table 11. In contrast to a design according to DIN 1053-1, the value for the full utilisation according to DIN EN 1996-1-2/NA is no longer 1.0, but a maximum of 0.7, since the design value of the action in the case of fire $N_{Ed,fi}$ is reduced compared to the design value of the action in the "cold" design N_{Ed} with the reduction coefficient η_{fi} :

$$N_{Ed,fi} = \eta_{fi} \cdot N_{Ed} \qquad (40)$$

DIN EN 1996-1-2/NA:2013, NDP to 4.5(3) Eq. (NA.4)

The reduction coefficient η_{fi} depends on the ratio of variable loads to permanent loads in the building and is set at a maximum of 0.7 in DIN EN 1996-1-2/NA, which is on the safe side. However, it can also be calculated more precisely with equation (41).

$$\eta_{fi} = \frac{G_k + \Psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \qquad (41)$$

DIN EN 1996-1-2:2011, clause 2.4.2 (3), eq. (2.5)

with:

$Q_{k,1}$	decisive variable load
G_k	characteristic value for permanent loads
γ_G	Partial safety factor for permanent loads
$\gamma_{Q,1}$	Partial safety factor for the variable load 1
Ψ_{fi}	Combination coefficient for frequent values, either $\Psi_{1,1}$ or $\Psi_{2,1}$

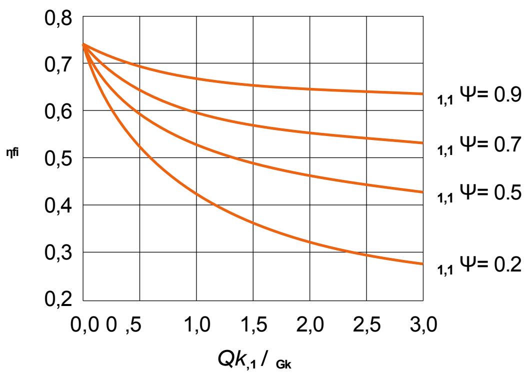


Figure 3: Reduction coefficient η_{fi} as a function of the ratio $Q_{k,1}$ / G_k from DIN EN 1996-1-2, section 2.4.2

For usual ratios between variable and permanent loads $Q_{k,1}$ / G_k = 0.5 is η_{fi} = 0.6.

Table 11	Definition of utilisation factors	
Utilisation factor	**Definition**	**Explanation**
α_2	α_2 = 1.0 corresponds to the full load-bearing capacity for a design according to the simplified calculation method of DIN 1053-1.	The value is used in DIN 4102-4 and in general building inspectorate approvals (abZ) / general building type approval (aBG) when dimensioning according to DIN 1053-1.
$\alpha 6_{,fi}$	$\alpha_{6,fi}$ = 0.7 corresponds to the maximum permissible load of a masonry component in the event of fire when dimensioned in accordance with DIN EN 1996/NA.	The maximum permissible load generally corresponds to the full load-bearing capacity when dimensioned according to the simplified calculation method of DIN 1053-1. The value is used in DIN EN 1996-1-2/NA for masonry made of masonry bricks with normal and light mortar.
α_{fi}	α_{fi} = 0.7 corresponds to the full load-bearing capacity for a design according to DIN EN 1996-1-1/ NA or according to the general technical approval (abZ) / general type approval (aBG) with the design rules according to DIN EN 1996-1-1/NA.	The value is used in general building inspectorate approvals (abZ) / general building type approvals (aBG) as well as in DIN EN 1996-1-2/NA A1 for plane vertically perforated bricks with thin-bed mortar.

7.2.1 Utilisation factor $\alpha_{6,fi}$

In DIN EN 1996-1-2/NA, the utilisation factor $\alpha_{6,fi}$ is used for masonry bricks in combination with normal or light masonry mortar.

The definition of a new utilisation factor $\alpha_{6,fi}$ as a replacement for the utilisation factor α_2 known from DIN 4102-4 became necessary because the extensive tabular values in DIN 4102-4 could not be easily transferred to a design according to DIN EN 1996-1-1/NA without new tests.

The utilisation factor $\alpha_{6,fi}$ takes into account that the maximum permissible normal forces can be greater or smaller in a design according to DIN EN 1996/NA than in a design according to the simplified calculation method of DIN 1053-1. This results essentially from the newly defined calculation of the Load-bearing capacity for the failure case buckling as well as on the basis of the newly defined characteristic compressive strength of masonry f_k .

The utilisation factor $\alpha_{6,fi}$ is determined as follows:

$$\alpha_{6,fi} = \omega \cdot \frac{15}{25 - \frac{h_{ef}}{t}} \cdot \frac{N_{Ed,fi}}{l \cdot t \cdot \frac{f_k}{k_0} \cdot \left(1 - 2 \cdot \frac{e_{mk,fi}}{t}\right)} \leq 0{,}7 \qquad (42)$$

for $10 \leq \frac{h_{ef}}{t} \leq 25$

DIN EN 1996-1-2/NA:2013, NDP to 4.5(3), eq. (NA.1)

$$\alpha_{6,fi} = \omega \cdot \frac{N_{Ed,fi}}{l \cdot t \cdot \frac{f_k}{k_0} \cdot \left(1 - 2 \cdot \frac{e_{mk,fi}}{t}\right)} \leq 0{,}7 \quad (43)$$

for $\frac{h_{ef}}{t} < 10$

DIN EN 1996-1-2/NA:2013, NDP to 4.5(3), eq. (NA.2)

with:

ω Adjustment factor of the masonry parameters to the different types of bricks (brick-mortar combinations) based on fire tests, see Table 12.

h_{ef} Bend length of the wall

t Wall thickness

$N_{Ed,fi}$ Design value of the normal force (action) in case of fire according to equation (40)

l Wall length

f_k characteristic compressive strength of the masonry

k_0 = 1.25 for wall cross-sections < 0.1 m²
= 1.00 for wall cross-sections ≥ 0.1 m²

$e_{mk,fi}$ Planned centre of $N_{Ed,fi}$ at half storey height

When using the simplified calculation methods, the following simplifications may be made in equations (42) and (43):

for full-bleed ceilings (a/t = 1.0):

$$\left(1 - 2 - \frac{e_{mk,fi}}{t}\right) = 1{,}0$$

for partially overlying ceilings (a/t < 1.0):

$$\left(1 - 2 - \frac{e_{mk,fi}}{t}\right) = a/t$$

Table 12 Adaptation factor ω depending on the brick-mortar combination used and associated tables for classification into a fire resistance class

Brick according to DIN EN 771-1 in connection with DIN 20000-401 as well as DIN 105-100	Mortar		associated table in DIN EN 1996-1-1/NA or DIN EN 1996-3/NA	ω [-]
Vertically perforated brick HLzA, HLzB, HLzE Masonry block T1	M 2,5 M 5 M 10 M 20	(NM II) (NM IIa) (NM III) (NM IIIa)	NA.4 NA.D.1	2,2
Vertically perforated brick HLzW, T2, T3, T4 bricks			NA.5 NA.D.2	1,8
Solid brick Mz	M 2,5	(NM II)	NA.6 NA.D.3	3,3
	M 5	(NM IIa)		3,0
	M 10 M 20	(NM III) (NM IIIa)		2,6
Brick	M 5	(LM)	NA.8 NA.D.5	2,2

7.2.2 Utilisation factor α_{fi}

The utilisation factor α_{fi} is used in general building inspectorate approvals (abZ) / general building inspectorate approvals (aBG) and in DIN EN 1996-1-2/NA Amendment A1 for vertically perforated bricks with thin-bed mortar.

$$\alpha_{fi} = \frac{N_{Ed,fi}}{N_{Rd}} \quad (44)$$

DIN EN 1996-1-2/NA:2013, NDP to 4.5(3), eq. (NA.3)

with:

$N_{Ed,fi}$ Design value of the normal force (action) in case of fire according to equation (40)

N_{Rd} Design value of the vertical load-bearing resistance in case of fire

In contrast to the cold measurement, the resistance $N_{Rd(fi)}$ in the case of fire is determined with a creep factor ζ = 1.0, since the case of fire represents a short-term stress [15]. Thus the following applies:

$$N_{Rd,fi} = \frac{1}{0{,}85} \cdot N_{Rd} = 1.176\ N_{Rd} \quad (45)$$

For usual load combinations in building construction (assumption: ratio $Q_{k,1}$ / G_k = 0.5 and thus η_{fi} = 0.6) the utilisation factor α_{fi} for the full "cold" load-bearing capacity according to DIN EN 1996-1-1/NA ($N_{Ed} = N_{Rd}$) thus amounts to

$$\alpha_{fi} = \frac{N_{Ed,fi}}{N_{Rd,fi}} = \frac{0{,}6 \cdot N_{Ed}}{1{,}176 \cdot N_{Rd}} = 0{,}51 \quad (46)$$

The required wall thickness for classification into a fire resistance class can be taken directly from the tables in DIN EN 1996-1-2/NA or in the general building inspectorate approvals (abZ) / general building type approvals (aBG) when the factor α_{fi} is applied.

7.3 Examples

Example 5 Double-skin exterior wall from example 2

HLzB 12 with standard masonry mortar M 5 (NM IIa) $\longrightarrow$ f_k = 5.0 N/mm² (see Table 5)

Required fire resistance class: REI 90 (fire-resistant)

Bulk density class 1,2
Wall thickness t = 0.24 m
Support depth a = 0.24

$$\frac{a}{t} = 1{,}0$$

$N_{Ed,fi}$ = 259 kN/m
h_{ef} = 2,36 m

$$\frac{h_{ef}}{t} = \frac{2{,}36}{0{,}24} = 9{,}83$$

$$\alpha_{6,fi} = \omega \cdot \frac{N_{Ed,fi}}{l \cdot t \cdot \frac{f_k}{k_0} \cdot \left(1 - 2 \cdot \frac{e_{mk,fi}}{t}\right)} = 2{,}2 \cdot \frac{0{,}7 \cdot 0{,}259}{1{,}0 \cdot 0{,}24 \cdot \frac{5{,}0}{1{,}0} \cdot (1{,}0)} = 0{,}33 \leq 0{,}42$$

Since only the load-bearing inner shell of double-skin exterior walls is assessed in terms of fire protection, the inner shell is classified as a load-bearing, room-enclosing single-skin wall according to Table NA.B.1.2 of DIN EN 1996-1-2/NA.

The non-load-bearing outer shell protects the inner shell from external fire loads. Therefore, according to NCI 4.2 "Interior and exterior plasters", it may be applied as a plaster layer.

For HLzB of gross density class 1.2 with $\alpha_{6,fi} \leq 0.42$, DIN EN 1996-1-2/NA:2013, Table NA.B.1.2, line 1.2 applies.
* Required minimum wall thickness for REI 90: 115 mm < 240 mm = t

Verification: t_{vorh} = 240 mm > min t (REI 90) = 115 mm

Verification fulfilled!

Example 6 Monolithic exterior wall from example 1

Highly insulating plane brick with thin-bed mortar according to general building approval (abZ) with f_k = 3.0 N/mm²

Required fire resistance class: REI 90 (fire-resistant) Bulk

density class 0,80
Wall thickness t = 0.365 m

Support depth a = 0.245 m $\frac{a}{t}$ = 0.67 > 0.45 = min t

N_{Ed} = 259 kN/m
$N_{Rd,fi}$ = 1.176 * 319 = 375 kN/m

Ratio $Q_{k,1}$ / G_k = 0.5; from Fig. 3 follows η_{fi} = 0.6

$$\alpha_{fi} = \frac{N_{Ed,fi}}{N_{Rd,fi}} = \frac{0,6 \cdot 259}{375} = 0,41$$

Required minimum wall thickness for a load-bearing, room-enclosing wall (one-sided fire load) REI90 according to abZ: 300 mm with max. $\alpha_{fi} \leq 0.59$

Available t = 365 mm with α_{fi} = 0.41

Verification: 1. t_{vorh} = 365 mm > min t (F 90) = 300 mm
2. ante. α_{fi} = 0.41 < max. α_{fi} = 0.59

Proof fulfilled!

7.4 Tips for cleaning

The "successors" to the light plasters already rated in DIN 4102-4, section 4.5.2.10 according to DIN 18550-4 or gypsum-containing plasters (mortar group P IV) according to DIN 18550-2 are also named in DIN EN 1996-1-2 as plasters effective in terms of fire protection.

According to DIN EN 1996-1-2, gypsum plaster mortar according to DIN EN 13279-1 or light plaster mortar LW or T according to DIN EN 998-1 are effective in terms of fire protection.

On brick masonry, good results were also achieved in current tests with GP CS II interior lime plasters according to DIN EN 998-1.

8 Simplified verification of external basement walls

8.1 General

According to DIN EN 1996-3/NA, a more precise calculated verification of earth pressure can be omitted for exterior basement walls if the following conditions are fulfilled and the design value of the wall standard force is within certain limits:

- Wall thickness $t \geq 240$ mm
- Clear height of the basement wall $h \leq 2.60$ m
- Basement ceiling acts as a disc and can absorb the forces arising from earth pressure.
- In the area of influence of the earth pressure on the basement wall, the characteristic value q_k of the live load on the ground surface does not exceed 5 kN/m².
- Terrain surface does not rise
- Backfill height h_e is not greater than 1.15 - h
- No concentrated load greater than 15 kN is present at a distance of less than 1.5 m from the basement wall.
- No hydrostatic pressure present (e.g. due to pressing water)
- The horizontal waterproofing (cross-sectional waterproofing) under the wall consists of sanded bitumen roofing membrane R500 according to DIN EN 13969 in conjunction with DIN V 20000-202, mineral waterproofing slurry according to DIN 18533 or material with at least equivalent friction behaviour.

Furthermore, it must be ensured that only non-cohesive soil according to DIN 1054 [16] and only vibratory plates or rammers with the following properties are used for backfilling and compacting the working space:

- Width of the compactor ≤ 50 cm
- Effective depth ≤ 35 cm
- Weight ≤ 100 kg or centrifugal forces ≤ 15 kN

If all conditions are met, the design value of the respective decisive wall normal force N_{Ed} must lie within the following limits at half the height of the backfill:

$$N_{Ed,max} \leq N_{Rd} = \frac{t \cdot b \cdot f_d}{3} \qquad (47)$$

DIN EN 1996-3:2010, section 4.5 (2), eq. (4.11)

$$N_{Ed,min} \geq N_{lim,d} = \frac{\rho_e \cdot h \cdot h_e^2 \cdot b}{t \cdot \beta} \qquad (48)$$

DIN EN 1996-3:2010, section 4.5 (2), eq. (4.12)

with:

N_{Ed} Design value of the wall standard force from the load case max N or min N at half the backfill height

N_{Rd} upper limit value of the wall normal force

$N_{lim,d}$ lower limit value of the wall normal force

t Wall thickness

b Wall length (wall width)

f_d Design value of the compressive strength of the masonry according to equation (12)

ρ_e Weight of the backfill

h Clear wall height

h_e Height of the backfill

β Coefficient for consideration of a horizontal load-bearing effect

β20	for $b_c \geq 2 - h$ (vertical load transfer only)
$\beta = 60 - 20 - b_c$ / h	for $h < b_c < 2 - h$
$\beta = 40$	for $b_c \leq h$
	b_c horizontal distance between bracing cross walls or other bracing elements

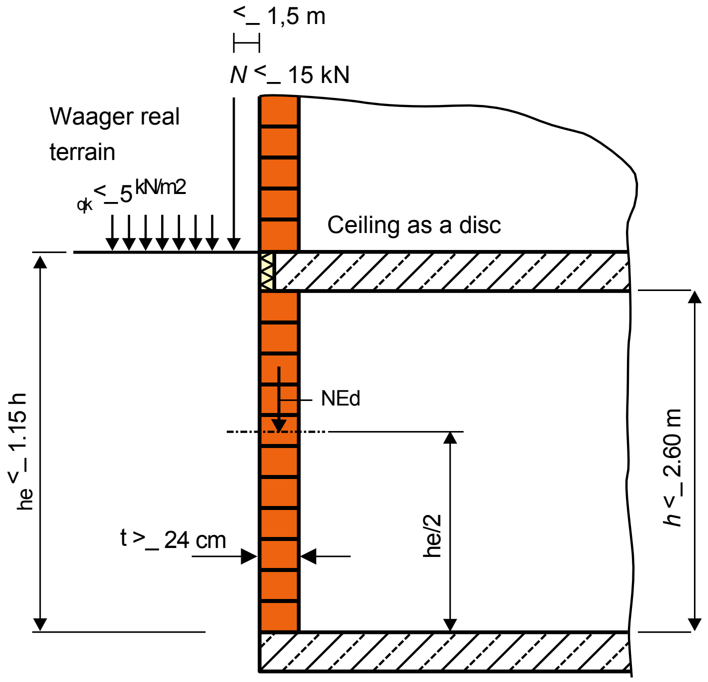


Figure 4: Boundary conditions for the simplified verification of an external basement wall

Table 13 Minimum superimposed load $N_{lim,d}$ in kN/m for exterior basement walls when evaluating equation (48) Boundary conditions: h = 2.5 m, ρ_e = 1800 kg/m³, $b_c \geq 2 - h$ (only vertical load transfer)

Wall thickness *t* [mm]	Height of backfill h_e [m]				
	1,0	**1,5**	**2,0**	**2,5**	**2,875**
240	9	21	38	59	77
300	8	17	30	47	62
365	6	14	25	39	51
425	5	12	21	33	44
490	5	10	18	29	38

Intermediate values are to be interpolated linearly.
It should be noted that the boundary conditions of equation (48) are based on an earth pressure coefficient of 0.33.

8.2 Example

Example 7 **Exterior basement wall**

HLzB 12 with standard masonry mortar M 5 (NM IIa) → f_k = 5.0 N/mm² (see Table 5)

Clear storey height h = 2.50 m
Backfill height Wall h_e = 2.68 m < 2.875 m = 1.15 ·
thickness h t = 0.365 m > 0.24 m
Dense backfill ρ_e = 18 kN/m³
No horizontal load transfer ⟶ β = 20
Live load on terrain q_k = 5.0 kN/m² ≤ 5.0 kN/m²

$N_{Ed,min}$ = 72.5 kN/m
$N_{Ed,max}$ = 121.0 kN/m

$$f_d = \zeta \cdot \frac{f_k}{\gamma_M} = 0.85 \cdot \frac{5.0}{1,5} = 2.83 \text{ N/mm}^2$$

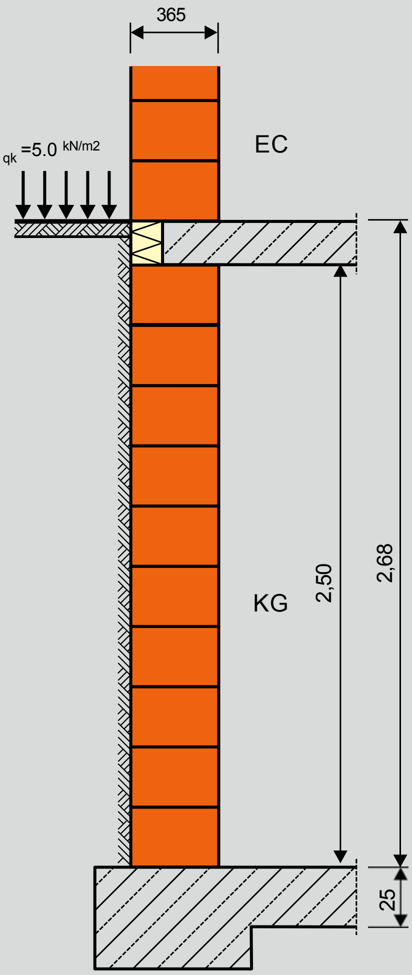
$$N_{Rd} = \frac{t \cdot b \cdot f_d}{3} = \frac{0.365 \cdot 1.0 \cdot 2.83}{3} = 0.344 \text{ MN/m} = 344 \text{ kN/m}$$

$$N_{lim,d} = \frac{\rho_e \cdot h \cdot h_e^2 \cdot b}{t \cdot \beta} = \frac{18 \cdot 2.5 \cdot 2.68^2 \cdot 1.0}{0,365 \cdot 20} = 44 \text{ kN/m}$$

Verification 1: $N_{Ed,max}$ = 121 kN/m < 344 kN/m = N_{Rd}

Verification 2: $N_{Ed,min}$ = 72.5 kN/m > 44 kN/m = $N_{lim,d}$

Proof fulfilled!



