ESECMaSE

Enhanced **S**afety and **E**fficient **C**onstruction of **Ma**sonry **S**tructures in **E**urope

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ESECMaSE

Enhanced Safety and Efficient Construction of Masonry Structures in Europe

Horizontal Research Activities Involving SMEs

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Work Package N°: 7

D 7.3

Identification of suitable behaviour factors for masonry members under earthquake loads

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PU	Public	X
PP	Restricted to other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the consortium (including the Commission Services)	
CO	Confidential, only for members of the consortium (including the Commission Services)	

Contents:

1. Introduction

Within ESECMaSE a significant number of static-cyclic tests and several dynamic tests have been conducted. The detailed results are depicted in the reports for the deliverables 7.1 (static cyclic tests on masonry walls [1-3]) and 7.2 (pseudodynamic tests on walls and on full scale buildings as well as shaking table tests [4-7]). One of the main aims of ESECMaSE is to provide information on the behaviour of masonry structures subject to seismic action.

The seismic behaviour of a structure, e.g. a building, depends on the strength of the structure as well as on the deformation capacity. Roughly spoken, the product of these two quantities determines the resistance of a structural system against seismic action. In addition the energy dissipation capacity of a structure is important (area and shape of hysteresis loops, equivalent viscous damping).

The lateral force resistance has been measured in the experiments being conducted within ESECMaSE and improved models have been proposed for computing the force resistance of masonry walls (see work packages 4 and 9). In order to express the seismic deformation behaviour of masonry walls, different measures are conceivable:

- ductility ratio μ,
- ultimate drift angle θ_u, expressed as maximum top deflection in relation to storey height,
- behaviour factor q,
- capacity curve, depicting e.g. top deflection versus top deformation.

In this report it will also be shown that the effect of ductile behaviour alone, which may be expressed by means of ductility and the q-factor, is not suited to explain the observed favourable behaviour in the full scale experiments.

2. Evaluation of ductility μ and behaviour factor q

2.1. General

Within the work of ESECMaSE, intensive discussions on the evaluation of the behaviour factor q have taken place. A first comprehensive report on possibilities to determine the q-factor has been submitted in April 2008 by Magenes and Morandi [10]. In deliverable 7.2c [6], a first proposal for the evaluation of q-factors from the shaking table tests is contained.

However, care should be taken when evaluating reliable values of the behaviour factor. The behaviour factor q as used in Eurocode 8 and pertinent national code regulations provides a simple approach enabling to transfer the response of a given structure with a certain nonlinear behaviour to the response of a linear elastic idealization of the same structure. As a consequence of this definition, it is important to

- observe and take into account the response of the whole structural system (e.g. a building) and not only of a part of it (e.g. a masonry wall as it has been used in an experiment). Local and global response may exhibit different ductility.
- make reference to the idealization of the structural system as linear elastic single or multiple degree of freedom oscillator. The key parameter is the elastic stiffness used in the analysis.
- observe and take into account the resistance criteria as adopted actually in the structural design checks in comparison to the real resistance. This refers to the aspect of overstrength and possible redistribution of forces in case of complex multiple wall systems.
- consider the influence of the shape of the response spectrum in conjunction with the fundamental period(s) of the structure on the actual structural response and the capacity curve.

Furthermore, the structural behaviour of an integral structural system may be influenced by a number of nonlinear effects. Examples are:

- action of the wall system not only as sequence of cantilever walls but as frame system with moment stiff connections,
- uplift of masses due to rocking motion (gaping) of walls, and thus, vertical inertia forces [8,9], and vertical forces on wall due to elastic restraint,
- redistribution of normal forces between different walls due to different uplift behaviour and / or frame action (walls + lintels or slabs),
- redistribution of horizontal shear forces between different walls in the storey,
- influence of multidirectional excitation,
- interaction between adjacent walls, especially action of orthogonal walls as composite cross section,
- non symmetric resistance of composite cross sections (e.g. T-shape) after onset of e.g. flexural cracking.

In general, the simplifying idealization of the structure as a linear elastic system is not able to capture all these effects. Even nonlinear models will not always be able to represent all the relevant effects for which examples are mentioned above. Hence, it will be difficult to come up with exact values for q-factors for masonry structures.

Furthermore, a large degree of scatter overlaying the experimental results on masonry elements has to be observed, especially with respect to the deformation capacity. Besides the scatter of material and specimen properties, reasons for this are the choice of the test procedure and the criteria for the abortion of a static cyclic test as well as of a pseudo dynamic test.

Nevertheless, it is important to identify the main influence parameters on the deformation capacity of masonry walls. From the preceding reports, especially in this deliverable and in deliverable 4, it becomes clear that the type of failure determines the available ductility and the ultimate deformation to a large extent. Other parameters significantly affecting ductility and ultimate deformations are the normal force and the aspect ratio (i.e. height to length ratio) of the wall.

2.2. Overview about Experimental Results

The results of the static cyclic tests performed at University Kassel [1], University Pavia [3], and Technical University Munich [2] are shown in the annex. Furthermore, the results of the pseudodynamic tests conducted in Kassel [4] are included. The results of the pseudodynamic tests at TU Munich [5] are not listed in this table since these tests cannot directly be compared with the static cyclic tests. The reason is the fact that no constant fixture at the cap of the wall has been used in the Munich tests. Instead, the cap moment has been derived from the MDOF-system represented by the test substructure and the computer model.

2.3. Influencing Parameters on Ductility

As could be expected, the ductility strongly depends on the type of failure. Fig. 1 to 3 show the ductility as obtained from the tests dominated by

- bending rocking failure,
- sliding in joints
- tension failure inside of units.

It should be noted, that in many cases combinations of failure types could be observed. Furthermore, the three above mentioned principal types may be differentiated further. Also additional failure types such as e.g. rocking of the units themselves may be considered.

However, with respect to be able to find relevant tendencies within sufficiently large sets of experimental data, the number of considered failure types must be limited to those mechanisms which mainly determine the ultimate deformation.

In the following graphs, the observed ductility ratio is plotted versus stress exploitation $\alpha=\sigma_k/f_k$; aspect ratio l_w/h_w, as well as versus l′ which is the product of both quantities

$$l' = \alpha \cdot l_w / h_w \tag{1}$$

with

$$\alpha = \frac{\sigma_v}{f_k} \tag{2}$$

σ_v denotes the stress due to the axial force.

It should be noted, that ductility values greater than 10 are not depicted in order to allow the visualization for the relevant range of q > 1.5.

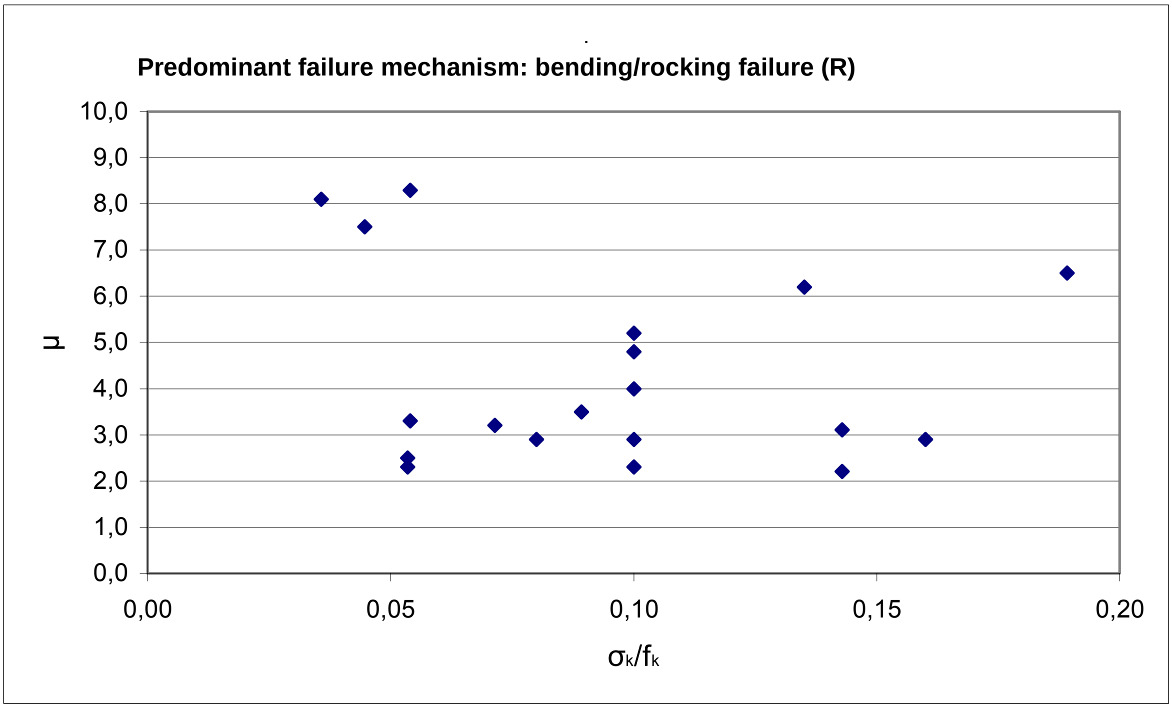


Fig. 1a: Ductility-Vertical Stress relationship for specimens with predominant bending/rocking failure

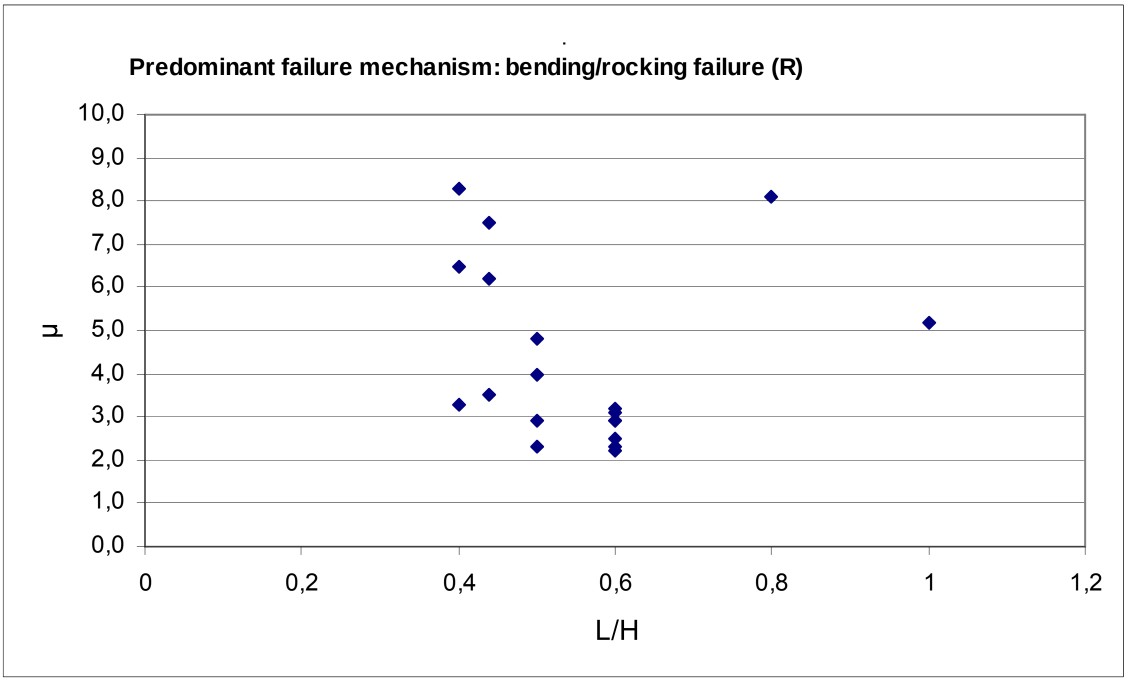


Fig. 1b: Ductility as a function of aspect ratio for specimens with predominant bending/rocking failure