9 Non-load-bearing exterior walls

Predominantly wind-loaded non-load-bearing exterior walls (infill areas) can be constructed up to a height of 20 m without a separate structural analysis if

- they are held on four sides (e.g. by interlocking, offset or anchors)
- the planned overbinding dimension $l_{ol} \geq 0.4 \cdot h_u$ (h_u = drawing height) is
- the execution is carried out with standard mortar M 5 (NM IIa), M 10 (NM III), M 20 (NM IIIa) or thin-bed mortar
- they meet the conditions set out in Table 14.

Table 14 Largest permissible values of infill areas in m² of non-load-bearing exterior walls without mathematical verification according to DIN EN 1996-3/NA, Table NA.C.1

Wall thickness [mm]	Height above ground			
	0 to 8 m		8 to 20 m[1]	
	Aspect ratio[2]		Aspect ratio[2]	
	$h_i / l_i = 1.0$	$h_i / l_i \geq 2.0$ or $h_i / l_i \leq 0.5$	$h_i / l_i = 1.0$	$h_i / l_i \geq 2.0$ or $h_i / l_i \leq 0.5$
115	12 (16)[3]	8 (10,6)[3]	-	-
150			8 (10,6)[3]	5 (6,3)[3]
175	20	14	13	9
240	36	25	23	16
≥ 300	50	33	35	23

1) In wind load zone 4, the specified values for heights between 8 and 20 m are only permitted inland.
2) h_i = height of the infill area; l_i = length of the infill area; intermediate values may be straight-line interpolated
3) Values in brackets apply to bricks of strength classes ≥ 12

For non-load-bearing internal partition walls that are not subjected to loads at right angles to the wall surface, DIN 4103-1 [17] is authoritative.

10 Execution of brick masonry according to DIN EN 1996-2/NA and DIN EN 1996-1-1/NA

10.1 General

The design rules for masonry are specified in the National Annexes of the DIN EN 1996 series of standards, among others. Design-relevant requirements are contained primarily in section 8 "Structural design" of DIN EN 1996-1-1/NA, general design rules in DIN EN 1996-2/NA. The most important aspects are briefly summarised below.

Unless greater thicknesses are required for reasons of stability, building physics or fire protection, the minimum wall thickness for load-bearing masonry is:

$$t_{min} = 115 \text{ mm} \qquad (49)$$

DIN EN 1996-1-1/NA:2019, NDP to 8.1.2 (2)

For the planned overbond dimension l_{ol} , the previously known rule continues to apply for common masonry units with layer heights h_u to 249 mm:

$$l_{ol} \geq 0.4 \cdot h_u \geq 45 \text{ mm} \qquad (50)$$

DIN EN 1996-1-1/NA:2019, NCI to 8.1.4.1

When using normal masonry mortar and light masonry mortar, the bearing joint thickness should normally be 12 mm.

For masonry with thin-bed mortar, the following information from the leaflet 'Masonry with thin-bed mortar' [18] applies. According to this, covering thin-bed mortars are particularly suitable for a fully jointing application and guarantee a continuous mortar band. The fresh mortar is applied to the bearing surface of the bricks in a thickness of 1 to 3 mm. The resulting joint thickness on the finished masonry is less than the application thickness. The joint made in this way ensures a force-fit bond.

In Germany, bricks with groove-and-groove systems for processing without butt-joint mortar are predominantly offered. These bricks are to be laid crisply. For butt joint widths > 5 mm, the joints must be sealed with a suitable mortar on both sides of the wall surface when laying bricks [compare DIN EN 1996-1-1/NA, NCI to 8.1.5 (NA.6)].

The maximum horizontal distance between expansion joints in non-load-bearing masonry (e.g. facing shells) is set at 12 m for brick masonry [according to DIN EN 1996-2:2010, section 2.3.4.2 (2), Note: The maximum horizontal distance between expansion joints in non-load-bearing masonry (e.g. facing shells) is 12 m.
kung 1].

10.2 Formation of the wall-ceiling junction for monolithic brickwork

In DIN EN 1996/NA, explicit instructions are given on how the partial support of slabs on monolithic exterior walls is to be taken into account in the design.

The minimum support depth of slabs a_{min} is according to DIN EN 1996-1-1/NA:

$$a_{min} = \min \begin{cases} \frac{t}{3} + 40 \text{ mm} \\ 100 \text{ mm} \end{cases} \qquad (51)$$

DIN EN 1996-1-1/NA:2019, NCI to 8.5.1.1 (NA.7)

For a 365 mm thick exterior wall, this means:

$$a_{min} = \min \begin{cases} \frac{t}{3} + 40 \text{ mm} \\ 100 \text{ mm} \end{cases}$$

$$\Longrightarrow a_{min} = \min \begin{cases} \frac{365 \text{ mm}}{3} + 40 \text{ mm} \\ 100 \text{ mm} \end{cases} = 162 \text{ mm}$$

$$\Longrightarrow a_{min} = 162 \text{ mm}$$

When using the simplified calculation methods according to DIN EN 1996-3/NA, the minimum ceiling support depth for masonry is basically:

$$a_{min} = \min \begin{cases} 0{,}5 \cdot t \\ 100 \text{ mm} \end{cases} \quad (52)$$

DIN EN 1996-3/NA:2019, NCI to 4.2.1.1 (NA.8)

The following applies to masonry with a wall thickness t = 365 mm:

$$a_{min} = 0.45 \cdot t \quad (53)$$

DIN EN 1996-3/NA:2019, NCI to 4.2.1.1 (NA.8)

When applying the highly simplified verification for unreinforced masonry walls in buildings with a maximum of three storeys according to DIN EN 1996-3/NA, NCI to Annex A, a minimum wall thickness $t \geq$ 365 mm applies for partially supported slabs. The following applies to the minimum slab bearing depth:

$$a_{min} = \min \begin{cases} \frac{2}{3} \cdot t \\ 85 \text{ mm} \end{cases} \quad (54)$$

DIN EN 1996-3:2010, Annex A, A.1 (1)

In general, it is recommended to use the largest possible ceiling support depths a/t *in* terms of statics, sound insulation and fire protection. In [19], the structural, static and constructional aspects of the detail "external wall-slab junction" are analysed and a related slab support depth of

$$a_{min} \geq \frac{2 \cdot t}{3} \quad (55)$$

recommended.

According to DIN 4108, Supplement 2:2019-06 [20], larger ceiling support depths are also permissible for monolithic exterior walls with a thermal conductivity $\lambda \leq$ 0.14 [W/(m - K)]. The prerequisite is that a front insulation of the thermal conductivity of $\lambda \leq$ 0.035 [W/(m - K)] of at least 50 mm remains. For example, for an exterior wall with a thickness of a = 365 mm, the ceiling support can be increased to 285 mm. This corresponds to a ratio of a/t = 0.78.

With the details shown in Fig. 5, all the requirements placed on the external wall-ceiling junction can be met without any problems. To ensure a homogeneous plaster base, a brick shell can be placed on the outside (Fig. 5, right). If a brick shell is not used, the material change in the plaster base must be taken into account during the plastering work in accordance with the generally recognised rules of technology (see e.g. [21], [22]).

It must be ensured that the face insulation can compensate for any shortening of the reinforced concrete slab due to creep and shrinkage. The fresh concrete must be prevented from bonding with the face insulation by design.

The insertion of a sanded bitumen membrane R500 decouples deformations of the reinforced concrete ceiling from the outer wall.

Similar positive experiences have been made regionally with a mortar levelling layer at the wall head, on which in-situ concrete or prefabricated slabs can then be laid after sufficient hardening.

However, the separating layer must not act as a sliding bearing, as otherwise the stiffening by the ceiling disc is no longer guaranteed.

Investigations on the required surcharge load at the wall head, above which an effect of bituminous sheeting as a sliding bearing is not to be considered, were carried out in [23].

According to [23], a minimum load of 10 kN/m is sufficient to prevent sliding on the R500 bituminous sheeting.

If these superimposed loads are not available, a ring anchor, preferably made of brick WU shells, must be arranged at the wall head.

For large ceiling spans or low superimposed loads on the top floors, a load-free strip (soft edge strip on the room-side ceiling support edge) reduces the eccentricity and edge pressure. A 30 mm to 50 mm wide soft felt strip on the inner wall edge to avoid stress peaks and improve the position of the load resultants is recommended for large ceiling spans over 6 m (see also [10]).

Figure 5 below shows the position of a load break strip.

For technical reasons, it is also recommended that a bituminous membrane is placed above the reinforced concrete slab to avoid different stiffening of the anchor mortar.

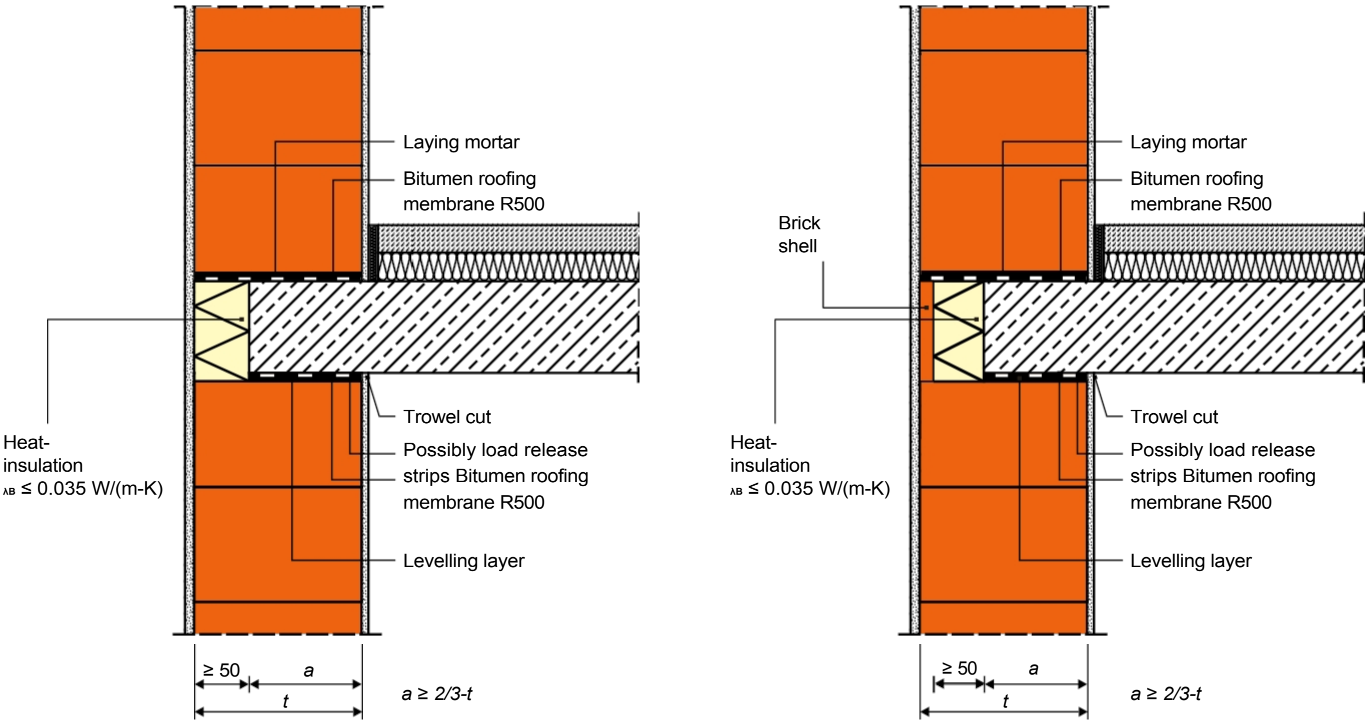


Figure 5: Design variants of an external wall-slab junction with monolithic brick masonry; on the left, slab face insulation, on the right, brick shell with additional thermal insulation.

10.3 Slots and recesses

Slots and recesses in masonry walls are basically differentiated according to their direction of travel (vertical, horizontal or oblique).

According to DIN EN 1996-1-1/NA, NCI to 8.6.2 and NDP to
8.6.2 (1), slots and recesses in load-bearing masonry walls are permitted if they do not endanger the stability of the walls.

Slots and recesses that exceed the limit values specified in Table 15 or Table 16 (from DIN EN 1996-1-1/NA 2019-12, Tables NA.20 and NA.20).
NA.21) may be neglected in the design. If the distances and dimensions of the slots and recesses exceed the values given in the tables, these shall be taken into account in the measurement of the masonry walls by reducing the cross-sectional values.

Some important boundary conditions for the arrangement of slots and recesses are given in Figure 6 and Figure 7. If the weakening of the cross-section of the wall in plan view due to a vertical slot is not more than 6 % related to 1 m wall length, a verification of the weakening may be omitted. However, this only applies if the wall under consideration was not designed as a three- or four-sided retained wall. In addition, the residual wall thicknesses and the minimum distances according to Table 16 must be observed.

General technical approvals (abZ) or general type approvals (aBG) can contain further regulations.

Table 15 Vertical slots and recesses permitted without verification **in masonry bond** according to [7].

<table>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><th rowspan="3">Wall thickness
t
[mm]</th><th colspan="4">Vertical slots and recesses in masonry bond</th></tr>
<tr><th rowspan="2">Slot width[1)] [mm]</th><th rowspan="2">Residual wall thickness [mm]</th><th colspan="2">Minimum spacing of slots and recesses</th></tr>
<tr><th>from openings</th><th>among each other</th></tr>
<tr><td>**115 - 174**</td><td>-</td><td></td><td rowspan="6">≥ two times the slot width or ≥ 240 mm</td><td rowspan="6">≥ slot width</td></tr>
<tr><td>**175 - 199**</td><td>≤ 260</td><td rowspan="3">≥ 115</td></tr>
<tr><td>**200 - 239**</td><td>≤ 300</td></tr>
<tr><td>**240 - 299**</td><td rowspan="3">≤ 385</td></tr>
<tr><td>**300 - 364**</td><td>≥ 175</td></tr>
<tr><td>**≥ 365**</td><td>≥ 240</td></tr>
<tr><td colspan="5">**[1)] The total width of slots according to column 2 must not exceed the dimensions in column 2 per 2 m wall length. For wall lengths less than 2 m, the values in column 2 shall be reduced in proportion to the wall length.**</td></tr>
</table>

Table 16 Slots and recesses in load-bearing walls permitted **without** verification according to DIN EN 1996-1-1/NA:2019 [7] Tables NA.20 and NA.21

<table>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th><th>6</th></tr>
<tr><th rowspan="4">Wall thickness
t
[mm]</th><th colspan="2">Horizontal and oblique slots[1]</th><th colspan="3">Vertical slots and recesses</th></tr>
<tr><th colspan="2">Slot length</th><th rowspan="3">Slot depth[4]
[mm]</th><th rowspan="3">Single slot width[5] [mm]</th><th rowspan="3">Spacing of slots and recesses of openings [mm]</th></tr>
<tr><th>unrestricted</th><th>≤ 1,25 m[2]</th></tr>
<tr><th>Slot depth
[mm]</th><th>Slot depth
[mm]</th></tr>
<tr><td>**115 - 149**</td><td>-</td><td>-</td><td>≤ 10</td><td rowspan="3">≤ 100</td><td rowspan="6">≥ 115</td></tr>
<tr><td>**150 - 174**</td><td>-</td><td>0 [3]</td><td>≤ 20</td></tr>
<tr><td>**175 - 199**</td><td>0 [3]</td><td rowspan="3">≤ 25</td><td rowspan="4">≤ 30</td></tr>
<tr><td>**200 - 239**</td><td>0 [3]</td><td>≤ 125</td></tr>
<tr><td>**240 - 299**</td><td>≤ 15 [3]</td><td>≤ 150</td></tr>
<tr><td>**≥ 300**</td><td>≤ 20 [3]</td><td>≤ 30</td><td>≤ 200</td></tr>
</table>

1) Horizontal and oblique slots are only permissible in an area ≤ 0.4 m above or below the raw ceiling and on one wall side each. They are not permissible with long-hole bricks.
2) Minimum longitudinal distance from openings ≥ 490 mm, from the nearest horizontal slot twice the slot length.
3) The depth may be increased by 10 mm if tools are used with which the depth can be precisely maintained. When using such tools, opposite slots with a depth of 10 mm each may also be made in walls ≥ 240 mm.
4) Slits that extend to a maximum of 1 m above floor level may be used for wall thicknesses of ≥ 240 mm to 80 mm depth and 120 mm width.
5) The total width of slots according to column 5 and column 2 of Table 15 shall not exceed the dimensions in column 2 of Table 15 per 2 m wall length. For wall lengths less than 2 m,the values in column 2 of Table 12 shall be reduced in proportion to the wall length.

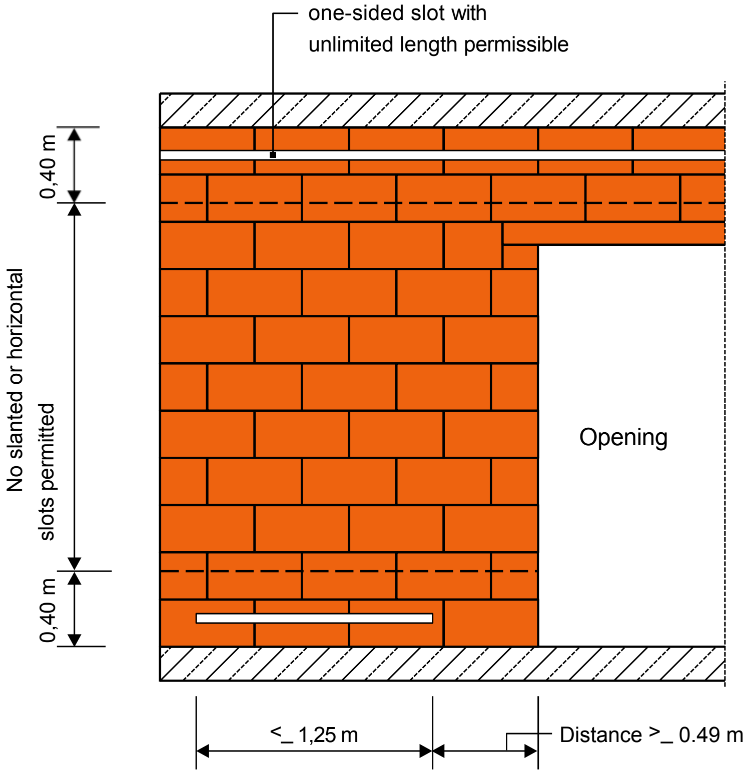


Figure 6: Permissible horizontal slots and recesses without mathematical verification

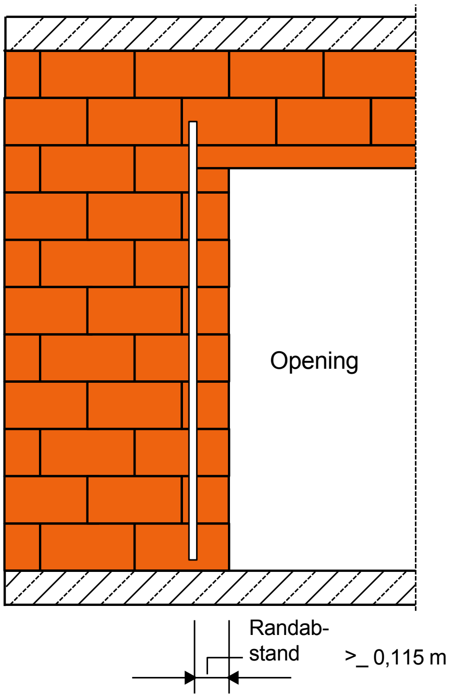


Figure 7: Permissible vertical slots and recesses without mathematical verification, see Table 15 and Table 16.