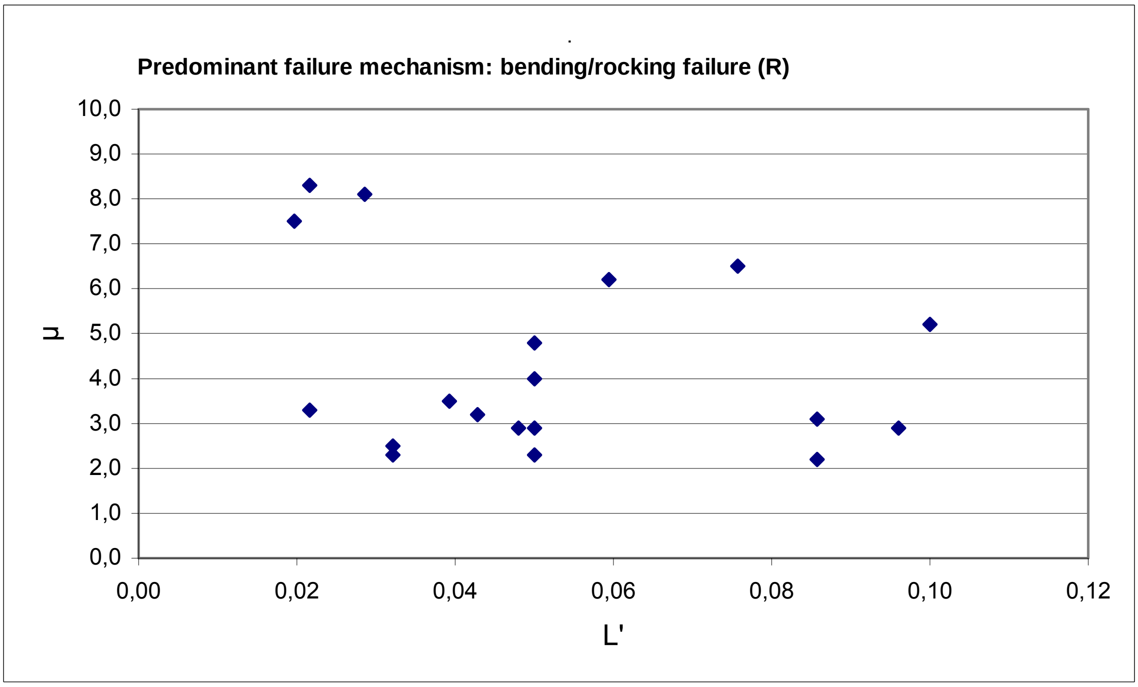


Fig. 1c: Ductility as a function of l´ for specimens with predominant bending/rocking failure

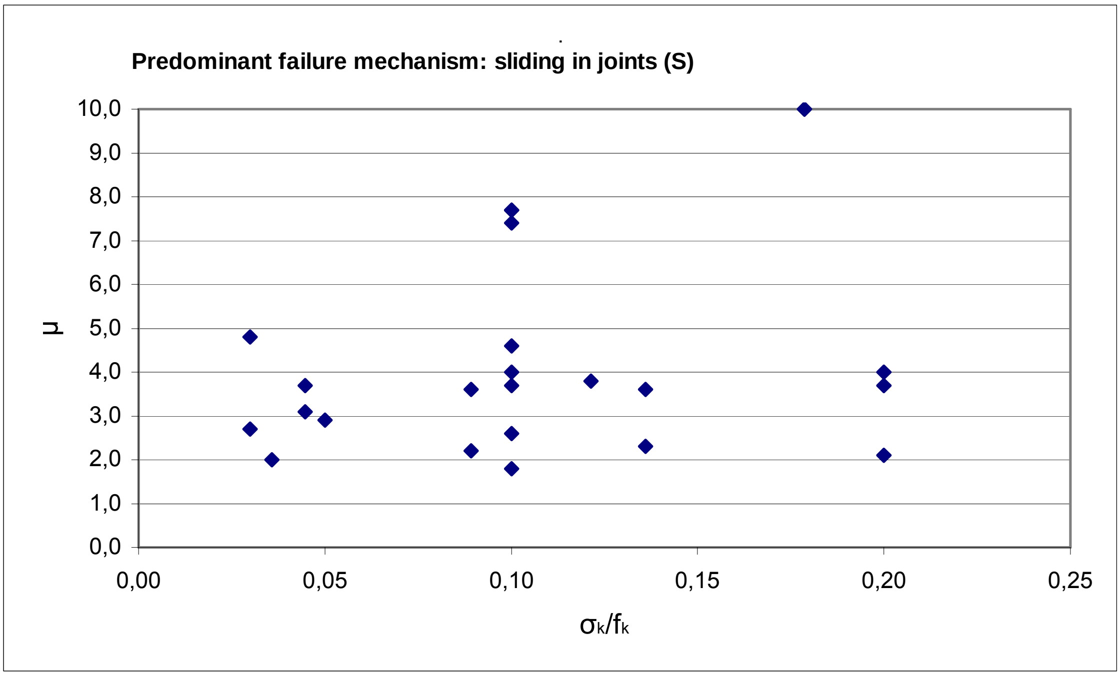


Fig. 2a: Ductility as a function of vertical stress exploitation for specimens with predominant sliding in joints