11 Literature

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A.1 Building description and geometry

The calculations presented below are carried out for a multi-family house. It is a four-storey building with a basement and a hipped roof. The roof construction is made of wood. All floors are made of brick walls with reinforced concrete ceilings. The reinforced concrete ceilings act as stiffening ceiling slabs. For the plastered, single-shell exterior walls, thermally insulating plane bricks with thin-bed mortar are used. The wall materials for

The intermediate walls, the non-load-bearing walls and the outer basement walls are also made of vertically perforated bricks with thin-bed mortar. The wall joints are made using the butt joint technique with wall connectors. The partition walls to the staircase are made of sound-insulating bricks (backfill bricks). This also applies to the partition walls between the residential units. Snow load zone 1 and wind load zone 2 are used for the calculation.

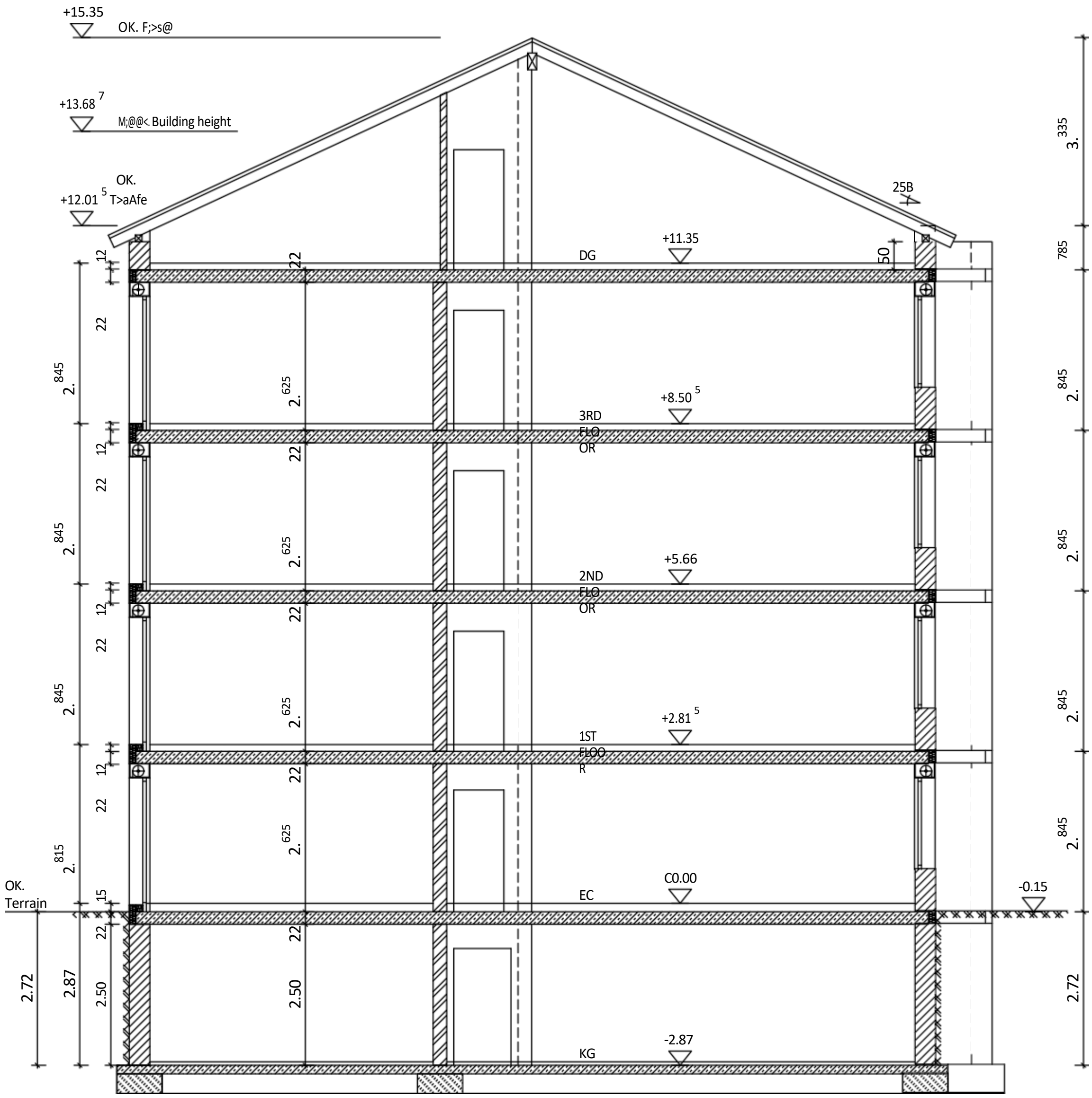


Figure A.1: Section through the apartment building

The reinforced concrete slabs have a thickness of 220 mm. These slabs rest on the exterior walls with a bearing depth of 245 mm.

In the area of the slab supports, a sanded bitumen roofing membrane R500 according to DIN EN 13969 is used on the underside and on the top side.

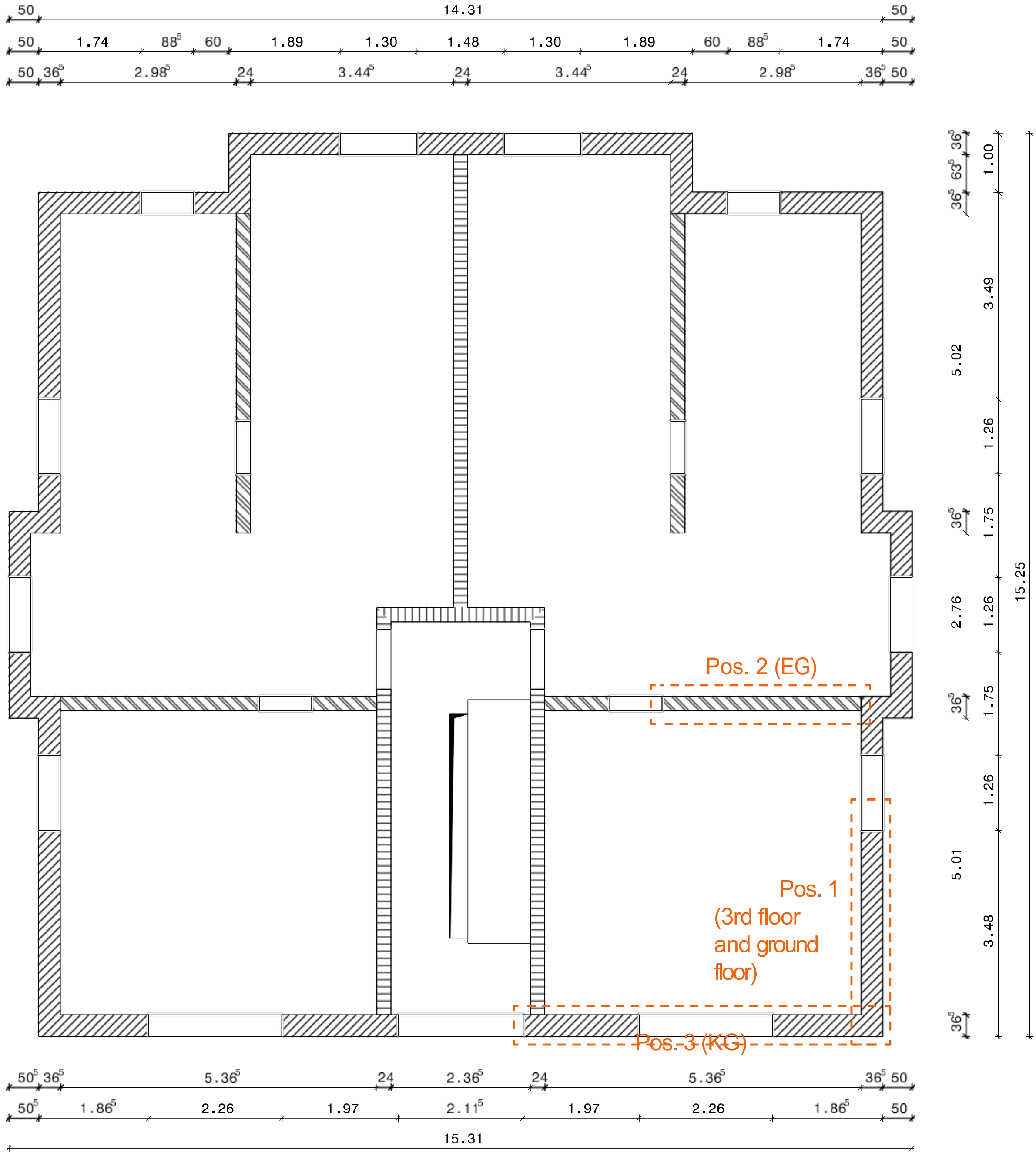


Figure A.2: Floor plan of the ground floor to the 3rd floor

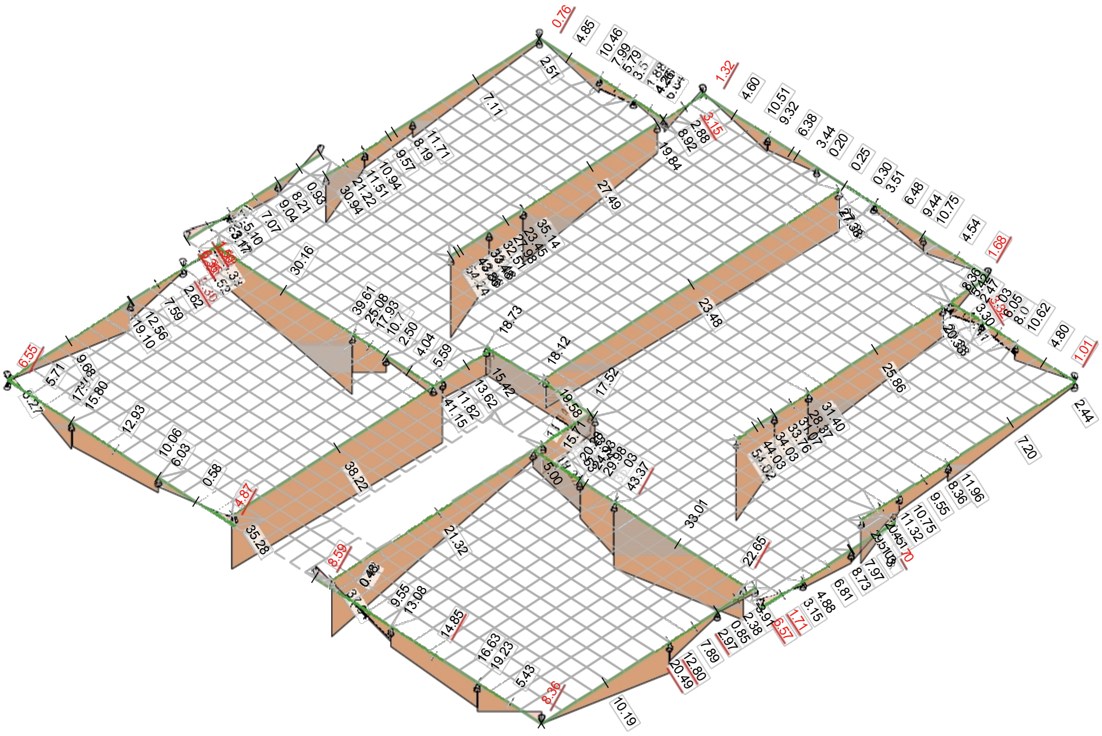


Figure A.3: Linearised support forces of the ceilings on the ground floor - 3rd floor from permanent loads (LF *g*)

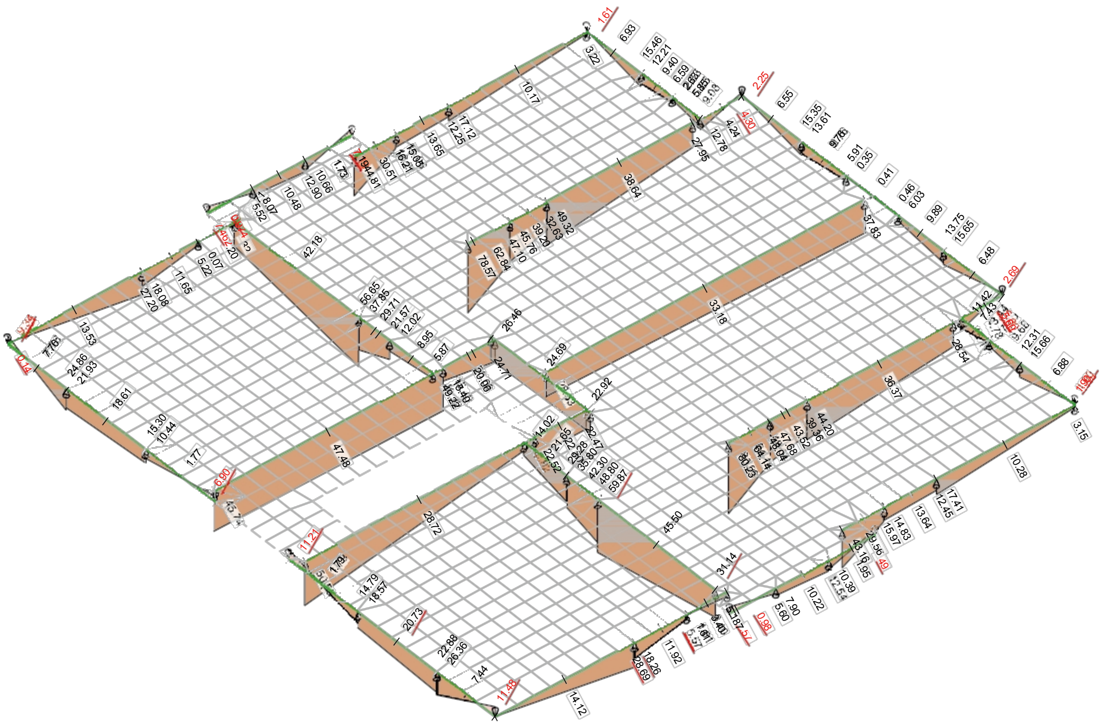
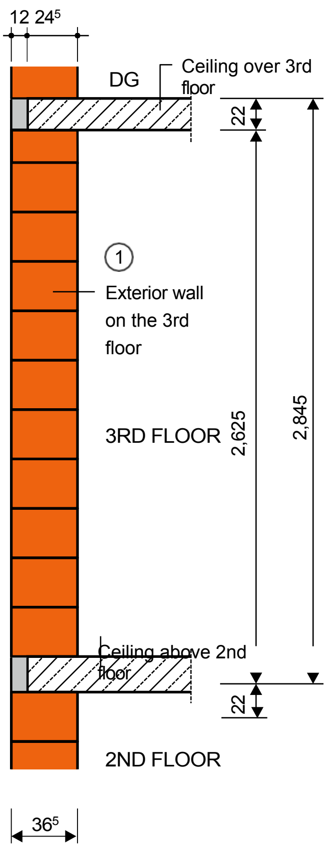


Figure A.4: Linearised max. support forces of the ceilings on the ground floor - 3rd floor from the load cases *g* + *q*

A.2 Verification of exterior and interior walls (ground floor - 3rd floor)

A.2.1 Pos. 1: Load-bearing outer wall

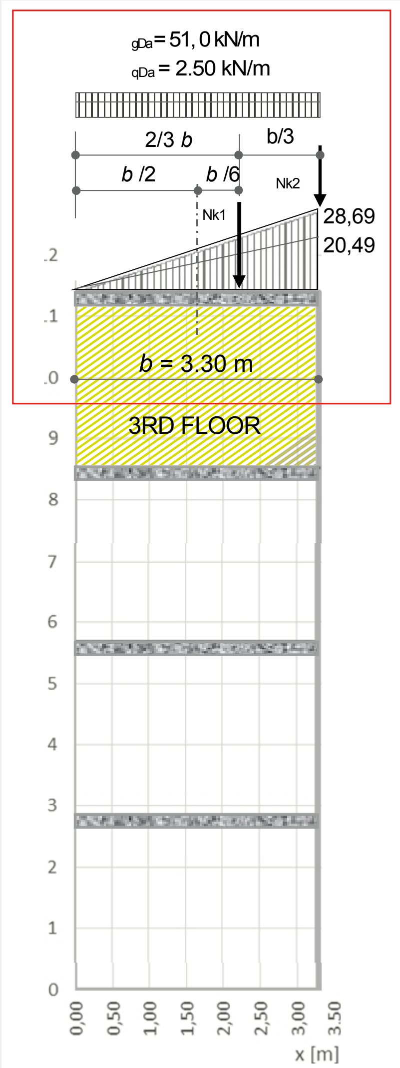
A.2.1.1 Item 1a: Load-bearing exterior wall on 3rd floor



Static system

Double-sided, single-shell exterior wall a end support

Figure A.5: Section through the outer wall



b to wall corner axis:
b = 3.48 - 0.365/2 = 3.30 m

Table A.1	Component data
Max. Building height	15,35 m
Brick compressive strength class	12
Brick density class	0,75
Masonry mortar	Thin-bed mortar
Wall thickness *t*	365 mm
Plaster (exterior / interior)	Lightweight plaster / gypsum plaster
Wall length *b*	3,300 m
Clear wall height *h*	2,625 m
Ceiling thickness $_{db}$	220 mm
Ceiling support width l_1	5,34 m
Ceiling support depth *a*	245 mm

Table A.2	Load composition wall on the 3rd floor	
Roof loads and jambs (from secondary calculation)	Permanent load g_{Da}	5.10 kN/m
	Variable load q_{Da}	2.50 kN/m
Ceiling loads	gConcrete	5.50 kN/m²
	gPlaster/covering	1.80 kN/m²
	Permanent load $\sum g_{De}$	7.30 kN/m²
	Payload category A2	1.50 kN/m²
	Partition wall surcharge	1.20 kN/m²
	Variable load $\sum q_{De}$	2.70 kN/m²
Dead load wall	γ_W = 0.75 - 10 + 1	8.50 kN/m³
	gWand	3.10 kN/m²
	gPutz	0.43 kN/m²
	Permanent load $\sum g_{Wand}$	3.53 kN/m²

Load at the wall head on the 3rd floor

Determination of equivalent block load from eccentric ceiling load:

g = Φ1 - 2 - 0.80 / 3.30 = 0.52 (according to equation (37))

gDe = $N_{g,k}$ /(Φ_g - b) = 43,6/(0,52 - 3,30) = 25.4 kN/m

gDe = $N_{q,k}$ /(Φ_q - b) = 16,6/(0,55 - 3,30) = 9.1 kN/m

Total loads at the wall head:

g_1 = g_{Da} + g_{De} = 5.1 + 25.4 = 30.5 kN/m

q_1 = q_{Da} + q_D = 2.5 + 9.1 = 11.6 kN/m

Internal forces

Normal force: $n_{Ed,j}$ = 1.35 - (g_1 + g_{Wand} - h) + 1.5 - q_1

Wall head: $nEd_{,o}$ = 1,35 - 30,5 + 1,5 - 11,6 = 58.6 kN/m

Centre of wall: $n_{Ed,m}$ = 1.35 - 2.625/2 - 3.53 + 58.6 = 64.9 kN/m

Wall base: $nEd_{,u}$ = 1.35 - 2.625 - 3.53 + 58.6 = 71.1 kN/m

gConcrete = d_b - γ_B
= 0,22 - 25
= 5.5 kN/m²

DIN EN 1991-1-1, Annex A Living spaces (A) with adequate transverse distribution DIN EN 1991-1-1, 6.3

gWand = t - γ_W
= 0,365 - 8,5
= 3.10 kN/m²

gPutz = 0,25 + 0,18
= 0.43 kN/m²

from line storage results of the FE calculation:

Wall area:

$Ng_{,k1}$ = 20,49 / 2 - 3,30
= 33.8 kN

$Nq_{,k1}$ = (28,69 - 20,49) / 2 - 3,30
= 13.53 kN

Window lintel area:

$Ng_{,k2}$ = ((12,8 + 2,97) / 2 + 5,1) - 1,26 - 0,6
= 9.82 kN

$Nq_{,k2}$ = ((18,26 + 5,57) / 2 + 2,5) -1,26 - 0,6 - 9,82
= 3.05 kN

$Ng_{,k}$ = 33.8+9.82 = 43.6 kN

$Nq_{,k}$ = 13.53+3.05 = 16.6 kN

$Mg_{,k}$ = 33,8-3,30/6 + 9,82-3,30/2
= 34.8 kNm

$Mq_{,k}$ = 13,53-3,30/6 +3,05-3,30/2
= 12.5 kNm

With e = M / N

$e_{g,k}$ = 34,8 / 43,6 = 0,80

$e_{q,k}$ = 12,5 / 16,6 = 0,75

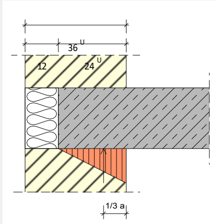
Table A.3	Review of the conditions for applying the simplified calculation methods		
Criterion	**Request**	**Actual value**	**Comment**
Maximum building height	$H \leq 20$ m	15,35 m	complied with
Maximum ceiling support width	$l \leq 6$ m	5,34 m	complied with
Maximum clear wall height	$h \leq 12 \cdot t$ = 4,38 m	2,625 m	complied with
Maximum traffic load on ceilings	$q_k \leq 5$ kN/m²	2.7 kN/m²	complied with
Minimum bearing depth	$a \geq 0.45 \cdot t$ = 164 mm	245 mm	complied with

Compare section 4

according to DIN EN 1996-3/NA, NCI to
4.2.1.1: for biaxially tensioned slabs, the shorter of the two spans:

clear ceiling span:

l_i = 5.01 + 0.365 - 0.24
= 5,135 m



Ceiling support width:

l = 0,245/3 + 5,135 + 0,24/2
= 5,34 m

(with depth of support on the outer
wall = 0.245 m)

N_{Rd} according to equation (11)

Verification of the normal force bearing capacity

Rated value N_{Rd} of the resistor
$N_{Rd} = \Phi - A -$
wit f_d
A Wall surface
A = 1.00 m - 0.365 m = 0.365 m²

Reduction factor Φ_1 from ceiling torsion

$\Phi_{1,o}$ according to equation (16) for two-axis tensioned plates

Wall head: $\Phi_{1,o} = 0.4 \cdot \frac{a}{t} = 0.4 \cdot \frac{245}{365} = 0.268$

Wall base: $\Phi_{1,u} = \left(1.6 - \frac{l_f}{6}\right)\frac{a}{t} = \left(1.6 - \frac{0.85 - 5.34}{6}\right)\frac{245}{365} = 0,566$

or $= 0.9 \cdot \frac{a}{t} = 0.9 \cdot \frac{245}{365} = 0,604$

The smaller value is decisive: $\Phi_{1,u}$ = 0,566

$\Phi_{1,u}$ according to equation (14)
l_f = 0.85 - shorter span of the biaxially tensioned slab

with the factor a / t the partial support on the floor slab is taken into account.

Reduction factor Φ_2 for buckling

Buckling length factor for t > 250 mm: ρ_2 = 1.0
Buckling length: $h_{ef} = \rho_2 \cdot h$ = 1.00 - 2.625 = 2.625 m
Slenderness: h_{ef} / t = 2.625 / 0.365 = 7.2 < 27 = perm h_{ef} / t

For the calculation of the buckling length only a two-sided wall bearing is considered, see equation (20).

$\Phi_2 = 0.85 - \frac{a}{t} -- 0.0011 \left(\frac{h_{ef}}{t}\right)^2$

$= 0,85 - \frac{245}{365} - 0,0011 - 7,2^2 = 0,513$

Φ_2 according to equation (19)

Dimensioning

w.: Masonry from vertically perforated flat brick according to Building authority approval (abZ)

Stone strength class: 12
Mortar: DM (thin bed mortar)

Value of the characteristic compressive strength (according to abZ):
$f_k = 3.0$ MN/m²

Design value of the compressive strength:

$f_d = 0.85 - \frac{3.0}{1.5} = 1.70$ MN/m²

According to equation (12) with $\gamma_M = 1.5$

Rated resistors:

Wall head
$n_{Rd,o} = \Phi_{1,o} - A - f_d$ $= 0.268 - 0.365 - 1.70 - 1000$
$= 166.3$ kN/m $> n_{Ed,o} = 58.6$ kN/m

According to equation (11)
The eccentricity of the normal force in the longitudinal direction of the wall was already taken into account in the approach to the ceiling loads.

Wall centre
$n_{Rd,m} = \Phi_2 - A - f_d$ $= 0.513 - 0.365 - 1.70 - 1000$
$= 318.3$ kN/m $> n_{Ed,m} = 64.9$ kN/m

Wall base
$n_{Rd,u} = \Phi_{1,u} - A - f_d$ $= 0.566 - 0.365 - 1.70 - 1000$
$= 351.2$ kN/m $> n_{Ed,u} = 71.1$ kN/m

Verification of the minimum superimposed load:

For exterior walls that serve as end supports for ceilings or roofs, a verification of the minimum superimposed load must be carried out according to equation (22).

Wind range D: $cp_{,10} = 0,8$

Wind zone 2: q_w = 0.80 kN/m²
$q_{Ewk} = 0,8 - 0,8 = 0.64$ kN/m²
$q_{Ewd} = 1,5 - 0,64 = 0.96$ kN/m²

Wind load q_w according to DIN EN 1991-1-4 for 10 m < $h \leq 18$ m

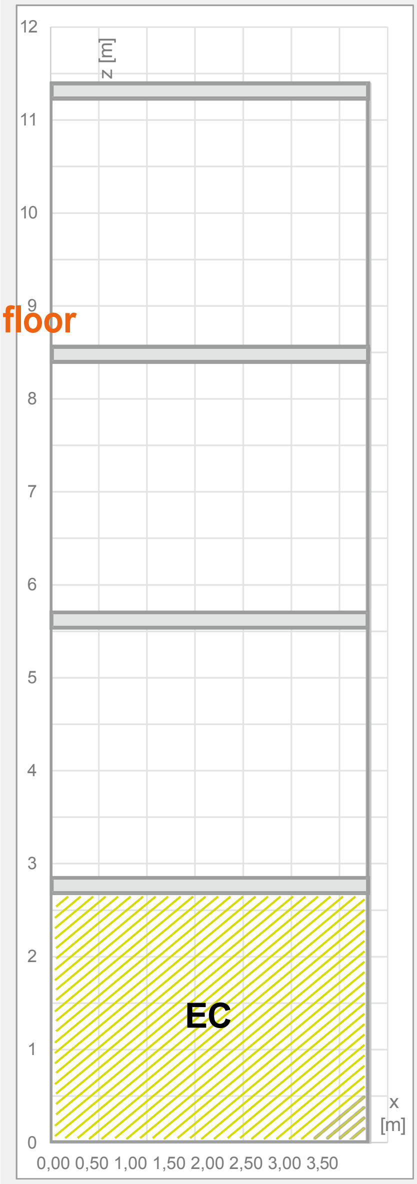
erf $n_{Ed} = 3 - 0.96 - 2.625^2 - 1/[16 - (0.245\text{-}2.625/300)] = 5.25$ kN/m

min $n_{Ed} = 1.0 - (25.3 + 2.625/2 - 3.53) = 29.9$ kN/m > 5.25 kN/m

min $n_{Ed} = 1.0 - (n_{g,k} + n_{Wand}/2)$

Table A.4	Compilation of the decisive values				
Location	Diminishing factors Φ_1 resp. Φ_2	Resistance n_{Rd}	Impact n_{Ed}	$\frac{n_{Ed}}{n_{Rd}}$	Remarks
	-	kN/m	kN/m		
Wall head	0,268	166,3	58,6	0,35	Evidence provided
Wall centre	0,513	318,3	64,9	0,20	Evidence provided
Wall base	0,566	351,2	71,1	0,20	Evidence provided

A.2.1.2 Item 1b: Load-bearing exterior wall on the ground floor



Wall view

Load composition like wall on 3rd floor

Load at the wall head on the ground floor

from attic:
$n_{g,Da}$ = 5.10 kN/m $\quad$ $n_{q,Da}$ = 2.5 kN/m

from truss ceilings incl. loads from lintel:
$n_{g,De} = 4 \cdot g_{De}$ = 4 · 25.4 = 101.6 kN/m
$n_{q,De} = 4 \cdot q_{De}$ = 4 · 9.10 = 36.4 kN/m

from wall loads:
$n_{gw} = 3 \cdot g_w$ = 3 · 2,625 · 3,53 = 27.8 kN/m

Total loads at the wall head:
g_4 = 5.1 + 101.6 + 27.8 = 134.5 kN/m
q_4 = 2.5 + 36.4 = 38.9 kN/m

Internal forces

Normal force $n_{Ed,j} = 1.35 \cdot (g_4 + g_w \cdot h) + 1.5 \cdot q_4$

Action at the wall head
$n_{Ed,o}$ = 1,35 · 134,5 + 1,5 · 38,9 = 239.9 kN/m

Action in the centre of the wall
$n_{Ed,m}$ = 1.35 - 2.625/2 - 3.53 + 239.9 = 246.2 kN/m
Action at the base of the wall
$n_{Ed,u}$ = 1.35 - 2.625 - 3.53 + 239.9 = 252.4 kN/m

Review of the conditions for the application of the simplified calculation methods

Verifications fulfilled, compare wall on 3rd floor

Verification of the normal force bearing capacity

Rated value $N_{Rd} = \Phi - A - f_d$

Reduction factor Φ_1 from ceiling torsion

Wall head:

$$\Phi 1_{,o} = \left(1.6 - \frac{l_f\, a}{6}\right) = 1.6 - \frac{}{t}\left(0.\frac{85 - 5.34}{6}\right)\frac{245}{365} = 0,566$$

Wall base: $\Phi_{1,u} = \Phi 1_{,o}$ = 0,566

resp. $= 0,9 - \frac{a}{t} = 0,9 - \frac{245}{365}$ = 0,604

The smaller value is decisive: $\Phi_{1,u}$ = 0,566

Reduction factor Φ_2 for buckling

Buckling length factor for t > 250 mm: ρ_2 = 1,0
h_{ef} = 2,625 m

h_{ef} / t = 2.625 / 0.365 = 7.2 < 27 = perm h_{ef} / t

$$\Phi_2 = 0.85 - \frac{245}{365} - 0.0011 - 7.2^2 = 0,513$$

The factor a / t takes into account the partial support on the floor slab.

h_{ef} according to equation (20)

Dimensioning according to DIN EN 1996-3

chosen: Masonry made of vertically perforated flat brick according to general building approval (abZ)

Stone strength class: 12
Mortar: DM (thin bed mortar)
Value of the characteristic compressive strength (according to abZ):
f_k = 3.0 MN/m²

Design value of the compressive strength:

$f_d = 0.85 \cdot \frac{3.0}{1.5} = 1.70$ MN/m²

Rated resistors:

$n_{Rd,o} = \Phi_{1,o} \cdot A \cdot f_d = 0.566 \cdot 0.365 \cdot 1.70 \cdot 1000$
$= 351.2$ kN/m $> n_{Ed,o} = 239.9$ kN/m

$n_{Rd,m} = \Phi_2 \cdot A \cdot f_d = 0.513 \cdot 0.365 \cdot 1.70 \cdot 1000$
$= 318.3$ kN/m $> n_{Ed,m} = 246.2$ kN/m

$n_{Rd,u} = \Phi_{1,u} \cdot A \cdot f_d = 0.566 \cdot 0.365 \cdot 1.70 \cdot 1000$
$= 351.2$ kN/m $> n_{Ed,u} = 252.4$ kN/m

According to equation (12)

The eccentricity of the normal force in the longitudinal direction of the wall was already taken into account in the approach to the ceiling loads.

Table A.5 Compilation of the decisive values

Location	Diminishing factors	Resistance	Impact	$\frac{n_{Ed}}{n_{Rd}}$	Remarks
	Φ_1 resp. Φ_2	n_{Rd}	n_{Ed}		
	-	kN/m	kN/m		
Wall head	0,566	351,2	239,9	0,68	Evidence provided
Wall centre	0,513	318,3	246,2	0,77	Evidence provided
Wall base	0,566	351,2	252,4	0,72	Evidence provided

A.2.2 Pos. 2: Load-bearing interior wall on the ground floo

Static system

Two-sided load-bearing interior wall

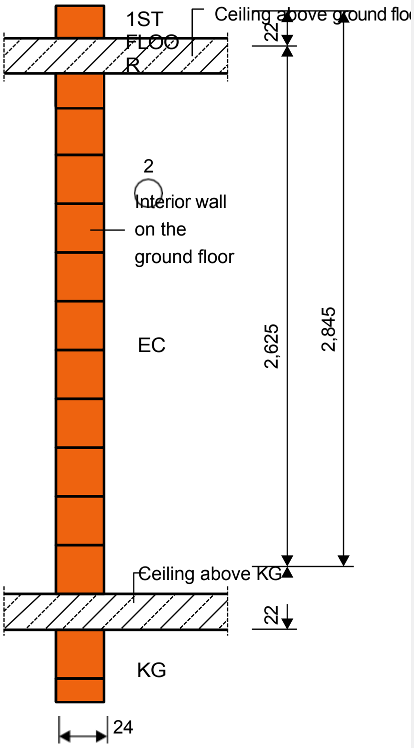


Figure A.6: Section through the load-bearing interior wall on the ground floor

Table A.6 Component data	
Max. Building height	15,35 m
Lithographic strength class	12
Brick density class	1,2
Masonry *mortar*	DM
Wall thickness *t*	240 mm
Wall length *b*	3,375 m
Clear wall height *h*	2,625 m
Ceiling thickness $_{db}$	220 mm
Ceiling support width $l1_{,o} = l1_{,u}$	5,34 m
Ceiling support width $l2_{,o} = l2_{,u}$	3,38 m

$l1_{,o}$ = 0,245/3+5,135+0,24/2
= 5,34 m

$l2_{,o}$ = 0,24/2+2,76+1,00/2
= 3,38 m

For the dimensioning of the wall panels, the occurring loads with their eccentricities must be taken into account. Depending on the floor plan situation, the moments occurring as a result of the load eccentricities can be centred by internal forces. Subsequently, on the basis of FE models were used to investigate whether centring can be used for the wall plates under consideration (here items W4-1 to W4-4) or whether a cantilever model without compensation of the moments should be used.

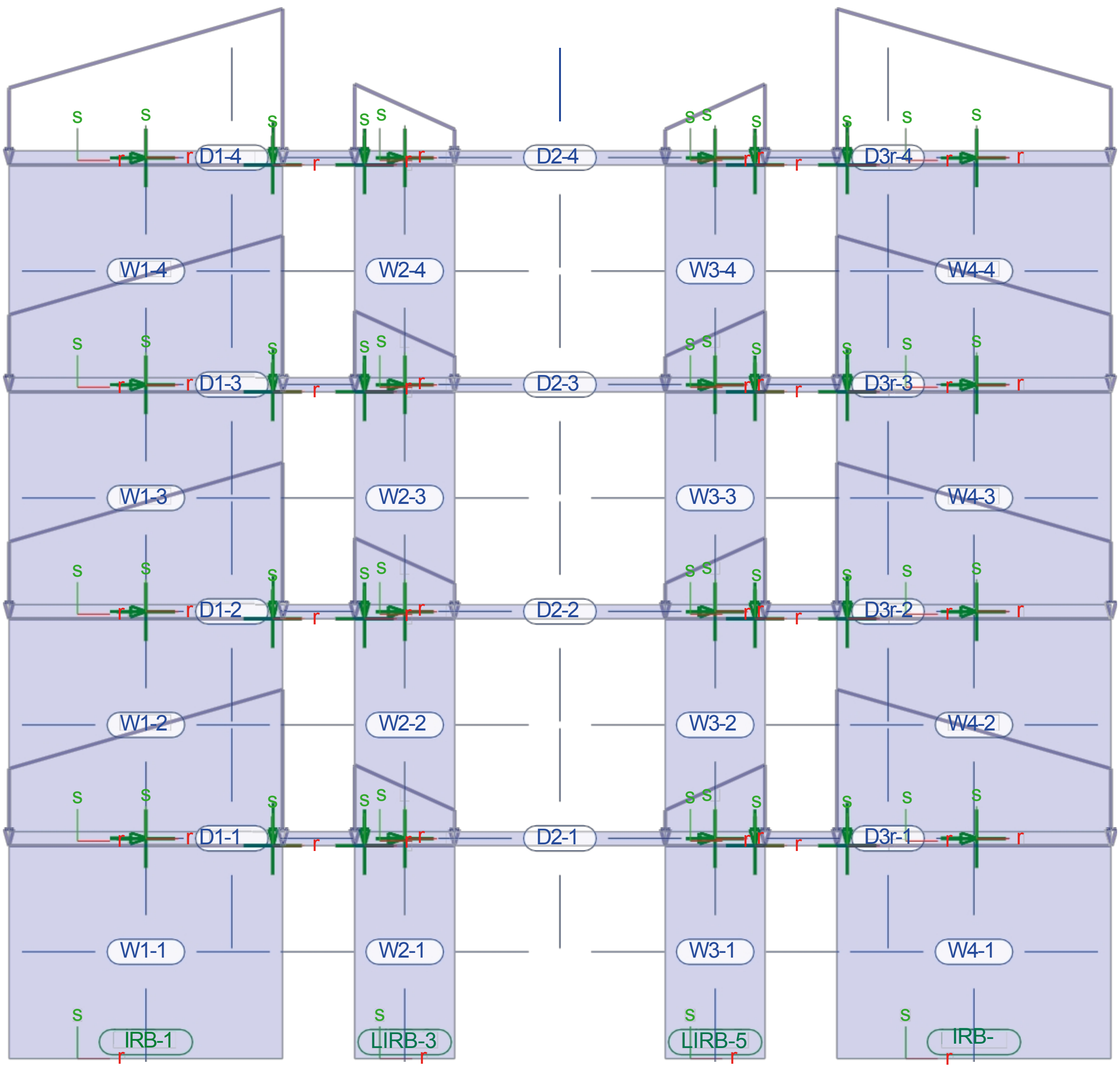


Figure A.7: FE model with position designations and representation of the loads

For the pure cantilever model, the continuous floor slabs were removed from the model. Subsequently, a comparison of the deformation images of the two models is used to examine which image is the best. The structural model of the wall is the one that most closely corresponds to the deformations that are actually to be expected and whose structural model is therefore to be used for the wall slab design.

Marking of the wall panel under consideration (pos. 2)

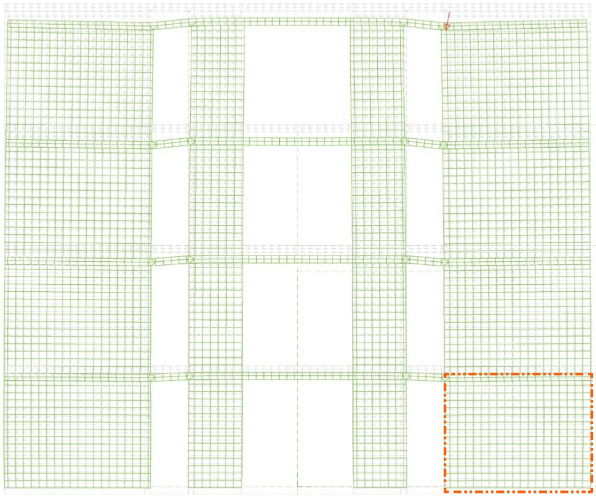


Figure A.8: Deformation pattern FE model with slab slices (framework model),
Load case 1: Vertical loads with eccentricities

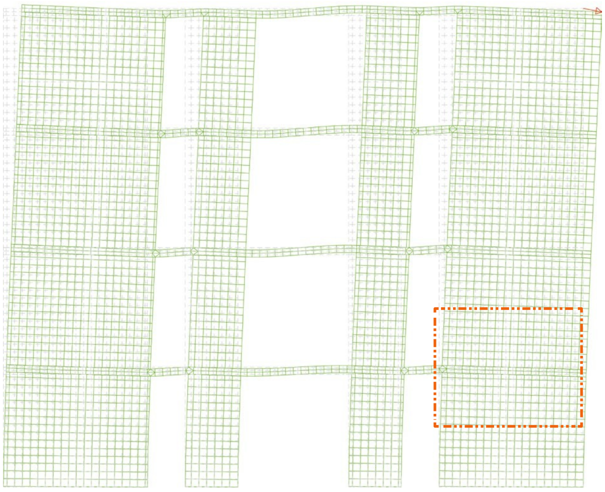


Figure A.9: Deformation pattern FE model with slab slices (framework model),
Load case 2: Horizontal loads (from wind)

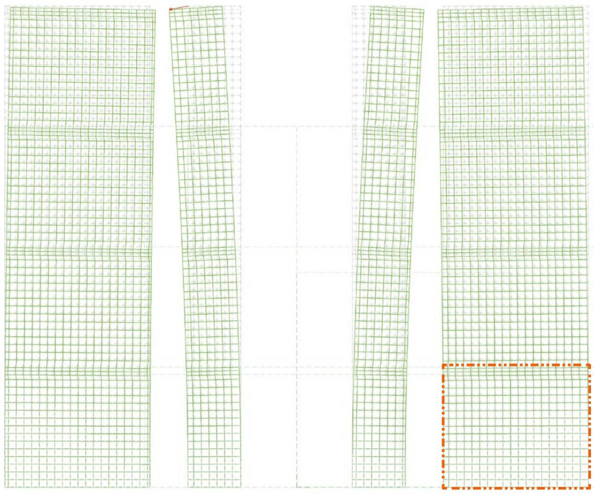


Figure A.10: Deformation pattern FE model without slab slabs (cantilever model),
Load case 1: Vertical loads (with eccentricities)

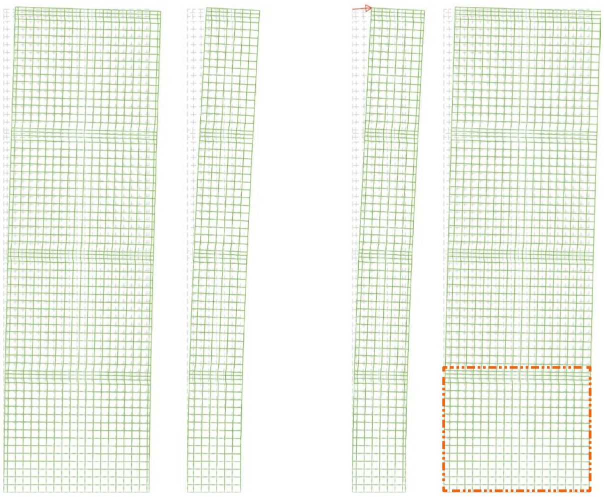


Figure A.11: Deformation pattern FE model without slab slabs (cantilever model)
Load case 2: Horizontal loads (from wind)

The deformation pattern of the frame model (Figure A.8) shows that despite the load eccentricities due to the system symmetry and the slab restraint moments, no significant horizontal displacements occur at the connecting slab slices. Figure A.10, on the other hand, shows that in the pure cantilever model, corresponding horizontal displacements of the wall slabs occur due to the moments resulting from the load eccentricities.

For the load case horizontal loads (here wind load), on the other hand, Figure A.9 (frame model) and Figure A.11 (cantilever model) show similar deformation patterns.

In the following, the support forces at the wall base of the ground floor linearised over the wall length are shown for the two FE models investigated.

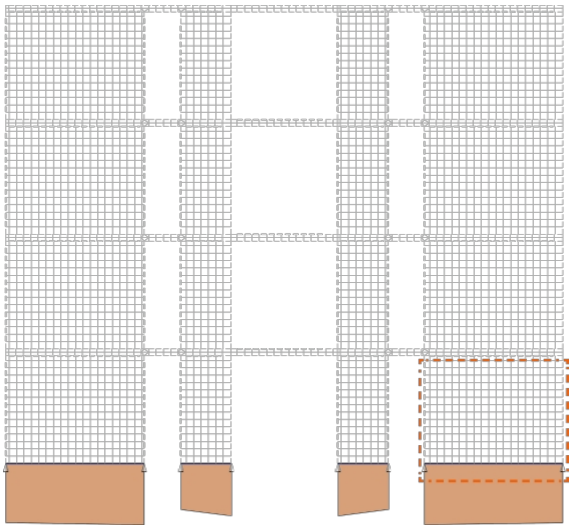


Figure A.12: Bearing forces on the FE model **with** slab shear
(frame model) from LF 1

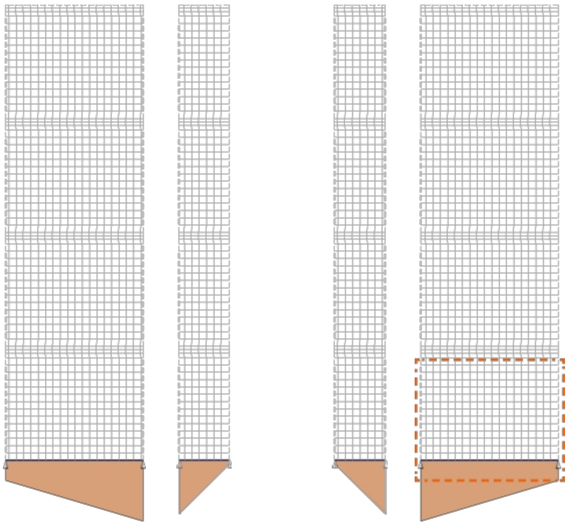


Figure A.13: Supporting forces on the FE model **without** slab discs (cantilever model) from LF 1

Figure A.12 shows that due to the symmetrical ground plan and the connecting floor slabs, the eccentrically acting loads from floor and lintel loads are almost centred up to the base of the wall. Without the load centring, the normal forces from ceiling, lintel and wall loads add up over the wall height as shown in Figure A.13.

Since the deformations shown in Figure A.10 and A.11 cannot occur in reality due to the existing floor slabs, load centring is used for the further calculation. In a simplified way, it is assumed that 90 % of the moments from load eccentricities are centred up to the wall base due to internal forces. These internal forces, in the form of horizontal forces in the wall plates, are to be added to the wind and stabilisation forces as horizontal forces when verifying the shear force bearing capacity. To calculate the internal forces, a model is used in which the floor slabs are connected to the wall slabs by joints (Figure A.14). If a rigid ceiling connection is assumed, additional moments would have to be taken into account in the design of the ceiling panels. This interaction can only be reasonably calculated using 3D finite element models. However, due to the high effort involved, this will only be used in exceptional cases.

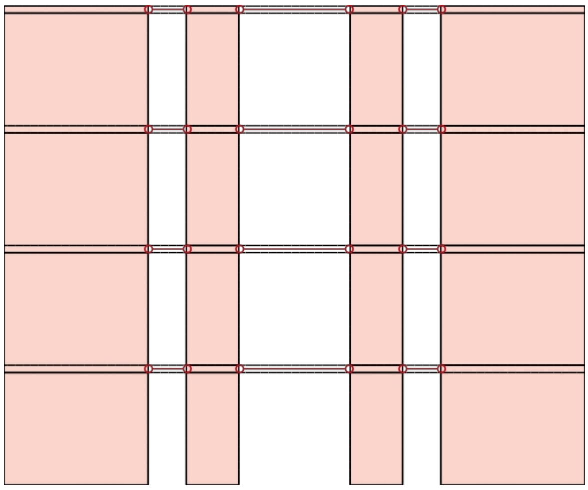


Figure A.14: Disc model with hinged ceiling panels

Caution: Load centring can only be applied if the redistribution of the horizon talk forces resulting from the load eccentricity is possible due to the floor plan situation. For asymmetrical systems, the use of the cantilever model is recommended. Since no additional internal forces arise without load centring, the horizontal forces to be verified are smaller with the cantilever model than with the frame model.

Table A.7	Load composition	
Roof loads	Permanent load g_{Da}	10.00 kN/m
	Variable load q_{Da}	2.50 kN/m
Ceiling loads	gConcrete	5.50 kN/m²
	gPlaster/covering	1.80 kN/m²
	Permanent load $\sum g_{De}$	7.30 kN/m²
	Payload category A2	1.50 kN/m²
	Partition wall surcharge	1.20 kN/m²
	Variable load $\sum q_{De}$	2.70 kN/m²
Dead load wall	γ_W = 1.2 - 10 + 1	13.00 kN/m³
	gMW	3.12 kN/m²
	gPutz	0.36 kN/m²
	Permanent load g_{Wa}	3.48 kN/m²

of inner wall in the attic and roof construction

gConcrete $= d_b - \gamma_B$
= 0,22 - 25
= 5.5 kN/m²

DIN EN 1991-1-1, Annex A

Living spaces (A) with sufficient transverse distribution

DIN EN 1991-1-1, 6.3

Surcharge for thin-bed mortar: 1.0 kN/m³

gMW $= t - \gamma_W$
= 0,24 - 13
= 3.12 kN/m²

gPutz = 0,18 - 2
= 0.36 kN/m²

Vertical load

from roof loads

GDa $= g_{Da} - b$ = 10,00 - 3,375 = 33.75 kN

QDa $= q_{Da} - b$ = 2,50 - 3,375 =8 .44 kN

Line load from ceiling (finite element calculation)

$gDe_{,li}$ =43.37 kN/m $qDe_{,li}$ = 16.50 kN/m

$gDe_{,re}$ =22.65 kN/m $qDe_{,re}$ = 8.49 kN/m

GDe = 111.4 kN QDe = 42.15 kN

e_g = -0,177 m e_q = -0,181 m

Single load from adjacent lintel (downstand beam)

GUz =14.6 kN QUz = 5.70 kN

Dead load wall (per storey)

GWa $= g_{Wa} - b - h$ = 3.48 - 3.375 - 2.625 = 30.8 kN

Support forces linearised over wall areas

$qDe_{,li}$ = 59,87-43,37
= 16.50 kN/m

$qDe_{,re}$ = 31,14-22,65
= 8.49 kN/m

GDe $= (gDe_{,li} + gDe_{,re}) / 2 - b$

QDe $= (qDe_{,li} + qDe_{,re}) / 2 - b$

Values e_g and e_q from subsidiary invoice

Values G_{Uz} and Q_{Uz} determined from line bearing in door area

Internal forces

Action at the wall head of the ground floor wall

$Ng_{,k}$ = 33.75 + 4 - 111.4 + 3 - (14.6 + 30.8) = 615.6 kN
$N_{q,k}$ = 8.44 + 4 - 42.15 + 3 - 5. 70= 194.1 kN

Moments from eccentric ceiling load EG - DG

$Mg_{,k}$ = 4 - 111,4 - -0,177 = -78.43 kNm
$Mq_{,k}$ = 4 - 42,15 - -0,181 = -30.52 kNm

Due to the symmetry of the floor plan, the moments from eccentricity can be centred by the transmission of horizontal forces in the hinged slab plates. For the centring of the vertical forces from the lintels and the eccentricities of the slab loads, the horizontal forces required for this are subsequently determined. Lying on the safe side, these eccentricities are only centred to 90 %. The slab load (EG) directly resting on the wall is applied with the full eccentricity. The following bending moments therefore result for the wall head:

$Mg_{,k,red}$ = 3·111,4·-0,176·(1-0,9)+111,4·-0,176 = -25.5 kNm

$Mq_{,k,red}$ = 3·42,15·-0,181·(1-0,9)+42,15·-0,181 = -9.92 kNm

$e_{g,k,red}$ = 25,5 / 615,6 = 0.041 m

$eq_{,k,red}$ = 9,92 / 194,1 = 0,051 m

Determination of the centring forces per floor

$eDe_{,g}$ = -0,177 m $eDe_{,q}$ = -0,181 m

$Hg_{,De,k}$ = -(111,4 - -0,177 - 0,9) / 2,845 = 6.24 kN

$Hq_{,De,k}$ = -(42,15 - -0,181 - 0,9) / 2,845 = 2.41 kN

e_{Uz} = -(3,375 - 0,25) /2 =-1 ,563 m

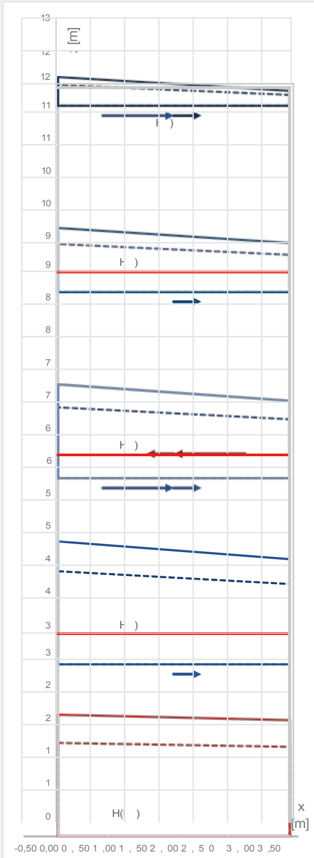
$Hg_{,Uz,k}$ = -(14,6 - -1,563) / (2,625 + 0,22) =8 .02 kN

$Hq_{,Uz,k}$ = -(5,70 - -1,563) / (2,625 + 0,22) =3 .13 kN

Design standard forces:

N_{Ed} = 1.35 - $N_{g,k}$ + 1.5 - $N_{q,k}$

Course of the vertical forces *g* and *q* (linearised) and the horizontal forces from centring the moments:



The horizontal forces

H = N - e / h

must be taken into account when dimensioning the wall panel for horizontal forces.
Eccentricity e_{Uz} is determined for a lintel bearing depth of 25 cm.

$HWall_{head}$ = *-M / h*
$HWall_{foot}$ = *+M / h*

The loads from the fall in the EC are taken into account separately.

Wall head:

$N_{Ed,o}$ = 1.35 - 615.6 + 1.5 - 194.1 = 1122.2 kN

$N_{Ek,o}$ = 1.00 - 615.6 + 0 =615 .6 kN

Wall centre:

$g_{w,m}$ = 2.625 / 2 - 3.48 = 4.57 kN/m

$N_{Ed,m}$ = 1.35 - 4.57 - 3.375 + 1122.2 = 1143.0 kN

Wall base:

g_{w} = 2,625 - 3,48 = 9.14 kN/m

$N_{Ed,u}$ = 1.35 - 9.14 - 3.375 + 1122.2 = 1163.8 kN

Associated bending moments in the longitudinal direction of the wall taking into account of the centring forces:

$M_{Ed,o}$ = 1,35 - -25,5 + 1,5 - - =-49.3 kNm

with $H_{Ed,De}$ = 1.35 - 6.24 + 1.5 - 2.41 =12 .0 kN

$M_{Ed,u}$ = -49.3 + 12.0 - 2,845 =-32.2 kNm

$M_{Ed,m}$ = -49.3 + 12.0 - 2,845/2 =-15.2 kNm

Normal force curve over the wall height:

Wall head:	$nEd_{o,li}$ = 358.6 kN/m	$nEd_{o,re}$ = 306.6 kN/m
Wall centre:	$nEd_{m,li}$ = 355.7 kN/m	$nEd_{m,re}$ = 321.7 kN/m
Wall base:	$nEd_{u,li}$ = 352.9 kN/m	$nEd_{u,re}$ = 336.9 kN/m

Consideration of the load from the lintel:

Support depth of lintel: a = 0.25 m

Lower edge of lintel: h_s = 2.25 m

Load length in the middle of the wall: l_m = 0.25 + 0.94 - tan(30°) = 0.79 m

Load length at the wall base: l_u = 0.25 + 2.25 - tan(30°) = 1.55 m

Maximum design normal force max $N_{Ed,o}$

$N_{Ek,o}$ = min $N_{Ed,o}$

Clear wall height = 2.625 m

from centring ceiling load in the centre of the wall:

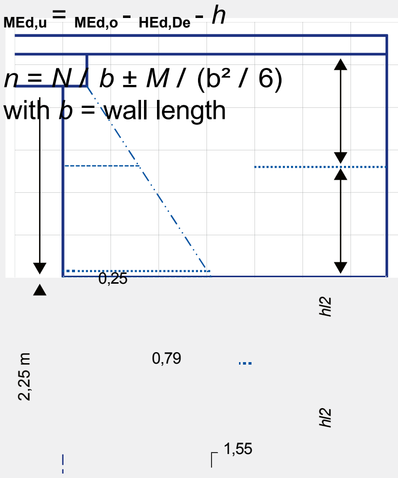
$M_{Ed,m} = M_{Ed,o} - H_{Ed,De} - h/2$

at the base of the wall

$M_{Ed,u} = M_{Ed,o} - H_{Ed,De} - h$

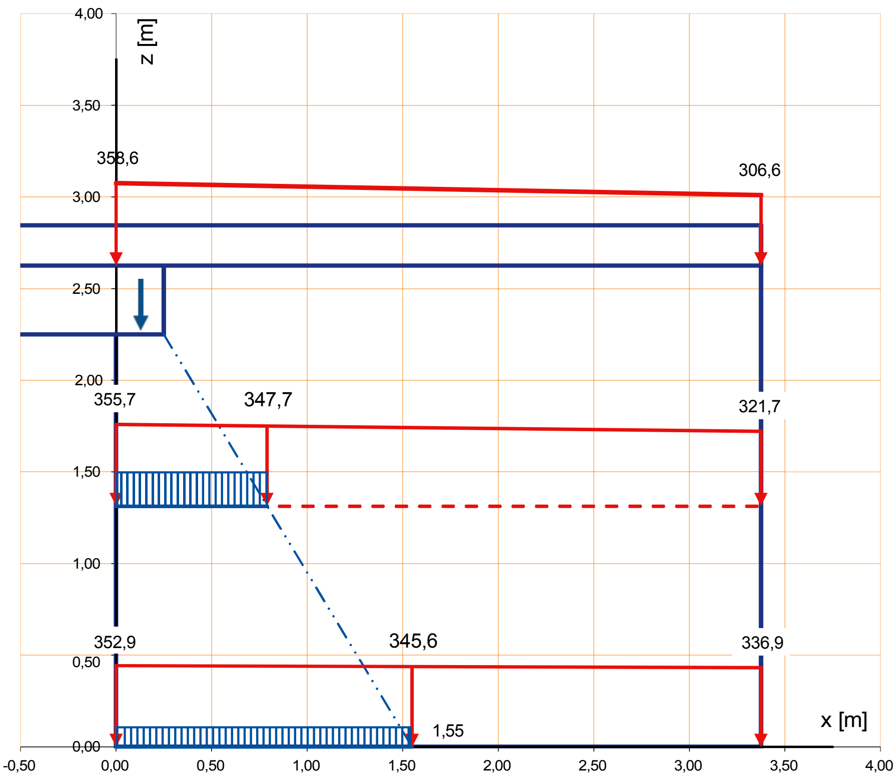
$n = N / b \pm M / (b^2 / 6)$

with b = wall length



Load spread under 60°

Load pattern



Wall centre (l_m = 0.79 m)
$N_{Ed,m}$ = (355.7 + 347.7)/2 - 0.79 + 28.26 = 306.1 kN
Wall base (l_u = 1.55 m)
$N_{Ed,u}$ = (352.9 + 345.6)/2 - 1.55 + 28.26 = 569.6 kN

The loads from the eccentric normal force are superimposed in the middle of the wall and at the wall base with the load from the door lintel (index Uz), which is applied as a block load.