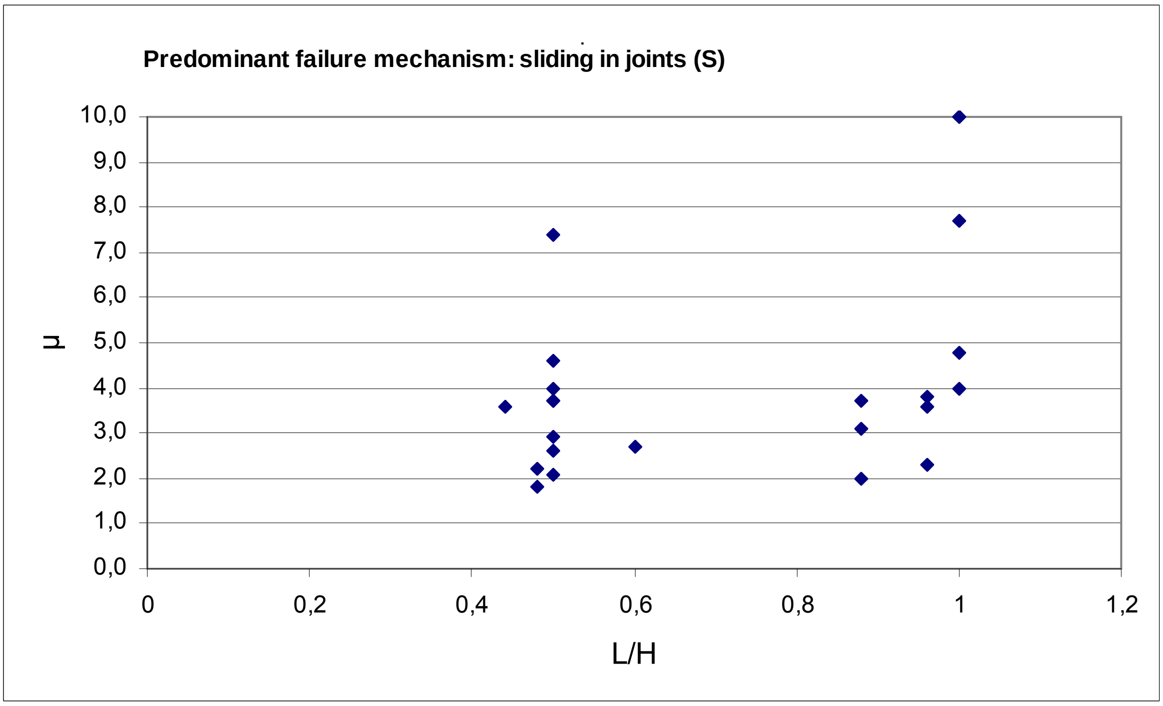


Fig. 2b: Ductility as a function of aspect ratio for specimens with predominant sliding in joints

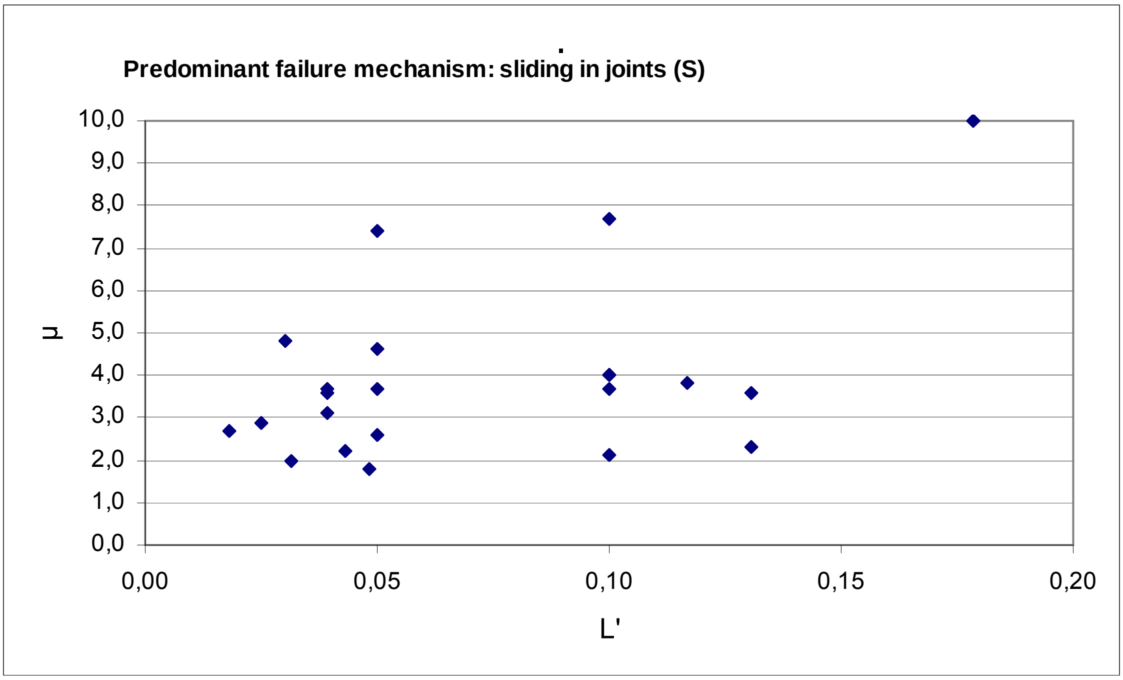


Fig. 2c: Ductility as a function of l ´ for specimens with predominant sliding in joints

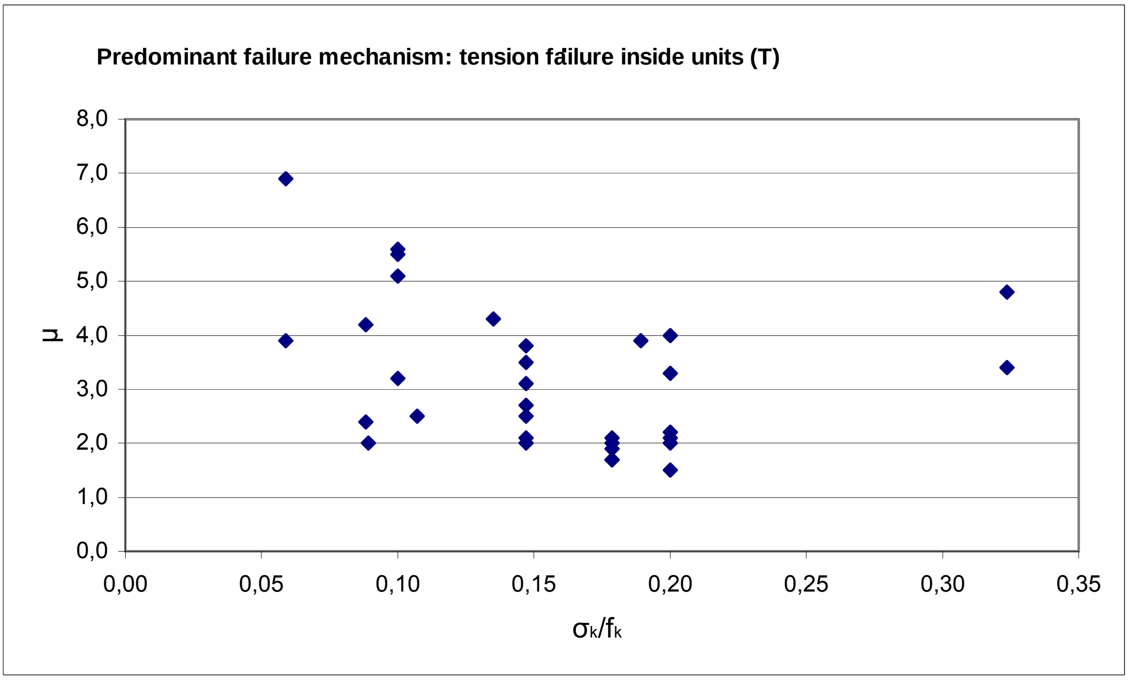


Fig. 3a: Ductility as a function of vertical stress exploitation for specimens with predominant tension failure inside units

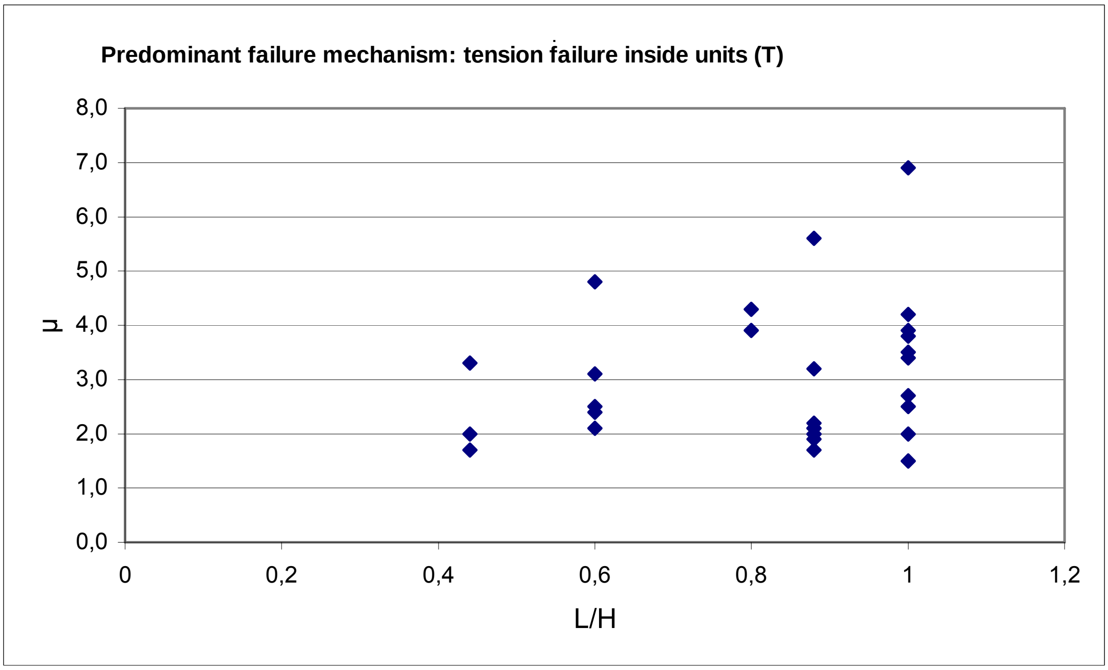


Fig. 3b: Ductility as a function of aspect ratio for specimens with predominant tension failure inside units

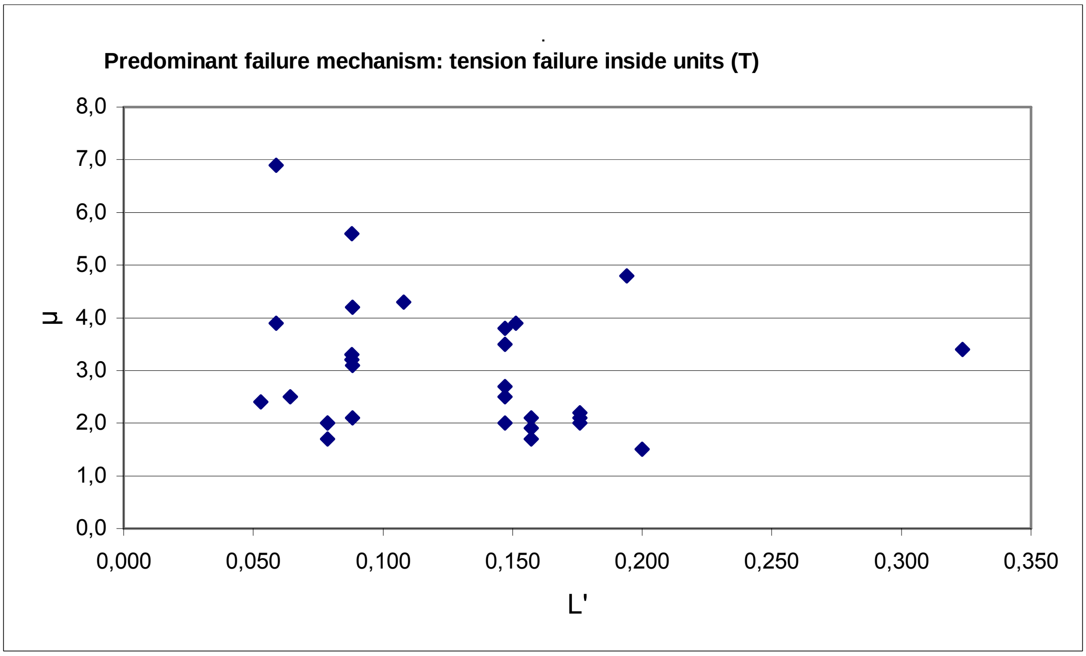


Fig. 3c: Ductility as a function of l´ for specimens with predominant tension failure inside units