$N_{Ed,Uz}$ = 1.35 - 14.6 + 1.5 - 5.7
= 28.26 kN

The load eccentricity over the range *l is* negligible due to the centring by horizontal forces.

Table A.8 Review of the conditions for applying the simplified calculation methods

Criterion	Request	Actual value	Comment
Maximum building height	$H \leq 20$ m	15,35 m	complied with
Maximum ceiling support width	$l \leq 6$ m	5,34 m	complied with
Maximum permissible storey height	no limitation	2,625 m	complied with
Maximum traffic load on ceilings	$q_k \leq 5$ kN/m²	2.7 kN/m²	complied with

Compare section 4

Verification of the normal force bearing capacity

Rated value N_{Rd} of the resistor

$N_{Rd} = \Phi \cdot A \cdot f_d$

A Wall cross section

Reduction factor Φ_1 from ceiling torsion

$\Phi_{1,u} = \Phi_{1,o} = 1$

The different support widths of the adjoining slabs are taken into account in the calculation of the reduction factor Φ for buckling in the middle of the wall. The slab twists are therefore only to be taken into account for end supports of a slab, i.e. $\Phi_1 = 1$.

Reduction factor Φ_2 for buckling

Buckling length factor for 175 < t ≤ 250 mm:

with t = 24 cm follows $\rho_2 = 0.9$

Kink length:

$h_{ef} = \rho_2 \cdot h = 0.9 \cdot 2.625 = 2.36$ m

h_{ef} according to equation (20)

Slenderness:

$h_{ef} / t = 2.36 / 0.24 = 9.8 < 27 = \text{perm } h_{ef} / t$

$$\Phi_2 = 0{,}85 \cdot \frac{a}{t} - 0{,}0011 \cdot \left(\frac{h_{ef}}{t}\right)^2$$

$$= 0{,}85 \cdot 1 - 0{,}0011 \cdot 9{,}8^2 = 0{,}74$$

Reduction factor Φ_y for eccentricities in longitudinal wall direction

Wall head: M_{Ed} = 49.3 kNm, M_{Ed} = 1122.2 kNm,
$\Phi_{y,o} = 1 - 2 \cdot 0{,}044 / 3{,}375 = 0{,}97$

Wall centre: $\Phi_{y,m} = 1$

Wall base: $\Phi_{y,u} = 1$

Φ_y according to equation (37)

with $e = M / N$

e_o = 49.3 / 1122.2 = 0.044 m

e_m ≈ 0.00 m

e_u ≈ 0.00 m

Dimensioning

chosen: Masonry made of vertically perforated bricks in accordance with DIN EN 771-1 in conjunction with DIN 20000-401.

Stone strength class: 12

Mortar: DM (thin-bed mortar)

Value of the characteristic compressive strength
(DIN EN 1996-3/NA, Tab. NA.D.10):
f_k = 4.7 MN/m²

Design value of the compressive strength:

$f_d = 0.85 \cdot \frac{4.7}{1,5} = 2.66$ MN/m²

Rated resistance at the wall head:

with $\Phi_o = \Phi_y = 0.97 > 0.85$

* No further verification required Rated

resistance in the middle of the wall:

with Φ_m = 1,0 - 0,74

$N_{Rd,m} = \Phi_m \cdot A \cdot f_d \cdot 1000$ = 0.74- 0.24 - 0.79 - 2.66 - 1000

= 373.2 kN > $N_{Ed,m}$ = 306.1 kN

Rated resistance at the wall base:

with $\Phi_u = \Phi_y$ = 1,0

$N_{Rd,u} = \Phi_u \cdot A \cdot f_d$ = 1,0 - 0,24 - 1,55 - 2,66 - 1000

= 989.5 kN > $N_{Ed,u}$ = 569.6 kN

With Φ > 0.85, the verification at the centre (or base) of the wall is decisive in any case.

In the centre of the wall, the reduction factors are to be superimposed in the longitudinal and transverse direction of the wall according to equation (38)
$\Phi_m = \Phi_2 \cdot \Phi_y$
with A = 0.24 m - 0.79 m

Note: Only in the case of large load excentricities can the design at the wall base become decisive. As a rule, the evidence for the Interior walls are omitted.

Table A.9 Compilation of the decisive values

Location	Diminishing factors Φ_i	Resistance N_{Rd} kN	In-effect N_{Ed} kN	$\frac{N_{Ed}}{N_{Rd}}$	Comments
Wall head	0,97	-	-	-	not normative
Wall centre	0,74	373,2	306,1	0,82	Evidence provided
Wall base	1,0	989,5	569,6	0,58	Evidence provided

Note: The verification of the partial surface pressure from the door lintel is not necessary due to
The low support load is not decisive and is not listed here!

Verification of the spatial bracing

Building geometry (lengths in m, top of basement ceiling = -0.15 m)

Dimension	Building	Bay 1	Bay 2 (left and right)	Total length or width
Length	14,25	1,00	3,49	15,25
Wide	14,37	7,86	0,5	15,365

Length = gable wall side

Width = eaves side

Height grade Clamping plane		-0,15 m
Height level OK last ceiling		11,23 m
Elevation height Eaves height		12,015 m
Height grade OK ridge		15,35 m
Roof pitch hip	sideways front / rear	25 ° 25 °
Storey heights		2,845 m

OK basement ceiling
(with covering thickness 15 cm)

2,625 + 0,22 = 2,845 m

medium height in the attic $\frac{15{,}35 - 12{,}015}{2} + 0{,}785 = 2{,}45$ m

Wall thickness outer wall 0,365 m

Wall thickness inner wall 0,24 m

Wall lengths inner walls

load-bearing inner walls x-direction. $2 \cdot l_1 = 2 \cdot 5.365 = 10{,}73$ m

load-bearing inner walls y-direction $2 \cdot l_2 = 2 \cdot 5.385 = 10{,}77$ m

Flat partition wall and staircase walls

$l_3 + 2 \cdot l_4 + l_5 = 7.645 + 2 \cdot 6.635 + 2.845 = 23.76$ m

Wall lengths without opening fume cupboard

l_1 = 5.365 m (from figure A.2)

l_2 = 1.00 + 0.885 + 3.50 = 5,385 m

l_3 = 1.26 + 5.385 + 1.00 = 7,645 m

l_4 = 5.135 + 0.24 + 1.26 = 6,635 m

l_5 = 0.24 + 2.365 + 0.24 = 2,845 m

Inclined position

Building height to OK foundation

h_{tot} = 15,35 + 0,15 = 15,50 m

$$\upsilon = \frac{1}{100\sqrt{h_{tot}}} = 0.00254 \text{ rad} = \frac{1}{394} \text{ rad}$$

according to DIN EN 1996-1-1, 5.3

Horizontal load from inclined position:

$$H = \sum N \cdot \upsilon = \frac{\sum N}{394}$$

Determination of the vertical loads for the calculation of the horizontal loads from inclined position

Component weights

External wall cf. item 1 3.53Load-
bearing internal walls cf. item 2 3.48 kN/m²
Apartment partition wall 24 - 20 + 0.36 5.16 kN/m²
Ceiling weight KG-DG= 7.30 kN/m²
Number of floor slabs without KG 4

Vertical permanent loads

Roof construction:

$G_{Da} = \frac{g_S}{\cos(25)} \cdot A_G = \frac{0,95}{\cos(25)} 215.2$ 225.6 kN

Reinforced concrete floors:
$G_{De,1}$ = 215.2 - 7. 30= 1571.0 kN

Walls: Attic:

Exterior walls:
$G_{A,DG}$ = 2 - (15.25 + 15.31) - 3.53 - 0. 50= 107.9 kN

Interior walls:
$G_{I,DG}$ = (10.73 - 3.48 + 10.77 - 3.48 + 23.76 - 5.16) - 2.45 = 483.7 kN

1st floor to 3rd floor:

Exterior walls:
$G_{A,1}$ = 2 - (15.25 + 15.31) - 3.53 - 2.845 - 3= 1841.5 kN

Interior walls:
$G_{I,1}$ = ((10,73 + 10,77) - 3,48 + 23,76 - 5,16) - 2.845 - 3 = 1685.0 kN

Ground floor:

Exterior walls:
$G_{A,EG}$ = 2 - (15.25 + 15.31) - 3.53 - 2. 845= 613.8 kN

Interior walls:
$G_{I,EG}$ = ((10.73 + 10.77) - 3.48 + 23.76 - 5.16) - 2. 845= 561.7 kN

Dead weights

0.22 - 25 + 1.8 = 7.30 kN/m²

Average surface load of the roof construction
g_s = 0.95 kN/m²

Floor space:
A_G = 14.25 - 14.305 + 1.0 - 7.86 + 3.49 - 0.5 - 2
= 215,2 m²

Wall height in the jamb area:
h = 0,50 m

Average wall height in the attic:
h = 2,45 m

with g_W = 3.53 kN/m²

with g_{W1} = 3.48 kN/m²
g_{W2} = 5.16 kN/m²

Total vertical loads from load case permanent loads

for inclined position at the height of the ground floor ceiling

$\sum G_{EG}$ = 225.6 + 4 - 1571.0 + 107.9 + 483.7 + 1841.5 + 1685.0 + (613,8 + 561,7) / 2 = 11216 kN

For the load application at the height of the ground floor ceiling, half the wall load from the ground floor is taken into account.

Vertical live load

Snow:

$Q_{Da,s}$ = *s* - *A* = 0.52 - 215.2 = 111.9 kN

According to DIN EN 1991-1-3
Snow load zone 1:
s_k = 0.65 kN/m²
μ_1 = 0.8
s = 0.65 - 0.8 = 0.52 kN/m²

Payloads Category A:

$Q_{De,1}$ = q_{De} - *A* = 2.70 - 215.2 = 581.0 kN

$QDe_{,ges}$ = 581.0 - 4 = 2324 kN

Payload + partition surcharge:
1.50 + 1.20 = 2.70 kN/m²

Horizontal loads from inclined position

from permanent loads

$_{HS,3OG,gk}$ = $\frac{225,6 + 1571,0 + 107,9 + 483,7 + (613,8 + 561,7) / 2}{394}$

= 7.55 kN

$_{HS,i,gk}$ = $_{Gi,gk}$ - *υ*
with *υ* = 1/394

$_{HS,2OG,gk}$ = $_{HS,1OG,gk}$ = $_{HS,EG,gk}$

= $\frac{1571,0 + 613,8 + 561,7}{394}$ = 6.97 kN

from non-permanent loads

$_{HS,3OG,sk}$ = $\frac{111,9}{394}$ = 0.28 kN

From snow

$_{HS,3OG,qk}$ = $_{HS,2OG,qk}$ = $_{HS,1OG,qk}$ = $_{HS,EG,qk}$ = $\frac{581,0}{394}$ = 1,47 kN

from payload category A

Horizontal load from wind according to DIN EN 1991-1-4

Prismatic structure		*h/b*	= 1,01 ≈ 1,0
Range D	(wind pressure)	$cpe_{,10}$	= 0,8
Area E	(wind suction)	$cpe_{,10}$	= -0,5
		$_{cpe}$	= 1,3

i.e. no graduated wind pressure distribution required

according to DIN EN 1991-1-4, Table 7.1

Only the main areas H and I according to DIN EN 1991-1-4, Figure 7.9 are used.

Wind parameters for the hip roof			
Inclination angle	**Area H**	**Area I**	**Vector sum**
***α* in °**	***cpe*$_{,10}$**	***cpe*$_{,10}$**	***cpe*$_{,Wa}$**
15	0,2	-0,5	-
30	0,4	-0,4	-
25 (interpolated)	0,33	-0,43	0,76

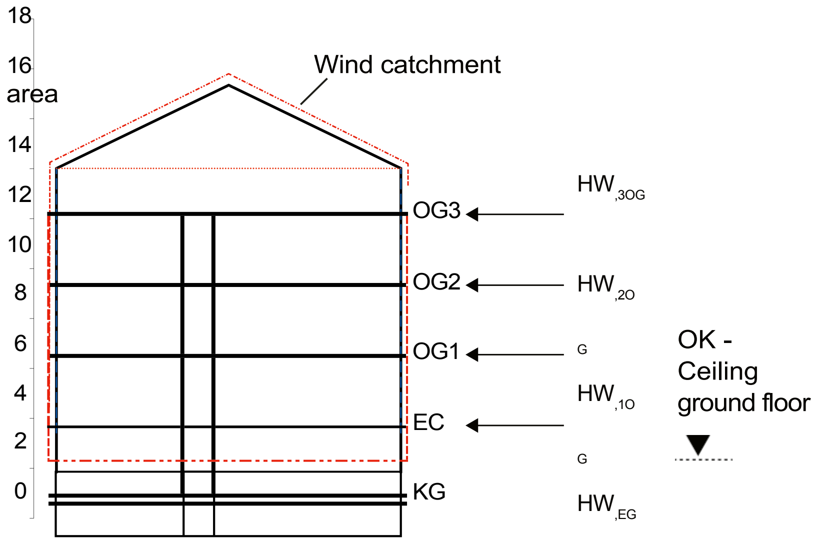
Determination of the wind catchment area

Wind zone WZ 2 Inland

Wide
b_p = 15.25 m ≈ h = 15.35 m

Gust velocity pressure
10 m < h < 18 m: with h = 14.86 m q_P = 0.80 kN/m²

Simplified gust speed pressure q_P as a function of the building height according to DIN EN 1991-1-4/NA, Table NA.B.3



Wind load on hip roof

$HW_{,Walm}$ = $_{qw}$ - $cpe_{,Wa}$ - $_{bp}$ - $_{hWaD}$ / 2

= 0,8 - 0,76 - 15,25 - $\frac{3,335}{2}$ =15 .5 kN

Wind loads on wall (resultant acting at the height of the ceiling panes)

$HW_{,Wa,i}$	= $_{qw}$ - $cpe_{,p}$ - $_{bp}$ - $hp_{,i}$		
$HW_{,Wa,3OG}$	= 0,8 - 1,3 - 15,25 - (0,785 + 2,845/2)	=	35.0 kN
$HW_{,Wa,2OG}$	= 0,8 - 1,3 - 15,25 - 2,845	=	45.1 kN
$HW_{,Wa,1OG}$	= 0,8 - 1,3 - 15,25 - 2,845	=	45.1 kN
$HW_{,Wa,EG}$	= 0,8 - 1,3 - 15,25 - 2,845	=	45.1 kN
$HW_{,Ges}$	=		185.8 kN

Characteristic horizontal loads from the load cases permanent loads, snow, live load and wind as well as from the decisive load combination of the design load:
$H_{Ed} = 1.35 - H_{g,k} + 1.5 - (\Psi_{0,s} - H_{s,k} + \Psi_{0,q} - H_{q,k} + H\,)_{s,w}$
Load share attributable to windscreen: 0.233

$\Psi 0_{,i}$	**1,0**	**0,5**	**0,7**	**1,0**	
	HLF g,k	HLF s,k	HLF q,k	HLF w,k	HEd
	kN	**kN**	**kN**	**kN**	**kN**
3RD FLOOR	1,76	0,065	0,343	11,77	20,43
2ND FLOOR	1,62		0,343	10,51	18,31
1ST FLOOR	1,62		0,343	10,51	18,31
EC	1,62		0,343	10,51	18,31
ΣH	**6,62**	**0,065**	**1,34**	**43,30**	**79,34**

Based on the stiffness distribution of the stiffening wall panels, a comparative calculation for the windscreen under consideration resulted in a load share of 23.3 % of the total load.

for the 3rd floor
$H_{g,k}$ = 0.233 - 7.55 = 1.76 kN
$H_{s,}$ = 0.233 - 0.28 = 0.065
$H_{q,k}$ = 0.233 - 1.47 = 0.343 kN

Proof of compliance

$h_{dead} - \sqrt{\frac{N_{Ed}}{\sum E - I}}$ ≤ 0,6 for n ≥ 4
≤ 0,2 + 0,1 - n for 1 ≤ n < 4

with:
h_{tot} Building height above clamping level
n Number of storeys
N_{Ed} Design value of the vertical actions up to the Clamping level

$\sum E\text{-}I$ Sum of the bending stiffness of the bracing walls in the direction under consideration

E = 1100 - f_k with f_k = 4.7 MN/m²
= 1100 - 4,7 = 5170 MN/m²

$I = \frac{t - b^3}{12} = \frac{0{,}24 - 3{,}375^3}{12}$ = 0,769 m4

$\sum I$ = 0,769 / 0,233 = 3,30 m4

N_{Ed} = 1,0 - 11803 + 1,0 - (0,5 - 111,9 + 0,7 - 2324) = 13486 kN

$h_{tot} - \sqrt{\frac{N_{Ed}}{\sum E - I}} = 18{,}35 - \sqrt{\frac{13{,}486}{5170 - 3{,}30}} = 0{,}52 < 0{,}6$

Proof is provided.
Therefore, changes in the shape of the stiffening components do not have to be taken into account when determining the section size!

According to DIN EN 1996-1-1, 5.4(1), the proof of compliance may not exceed
(service load level).

The partial safety factors are therefore set at γ_g = 1.0 and γ_q = 1.0.

$N_{Ek.g}$ = 11216 +
(613,8 + 561,7) / 2
= 11803 kN

t = wall thickness
b = wall length

h_{tot} = 2.875 + 2.815 + 3 - 2.845 + 0,785 + 3,335
h_{tot} = 18.35 m

As described above, the wall plate takes on 0.233 times the horizontal force. Consequently, the total moment of inertia ΣI is 1/0.233 times the moment of inertia of the wall with b = 3.375 m.