2.4. Evaluation of Ultimate Storey Drift Angle

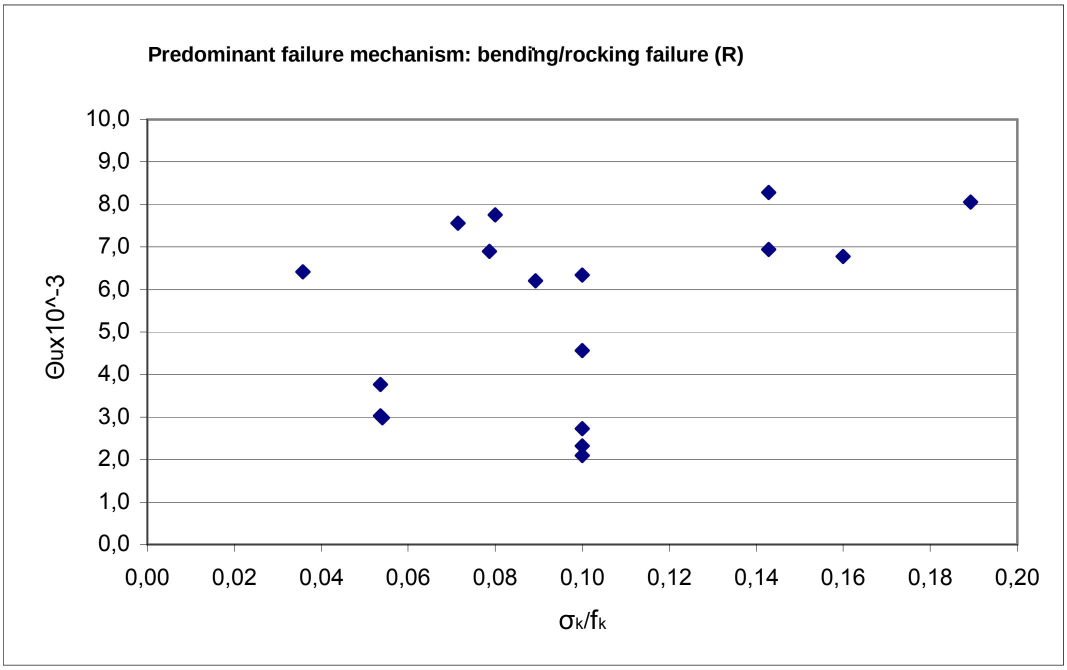


Fig. 4a: Interstorey drift angle versus vertical stress relationship for specimens with predominant bending/rocking failure

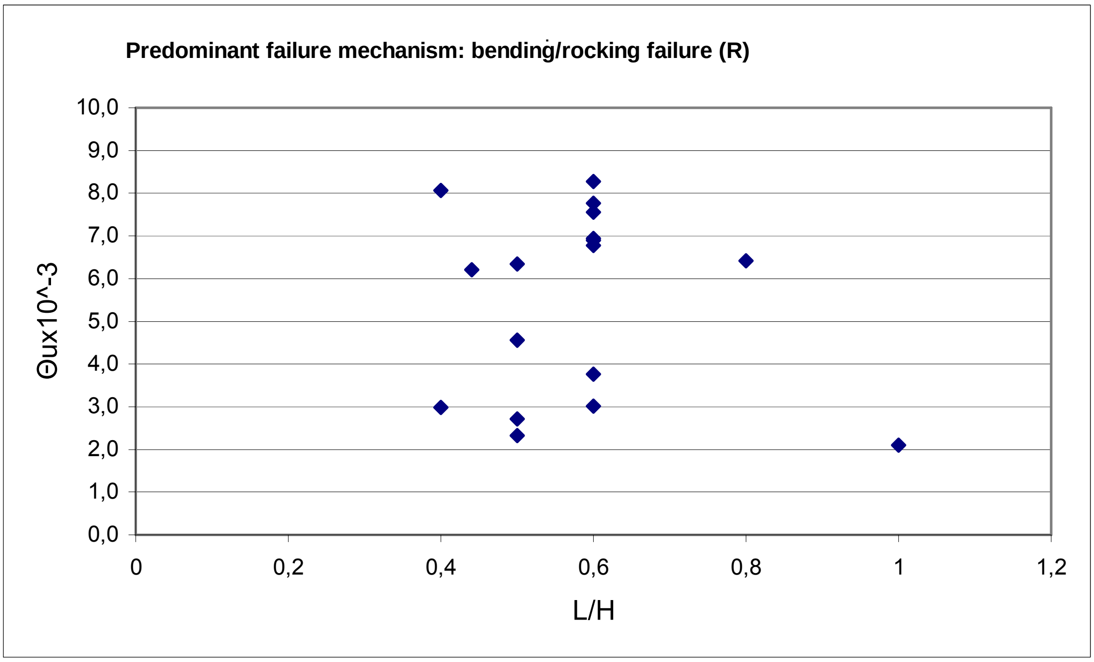


Fig. 4b: Interstorey drift angle as a function of aspect ratio for specimens with predominant bending/rocking failure