Determination of the decisive cutting forces:

Horizontal forces H and moments M_H in longitudinal wall direction

i	Height load attack		Hk	HEd	$Mk_{,H}$	MEd,H
	z [m]	Δz [m]	kN	kN	kNm	kNm
4	11,380	8,535	13,80	20,43	117,8	174,4
3	8,535	5,690	12,37	18,31	70,4	104,2
2	5,690	2,845	12,37	18,31	35,2	52,1
1	2,845	0,000	12,37	18,31	0,0	0,0
	Total		50,91	75,36	223,4	330,7

Normal forces N_{Ed}

	Normal force max N_{Ed}	Normal force min N_{Ed} (γ_G = 1.00)
Wall centre	Wall load $G_{W,m}$ = 2.625/2 - 3.48 - 3.375 = 15.42 kN	
	$N_{Ed,m}$ = 1122.2 + 28.26 + 1,35 - 15,42 $N_{Ed,m}$ = 1171.3 kN	$N_{Ed,m}$ = 615,6 + 14,6 + 1,0 - 15,42 $N_{Ed,m}$ = 645.6 kN
Wall foot	Wall load $G_{W,u}$ = 2.625 - 3.48 - 3.375 = 30.83 kN	
	$N_{Ed,u}$ = 1122.2 + 28.26 + 1,35 - 30,83 $N_{Ed,u}$ = 1192.1 kN	$N_{Ed,u}$ = 615,6 + 14,6 + 1,0 - 30,83 $N_{Ed,u}$ = 661.0 kN

Moments MEd for load case max NEd

Wall centre	MEd,N	= 1,35 - -25,5 + 1,5 - -9,92	=-49.3 kNm
	MEd,UZ	= $N_{Ed,UZ}$ - e_{UZ} = 28.26 - -1.563	=-44.2 kNm
	MEd,o,Z	= $H_{Ed,z}$ - z = 27.56 - 1.423	= 39.22 kNm
	with z	= (2,625 + 0,22)/2 = 1,423 m	
	MEd,o,H	= ± (330,7 + 75,36 - 1,423)	= ±437.9 kNm
	min $M_{Ed,m}$	= -49,3 - 44,2 + 39,2 - 437,9	= **-492.2 kNm**
Wall foot	MEd,N	=	=-49.3 kNm
	MEd,UZ	=	=-44.2 kNm
	MEd,o,Z	= $H_{Ed,z}$ - z = 27.56 - 2.735	=75.4 kNm
	with z	= 2,625 + 0,22/2 = 2,735 m	
	MEd,o,H	= ± (330,7 + 75,36 - 2,735)	= ±536.8 kNm
	min $M_{Ed,u}$	= -49,3 - 44,2 + 75,4 - 536,8	= **-554.9 kNm**

Note: The load application points of the horizontal loads are located in the centre axes of the ceiling panels!

$Hk_{,EG}$ = 1,62 + 0,7 - 0,343 + 10,51
= 12.37 kN

$H_{Ed,EG}$ = 1.35 - 1.62 + 1.5 - (0,7 - 0,343 + 10,51)
= 18.31 kN

M_H = H-Δz

GW,m = h / 2 - γ_W - b

NEd,m = NEd,o + NEd,Uz + γg - GW,m

GW,u = h - γ_W - b

NEd,u = NEd,o + NEd,Uz + γg - GW,u

MEd,N = 1.35 - $Mg_{,k,red}$ + 1.5 - $Mq_{,k,red}$

from door lintel (index UZ):

$N_{Ed,Uz}$ = 28.26 kN

from centring forces (index Z):

HEd,Z = 1,35 - (6,24 + 8,02) + 1,5 - (2,41 + 3,13)
= 27.56 kN

MEd,o,H = MEd,H + HEd - z

see wall centre

Moments M_{Ed} for load case min N_{Ed}				
Wall centre	$M_{Ed,N}$	= 1,0 - -25,5 + 0,0	=	-25.5 kNm
	$M_{Ed,UZ}$	= $N_{Ed,UZ}$ - e_{UZ} = 14.6 - -1.563	=	-22.8 kNm
	$M_{Ed,o,Z}$	= $H_{Ed,Z}$ - z = 14,26 - 1,423	=	20.3 kNm
	$M_{Ed,o,H}$	= ± (330,7 + 75,36 -1,423)	=	±437.9 kNm
	min $M_{Ed,m}$	= -25,5 - 22,8 + 20,3 - 437,9	=	-465.9 kNm
Wall foot	$M_{Ed,N}$	=	=	-25.5 kNm
	$M_{Ed,UZ}$	=	=	-22.8 kNm
	$M_{Ed,o,Z}$	= $H_{Ed,Z}$ - z = 14,26 - 2,735	=	39.0 kNm
	$M_{Ed,o,H}$	= ± (330,7 + 75,36 - 2,735)	=	±536.8 kNm
	min $M_{Ed,u}$	= -25,5 - 22,8 + 39,0 - 536,8	=	-546.1 kNm
	max $M_{Ed,u}$	= -25,5 - 22,8 + 39,0 + 536,8	=	+527.5 kNm
Characteristic moments M_{Ek} for load case min N_{Ed}				
Wall foot	$M_{Ek,N}$	= -25,5 - 0	=	-25.5 kNm
	$M_{Ek,UZ}$	= (14,6 + 0) - -1,563	=	-22.8 kNm
	$M_{Ek,o,Z}$	= $H_{Ek,Z}$ - z = 14,26 - 2,735	=	39.0 kNm
	$M_{Ek,o,H}$	= ± (223,4 + 50,91 - 2,735)	=	±362.6 kNm
	min $M_{Ek,u}$	= -25,5 - 22,8 + 39,0 - 362,6	=	371.9 kNm

Determination of the eccentricities $e = M / N$

Wall centre:		
from max N_{Ed}	e = 492,2 / 1171,3	= 0,420 m
off min N_{Ed}	e = 465,9 / 645,6	= 0,722 m
Wall base:		
from max N_{Ed}	e = 554,9 / 1192,1	= 0,465 m
off min N_{Ed}	e = 546,1 / 661,0	= 0,826 m
from N_{Ek}	e_k = 371,9 / 661,0	= 0,563 m

$M_{Ed,N}$ = 1.0 - $Mg_{,k,red}$

$N_{Ed,UZ}$ = 1.0 - 14.6 =14.6 kN

from centring forces:

$H_{Ed,Z}$ = 1,0 - (6,24 + 8,02) = 14,26 kN

$M_{Ed,o,H}$ = $M_{Ed,H}$ ± H_{Ed}-z

see wall centre

$H_{Ek,Z}$ = 6,24 + 0 + 8,02 + 0 = 14.26 kN

Proof of the windshield at the base of the wall

Rated value N_{Rd} of the resistor

$N_{Rd} = \Phi \cdot A \cdot f_d = \Phi \cdot t \cdot b \cdot f_d$

Reduction factor Φ_1

with predominant bending stress:

$\Phi_1 = 1 - 2 \cdot e_w / b$

Wall centre:
For max N: $\Phi_{m,maxN}$ = 1 - 2 - 0,420 / 3,375= 0,75
For min N: $\Phi_{m,minN}$ = 1 - 2 - 0,722 / 3,375= 0,57
Wall foot:
For max N: $\Phi_{u,maxN}$ = 1 - 2 - 0,465 / 3,375= 0,72
For min N: $\Phi_{u,minN}$ = 1 - 2 - 0,826 / 3,375= 0,51

according to DIN EN 1996-1-1/NA, NCI to 6.1.2.2 (NA.3)
(i = authoritative detection point)

with e_w = e from internal force calculation

Rated resistors

Due to the short-term load from wind, the creep rupture factor ζ = 1.0 can be assumed.

f_d = 1.0 - 4.70 / 1.50 = 3.13 MN/m³

Wall centre:
$N_{Rd,m,maxN}$ = 0.75 - 0.74 - 0.24 - 3.375 - 3.13 - 1000 = 1407.1 kN
$N_{Rd,m,minN}$ = 0.57 - 0.74 - 0.24 - 3.375 - 3.13 - 1000 = 1069.4 kN
Wall base:
$N_{Rd,u,maxN}$ = 0,72 - 0,24 - 3,375 - 3,13 - 1000 = 1825.4 kN
$N_{Rd,u,minN}$ = 0,51 - 0,24 - 3,375 - 3,13 - 1000 = 1293.0 kN

In order to take into account a normal stress distribution that varies over the wall length as a result of the moment from the horizontal load, the reduction value Φ_y is then superimposed with the previously determined reduction factor Φ_2 (buckling).

$N_{Rd,m} = \Phi_y \cdot \Phi_2 \cdot t \cdot b \cdot f_d$
according to equation (38)

Verification of the normal force bearing capacity

Wall centre:
For max N: $N_{Ed,m,maxN}$ = 1171.3 kN < $N_{Rd,m,maxN}$ = 1407.1 kN
For min N: $N_{Ed,m,minN}$ = 645.6 kN < $N_{Rd,m,minN}$ = 1069.4 kN

Wall foot:
For max N: $N_{Ed,u,maxN}$ = 1192.1 kN < $N_{Rd,u,maxN}$ = 1825.4 kN
For min N: $N_{Ed,u,minN}$ = 661.0 kN < $N_{Rd,u,minN}$ = 1293.0 kN

Evidence provided

Utilisation rates:

N_{Ed} / N_{Rd} = 0,83 < 1,0
N_{Ed} / N_{Rd} = 0,60 < 1,0

N_{Ed} / N_{Rd} = 0,65 < 1,0
N_{Ed} / N_{Rd} = 0,51 < 1,0

Verification of shear stress at the wall base

$V_{Ed} \leq V_{Rdlt}$

from wind and inclined	$V_{Ed,S} = H_{Ed}$	= 75.36 kN
position: from centring:	$V_{Ed,Z}$	= 14.26 kN
design shear force:	V_{Ed}	= 89.62 kN

Equation (23)

$V_{Ed,Z}$ = 1,0 - (6,24 + 8,02) = 14.26 kN

Determination of V_{Rdlt}:

Pressed-over wall length for LF min *N* and max *M*:

$l_{c,lin}$ = 1.5 - (1 - 2 - e_w / l) - l < l
= 1.5 - (l - 2 - e_w) = 1.5 - (3.375 - 2 - 0.798) = 2,67 m

$l_{c,lin}$ according to equation (27)

e_w = 527.5 / 661.0 = 0.798 m

Compressive stress in masonry:

σ_d = 661.0 / (0.24 - 2.67) / 1000 = 1.03 MN/m²

Friction failure

The bond shear strength is not applied to the wall on the ground floor for the verification of the shear load-bearing capacity:

f_{vk0} = = 0.0 MN/m²

Shear strength f_{vlt1}

f_{vlt1} =0 + 0,4 - 1,03 = 0.412 MN/m²

f_{vlt1} according to equation (30)

Stone tensile failure

Type of stone: perforated stone

Calculated compressive strength: f_{st} = 15

for stone strength class 12

N/mm² Calculated stone tensile strength

$f_{bt,cal}$ = 0.026 - f_{st} = 0.026 - 15 = 0.39 MN/m²

Shear strength stone pull

f_{vlt2} = 0,45 - 0,39 $\sqrt{1+\frac{1,03}{0,39}}$ - = 0.335 MN/m²

f_{vk} = min (0.412; 0.335) = 0.335 MN/m²

f_{vlt2} according to equation (32) f_{vk} = min (f_{vlt1}, f_{vlt2})

Note: according to DIN EN 1996-1-1/NA, NA.K.3 (2) the increase of the calculated wall length to l_{cal} = 1,125 - l or l_{cal} = 1,333 - $l_{c,lin}$ may only be carried out when using simple cantilever models.

Design value of the component resistance for shear force loading with

$l_{cal} = l_{c,lin}$ = 2,67 m

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

$$= 2.67 \cdot \frac{0.335}{1,5} \cdot \frac{0.24}{1,0} \cdot 1000 = 143.1 \text{ kN}$$

V_{Rdlt} according to equation (24)

with h / l = 2,625 / 3,375 = 0,78 < 1

the shear stress distribution factor according to DIN EN 1996-1-1/ NA, Annex NA.K.3(1) c = 1,0

Shear proof:

V_{Ed} = 89.62 kN < V_{Rdlt} = 143.1 kN

Evidence provided

V_{Ed} / V_{Rdlt} = 0,63 < 1,0

Verification of the edge strain

Since the initial shear strength f_{vk0} was not used for the verification of the shear capacity, a verification of the edge strain for the pane stress is not necessary here.

A.2.3 Summary of the proofs of all exterior and interior walls

AW = outer wall

IW = inner wall

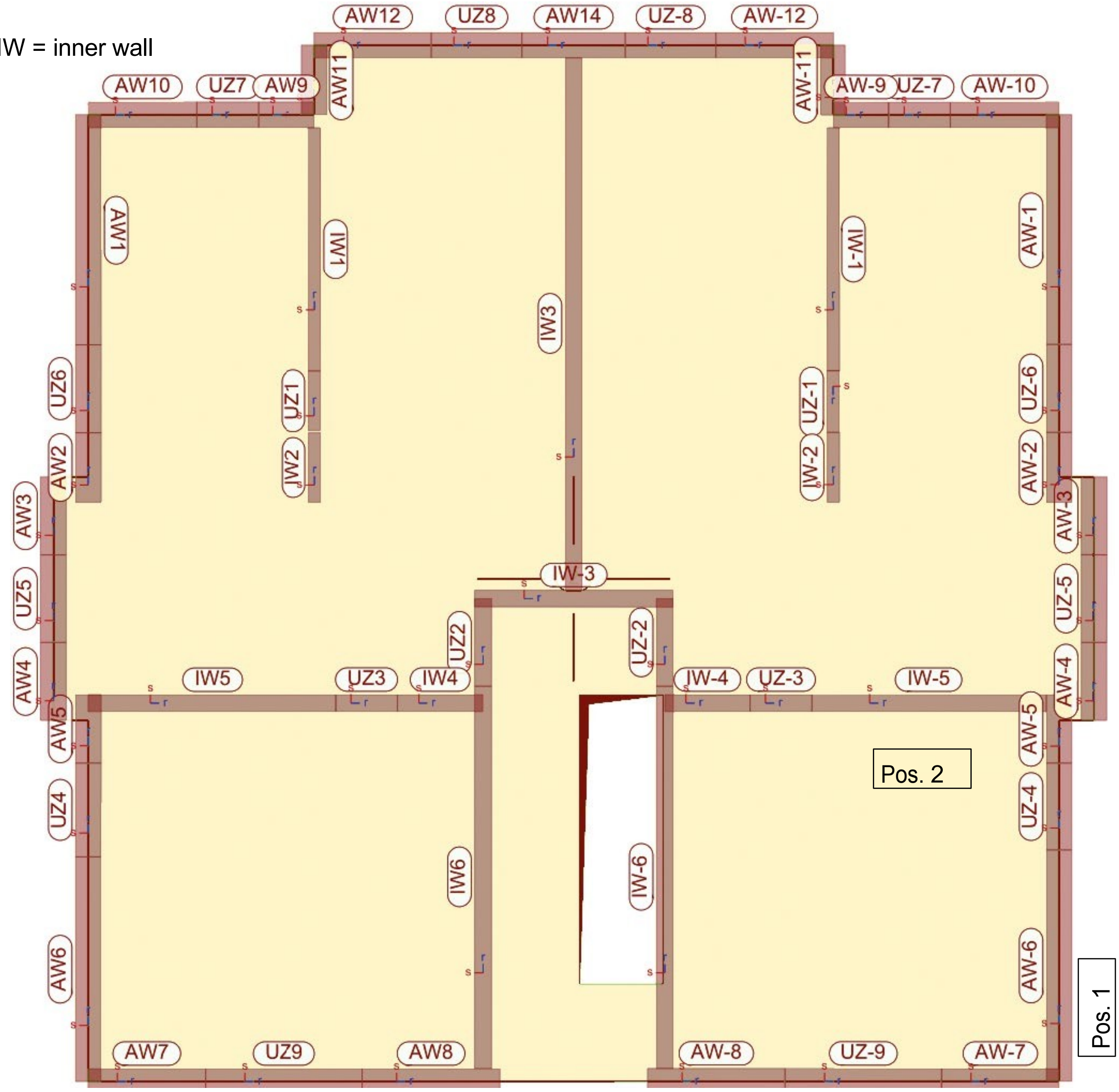


Figure A.15: Floor plan and wall designations

For the load capacity verifications with wind as the predominantly acting horizontal force, the creep factor ζ was increased from 0.85 to 1.0 and the load from lateral camber was applied as an eccentrically acting normal force over the entire wall cross-section.

Wall	t	1st to 3rd floor		Ground floor		Utilisation rates			
						Normal force		Thrust	Max
	cm	Stone rough-density-class	f_k MN/m²	Stone rough-density-class	f_k MN/m²	*N* only [-]	with [H1)] [-]	[-]	[-]
AW1	36,5	0,75	3,0	0,75	3,0	0,51	0,43	0,63	0,63
AW2	36,5	0,75	3,0	0,75	3,0	0,99	0,83	0,07	0,99
AW3	36,5	0,75	3,0	0,75	3,0	0,45	0,42	0,24	0,45
AW4	36,5	0,75	3,0	0,75	3,0	0,38	0,36	0,28	0,38
AW5	36,5	0,75	3,0	0,75	3,0	0,45	0,41	0,16	0,45
AW6	**36,5**	**0,75**	3,0	**0,75**	3,0	**0,61**	**0,53**	**0,71**	**0,71**
AW7	36,5	0,75	3,0	0,75	3,0	0,79	0,76	0,62	0,79
AW8	36,5	0,75	3,0	0,75	3,0	0,60	0,58	0,85	0,85
AW9	36,5	0,75	3,0	0,75	3,0	0,42	0,49	0,40	0,49
AW10	36,5	0,75	3,0	0,75	3,0	0,44	0,49	0,56	0,56
AW11	36,5	0,75	3,0	0,75	3,0	0,26	0,26	0,18	0,26
AW12	36,5	0,75	3,0	0,75	3,0	0,49	0,54	0,63	0,63
AW14	36,5	0,75	3,0	0,75	3,0	0,34	0,44	0,76	0,76
IW1	24	1,2	4,7	1,4	4,7	0,67	0,55	0,20	0,67
IW2	24	1,2	4,7	1,4	6,3	0,78	0,67	0,03	0,78
IW3	24	2,0 [2)]	4,4	2,0 [2)]	4,4	0,58	0,55	0,59	0,59
IW4	24	1,2	4,7	1,4	4,7	0,38	0,38	0,16	0,38
IW5	**24**	**1,2**	4,7	**1,4**	4,7	**0,75**	**0,78**	**0,65**	**0,78**
IW6	24	2,0 [2)]	4,4	2,0 [2)]	4,4	0,90	0,80	0,55	0,90
AW-1	36,5	0,75	3,0	0,75	3,0	0,52	0,44	0,63	0,63
AW-2	36,5	0,75	3,0	0,75	3,0	0,97	0,82	0,07	0,97
AW-3	36,5	0,75	3,0	0,75	3,0	0,45	0,41	0,23	0,45
AW-4	36,5	0,75	3,0	0,75	3,0	0,38	0,35	0,28	0,38
AW-5	36,5	0,75	3,0	0,75	3,0	0,46	0,41	0,16	0,46
AW-6	**36,5**	**0,75**	3,0	**0,75**	3,0	**0,61**	**0,51**	**0,71**	**0,71**
AW-7	36,5	0,75	3,0	0,75	3,0	0,82	0,77	0,63	0,82
AW-8	36,5	0,75	3,0	0,75	3,0	0,65	0,61	0,88	0,88
AW-9	36,5	0,75	3,0	0,75	3,0	0,45	0,51	0,42	0,51
AW-10	36,5	0,75	3,0	0,75	3,0	0,44	0,49	0,57	0,57
AW-11	36,5	0,75	3,0	0,75	3,0	0,28	0,27	0,18	0,28
AW-12	36,5	0,75	3,0	0,75	3,0	0,49	0,54	0,63	0,63
IW-1	24	1,2	4,7	1,4	4,7	0,66	0,54	0,22	0,66
IW-2	24	1,2	4,7	1,4	6,3	0,80	0,69	0,03	0,80
IW-3	24	2,0 [2)]	4,4	2,0 [2)]	4,4	0,61	0,73	0,75	0,75
IW-4	24	1,2	4,7	1,4	4,7	0,59	0,55	0,15	0,59
IW-5	**24**	**1,2**	4,7	**1,4**	4,7	**0,82**	**0,83**	**0,63**	**0,83**
IW-6	24	2,0 [2)]	4,4	2,0 [2)]	4,4	0,69	0,61	0,88	0,88

[1)] **Increase of the permanent factor ζ from 0.85 to 1.0;**
[2)] **Calculated value including concrete filling**

The load-bearing capacity verifications are provided for all walls.

The exterior walls AW6 or AW-6, highlighted in bold, have been included under item number 1 and the Interior walls IW5 or IW-5 verified under item number 2.

A.2.4 Fire design according to DIN EN 1996-1-2

The requirement for load-bearing walls in buildings of building class 4 (*h* ≤ 13 m, top edge of finished floor) is as follows according to the State Building Code
"highly fire-retardant" and can be used with classifications ≥ F60 or ≥ REI60
be fulfilled.

The utilisation factor becomes

$$\alpha_{fi} = \frac{N_{Ed,fi}}{N_{Rd,fi}}$$

is determined. According to Eq. (43), $N_{Rd,fi}$ = 1.176 - N_{Rd}

The effect in case of fire becomes

$$N_{Ed,fi} = \eta_{fi} - N_{Ed}$$

is determined. The more precise determination of η_{fi} is carried out according to Eq. (39)

<u>Example 1: External wall AW 6 on the ground floor (Item 1)</u>

N_{Ed}/N_{Rd} = 0,71
existing α_{fi} = η_{fi} - N_{Ed} / (1.176 - N_{Rd}) = 0.64 - 0.71 / 1.176 = 0.386
from approval: perm α_{fi} = 0.7 for wall thickness *t* = 365 mm = $t_{vorh.}$

Proof:

1. existing *t* = 365 mm = min *t* (REI90)
2. existing α_{fi} = 0.386 < perm α_{fi} = 0.7.

Evidence provided.

<u>Example 2: Internal wall IW-5 on the ground floor (Item 2)</u>

N_{Ed}/N_{Rd} = 0.83
existing α_{fi} = η_{fi} - N_{Ed} / (1.176 - N_{Rd}) = 0.635 - 0.83 / 1.176 = 0.448 from approval: perm α_{fi} = 0.61 for wall thickness *t* = 175 mm

Proof:

1. existing *t* = 175 mm < $t_{vorh.}$ = 240 mm
2. existing α_{fi} = 0.448 < perm α_{fi} = 0.61.

Evidence provided.

A.2.5 Pos. 3: Exterior basement wall

System

Double-sided, single-shell basement outer wall.
The window parapet on the ground floor is set as a non-load-distributing surface (i.e. as a non-load-bearing wall).

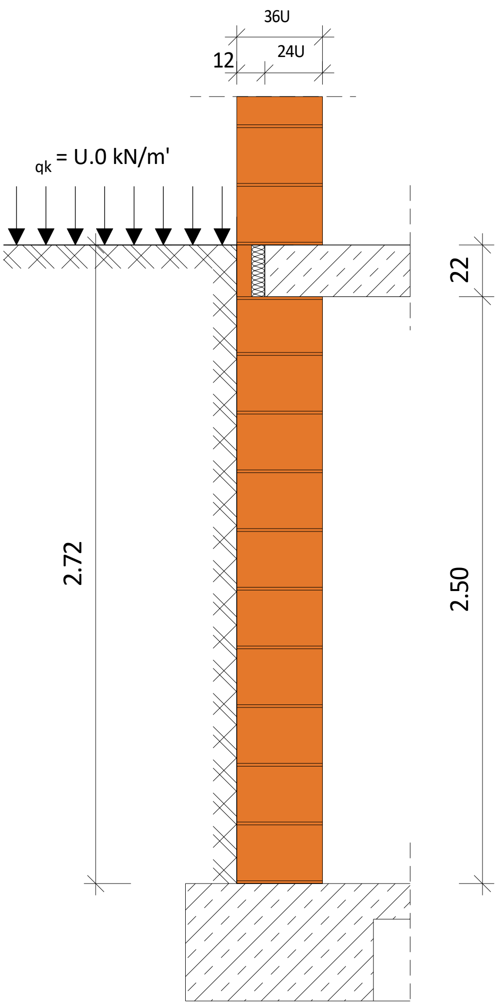
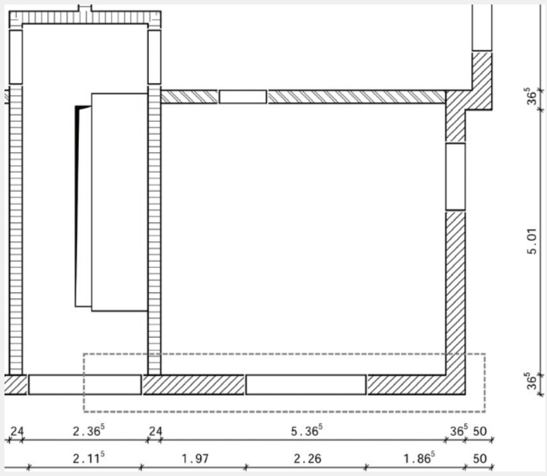


Figure A.16: Section through the basement wall

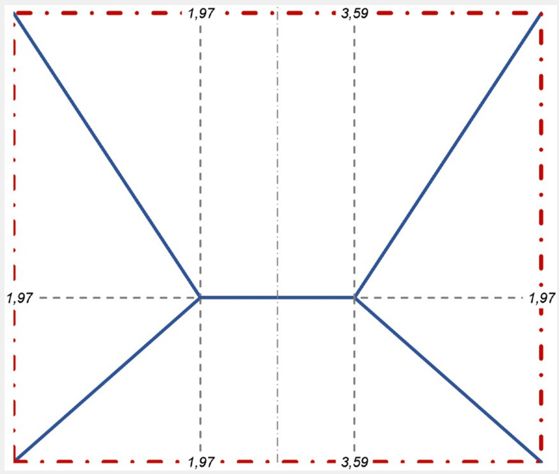
Ground floor plan:



Ceiling support widths:

l_x = 5.61 m
l_y = 5.34 m

Load distribution area of the ceiling loads:



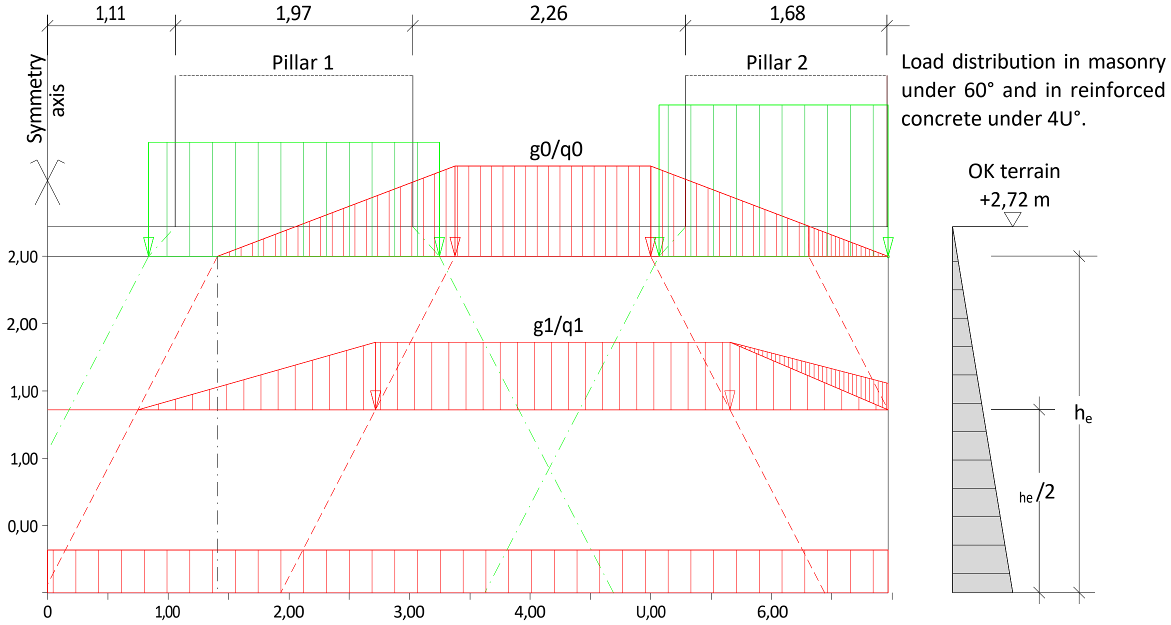


Figure A.17: Wall view with load distribution and normal force curve

Component data	
Brick compressive strength class	12
Brick density class	0,75
Masonry mortar	Thin-bed mortar
Wall thickness *d*	365 mm
Plaster thickness $_{dP}$	35 mm (20 + 15)
Wall length *b*	5,61 m
Clear wall height *h*	2,50 m
Backfill height $_{he}$	2,72 m
Ceiling thickness $_{db}$	220 mm
Ceiling support width l_1	5,34 m
Live load on terrain $_{qk}$	5.0 kN/m²

b = 0,24 / 2 + 5,365 + 0,245 / 2
= 5,61 m

Load composition		
Roof loads	**Constant load $_{gDa}$**	**5.10 kN/m**
	Variable load $_{qDa}$	**2.50 kN/m**
Ceiling loads	gPlate	5.50 kN/m²
	gPlaster/covering	1.80 kN/m²
	Permanent load $_{gDe}$	**7.30 kN/m²**

Load composition		
	Payload category A2	1.50 kN/m²
	Partition wall surcharge	1.20 kN/m²
	Variable load q_{De}	**2.70 kN/m²**
Dead load walls	g_{MW}	3.10 kN/m²
	g_{Putz}	0.43 kN/m²
	Permanent load g_{Wa}	**3.53 kN/m²**
Weight floor	γ_e	18 kN/m³

cf. item 1

Load at the wall head

From jamb and roof

g_0 = 5.1 kN/m
q_0 = 2.5 kN/m

from FE calculation ceilings ground floor to 3rd floor (mean value)

g_{De} = 14.85 kN/m
q_{De} = 20.73 - 14.85 = 5.88 kN/m

Mean value over lintel

Loads from wall pillar 1 and 2 at wall base EG

(The wall pillar loads result from taking into account the static calculation of the existing window lintels).

from auxiliary calculation:

l_1 = 1,97 m G_1 = 149.54 kN Q_1 = 52.15 kN

l_2 = 1,68 m G_2 = 174.26 kN Q_2 = 55.17 kN

The load transfer of the ceiling and wall loads from the ground floor to the first floor to the basement wall is carried out via the wall pillars.

l_1 and l_2 are the lengths of the wall pillars on the ground floor.

from ceiling above KG

$gDe_{,KG} = {}_{gDe} - gDe_{;KG} / gDe_{,EG-DG}$ = 14.85 kN/m

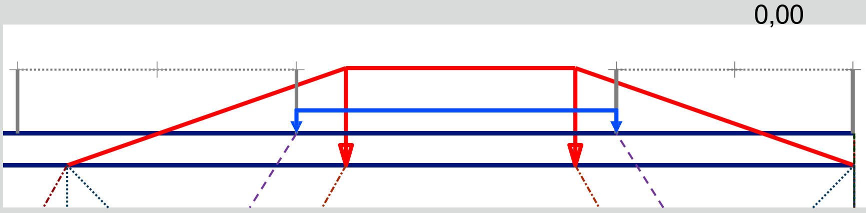
$q_{De,KG} = q_{De}$ - 1.50 / 2.70 = 5.88 - 1.50 / 2.70 =3 .27 kN/m

Only the live load share without partition surcharge is used, i.e.:

q = 1.50 kN/m².

Area lengths of the ceiling load in m:

$lanf_{,0}$	lanf	lanf	lende	$lende_{,0}$
1,41	1,97	1,62	1,97	0,00



Ceiling load from basement ceiling (from simplified load introduction area)

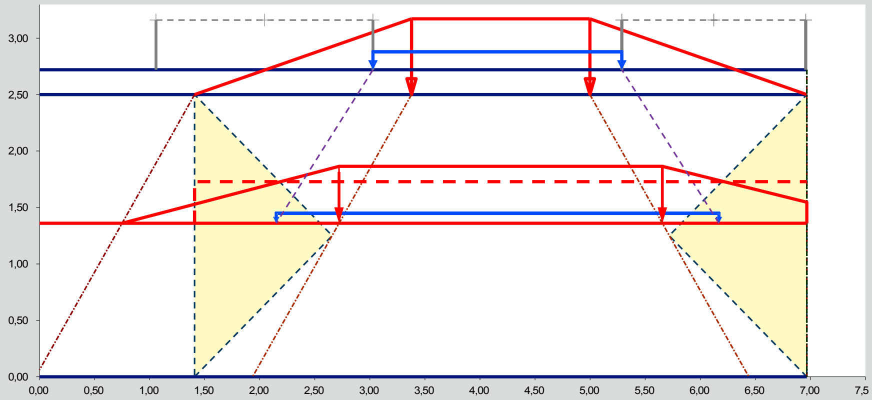
from parapet under wall opening on ground floor and window element

g_{Br} = 3.53 - 0.25 + 1.25 = 2.13 kN/m

Plinth height = 0.25 m

Internal forces at half backfill height

from KG ceiling load



Normal force curve at $h/2_e$

A load distribution of 60° set.

Due to the existing projecting cross walls, verification of the minimum load in the hatched wall area is not required.

G_D = 14.85 - (0.5 - 1.97 + 1.62 + 0.5 - 1.97) = 53.5 kN
Q_D = 3.27 - (0.5 - 1.97 + 1.62 + 0.5 - 1.97) = 11.7 kN

g_D = 52.3 / (1.97 + 1.62 + 1.97) = 9.59 kN/m
q_D = 11.7 / 5.56 = 2.10 kN/m

l_{ende} cf. area lengths ceiling load

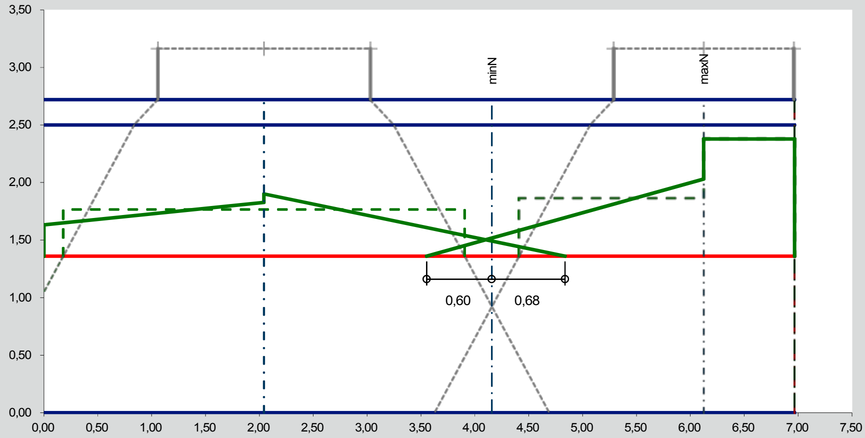
due to redistribution of uniform load - trapezoidal load (see sketch)
$g_{D,T}$ = 1,143 - 9,59 = 10.96 kN/m
$q_{D,T}$ = 1,143 - 2,10 =2 .40 kN/m

At h_e /2, the load distribution results in an average block load of q_m = 9.59 + 2.10 = 11.69 kN/m,
or, as a result of redistribution, a maximum applicable equalisation load of
q = 10.96 + 2.40 = 13.36 kN/m.

from window parapet:
l_3 = 2 - (1.14 - tan(30) + 0.20) + 2.26 = 4,02 m
g_{Br} = 2.13 - 2.26 / 4.02 = 1.20 kN/m

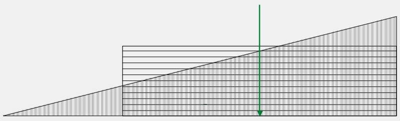
Loads from wall piers 1 and 2:

With a block-shaped load distribution for the pier loads, there is no overlapping of the loads from the two pier loads here. The verification of the minimum superimposed load can then only be carried out taking into account the loads from the basement ceiling, the parapet and the dead load of the basement wall. Since the verification cannot be carried out with this low normal force, a linear normal force curve with a longer distribution width is used.



Distribution of the normal force from wall pier loads at half the backfill height with consideration of a load propagation in the masonry at 60°. In the area of the floor slab, a load propagation of 45° is assumed.

Equal area redistribution of the block load with equal position of the resultant:



Verification at parapet centre (x = 4.16 m)

$g_{lin,Pf}$ = 53,60 - 0,68 / 2,79 + 67,54 - 0,60 / 2,58 = 28.77 kN/m

$q_{lin,Pf}$ = 18,69 - 0,68 / 2,79 + 21,38 - 0,60 / 2,58 = 9.53 kN/m

$n1_{,Ed,min}$ = 1.0 - ($\sum g_i$ + $n_{g,W}$)
= 1,0 - (9,59 + 1,20 + 28,77 + 4,02) = 43.6 kN/m

Load lengths

l_{lin} = 1,5 - l_{block}

$l_{lin,Pf1,re}$ = 2,79 m

$l_{lin,Pf2,li}$ = 2,58 m

Normal force at half wall height $n_{1,Ed}$

$n1_{,Ed,min}$ = 1.0 - ($\sum g_i$ + n)$_{g,W}$

= 1,0 - (9,59 + 1,20 + 28,77 + 4,02) = 43.6 kN/m

$n1_{,Ed,max}$ = 1.35 - ($\sum g_i$ + $n_{g,W}$) +1.5 - $\sum q_i$

= 1,35 - (9,59 + 1,20 + 68,1 + 4,02) + 1,5 - (2,10 + 21,6)
= 111,93 + 35,55 = 147.5 kN/m

$n_{g,W}$ = (2.50 - 2.72/2) - 3.53
= 4.02 kN/m

with consideration of the pier loads g_2 and q_2

Dimensioning

Review of the general conditions for the application of the unified method according to DIN EN 1996-3, 4.5			
Criterion	**Request**	**Actual value**	**Comment**
Minimum wall thickness	$t \geq 240$ mm	365 mm	complied with
Clear storey height	$h \leq 2.60$ m	2,50 m	complied with
Permissible backfill height	$h_e \leq$ 1,15 - 2,50 = 2,875 m	2,72 m	complied with
Maximum traffic load on terrain	$q_k \leq 5$ kN/m²	5.0 kN/m²	complied with
Distance of concentrated loads *Q* > 15 kN from basement wall	$a \geq 1.50$ m	-	complied with
Training KG ceiling	Basement ceiling acts as a disc and can absorb earth pressure forces		complied with
Terrain	Terrain surface does not rise		complied with

Verification at half backfill height

$N_{Ed,max} \leq N_{Rd} = \frac{t \cdot b \cdot f_d}{3}$

$N_{Ed,min} \geq N_{lim,d} = \frac{\rho \cdot b \cdot h \cdot h_e^2}{\beta \cdot t}$

with

characteristic compressive strength	f_k	= 3.0 N/mm²
Permanent standing factor	ζ	= 0,85
Partial safety factor	γ_M	= 1,50

$f_d = \frac{0.85 \cdot 3.0}{1,5} = 1.70$ N/mm²

$N_{Ed,max}$ according to equation (47) $N_{Ed,min}$ according to equation (48)

f_k according to approval

Rated resistance $n_{1,Rd}$ at half the backfill height

$n_{1,Rd} = \frac{0.365 \cdot 1.70}{3} \cdot 1000 = 206.8$ kN/m

Minimum value of the required normal force $N1_{,lim,d}$

$n1_{,lim,d}$	$= \frac{18 \cdot 2{,}5 \cdot 2{,}6^2}{20 \cdot 0{,}365}$	= 41.7 kN/m

Evidence

$n1_{,Ed,min}$ = 43.6 kN/m ≥ $n1_{,lim,d}$ = 41.7 kN/m
Condition 1 is met

$n1_{,Ed,max}$ = 147.5 kN/m ≤ $n_{1,Rd}$ = 206.8 kN/m
Condition 2 is met

ρ_e = 18 kN/m³
β = 20 for $b_c \geq 2 \cdot h$

b_c : Distance between bracing cross walls or other bracing elements (according to DIN EN 1996-3:2010-12, Ab-section 4.5(2))

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