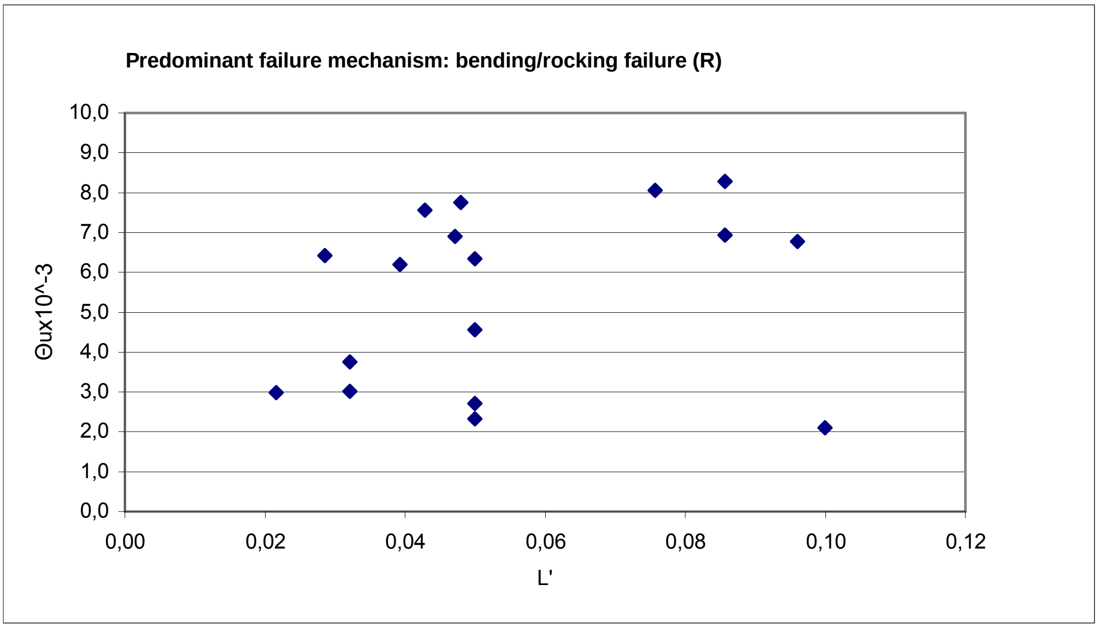


Fig. 4c: Interstorey drift angle as a function of l´ for specimens with predominant bending/rocking failure

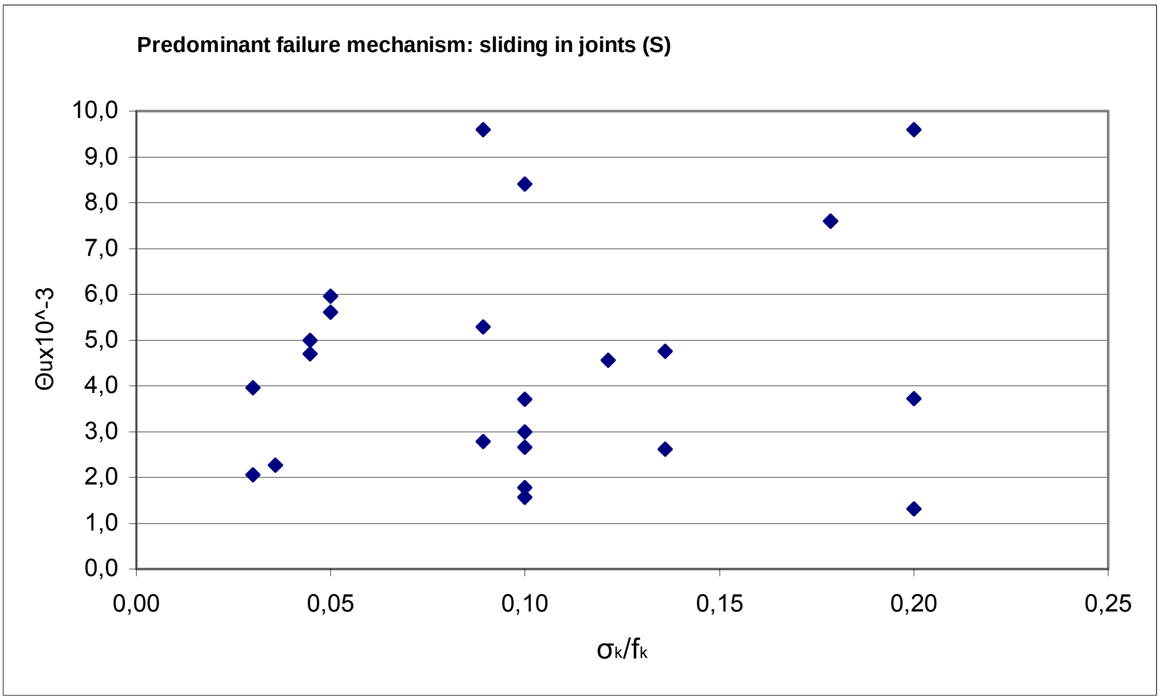


Fig. 5a: Interstorey drift angle as a function of vertical stress exploitation for specimens with predominant sliding in joints

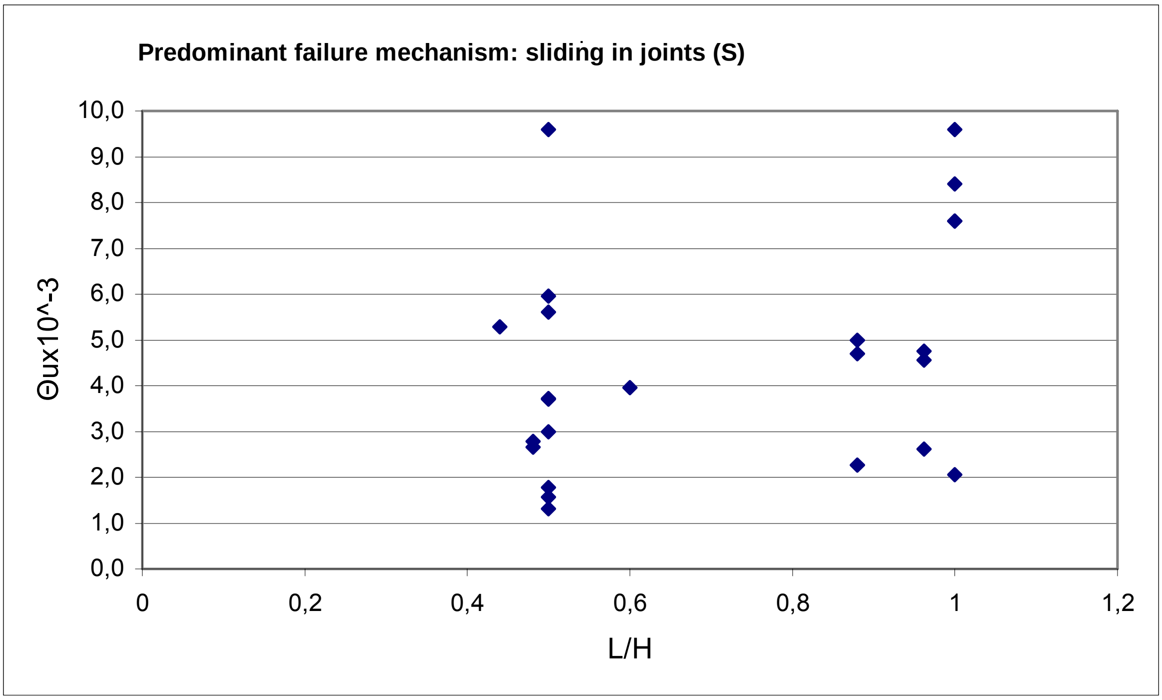


Fig. 5b: Interstorey drift angle as a function of aspect ratio for specimens with predominant sliding in joints

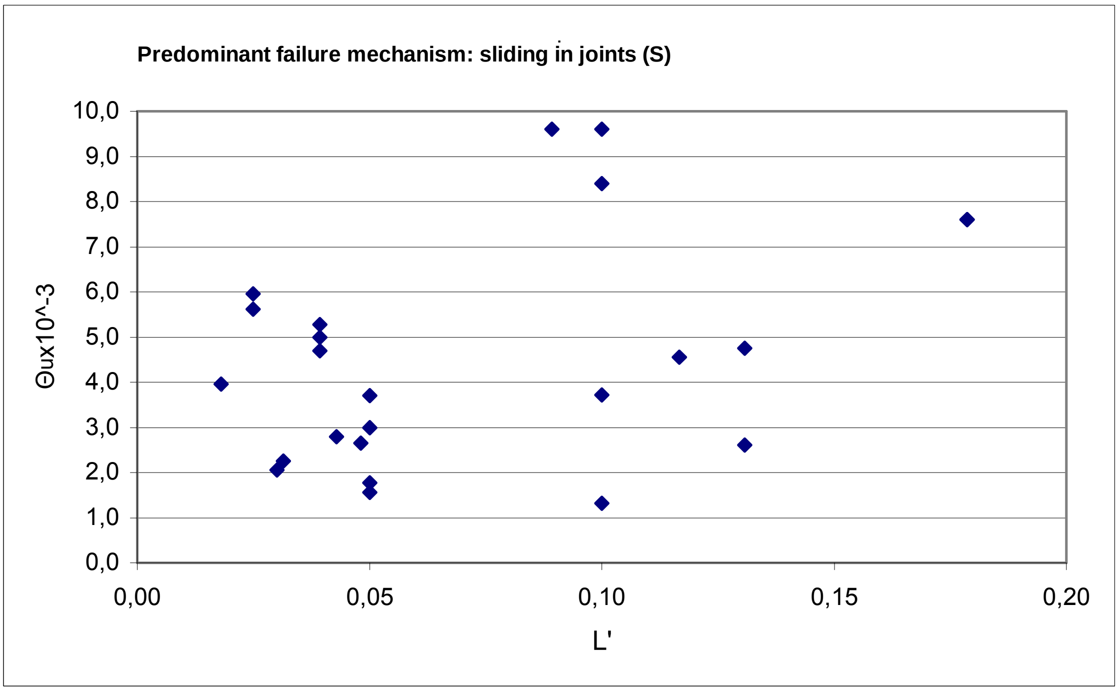


Fig. 5c: Interstorey drift angle as a function of l´ for specimens with predominant sliding in joints

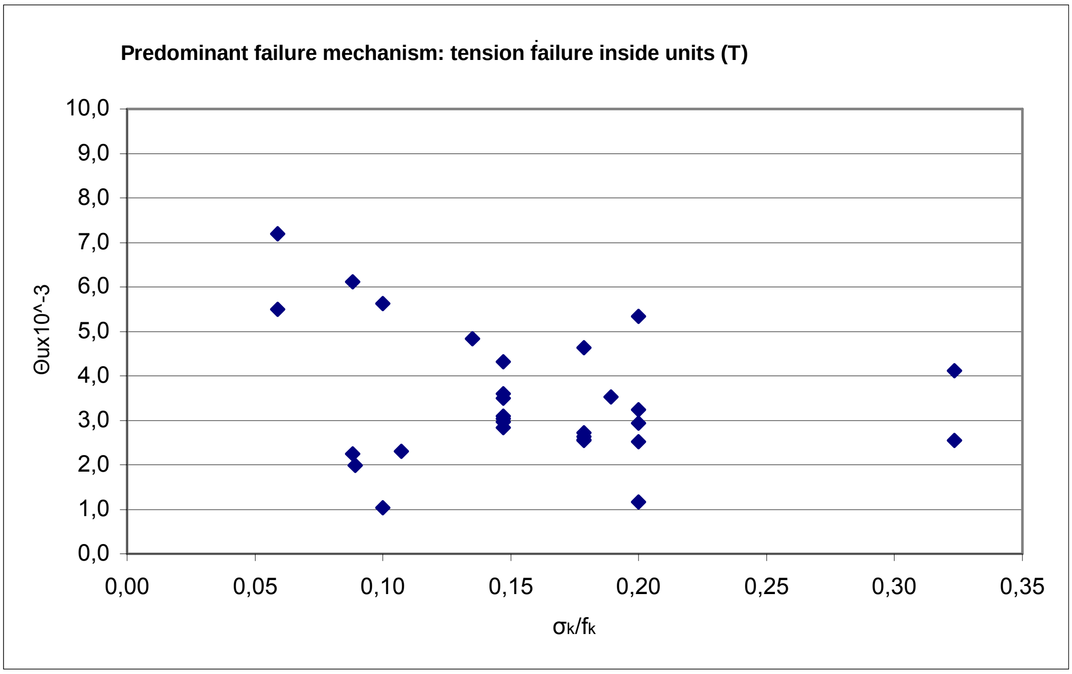


Fig. 6a: Interstorey drift angle as a function of vertical stress exploitation for specimens with predominant tension failure inside units

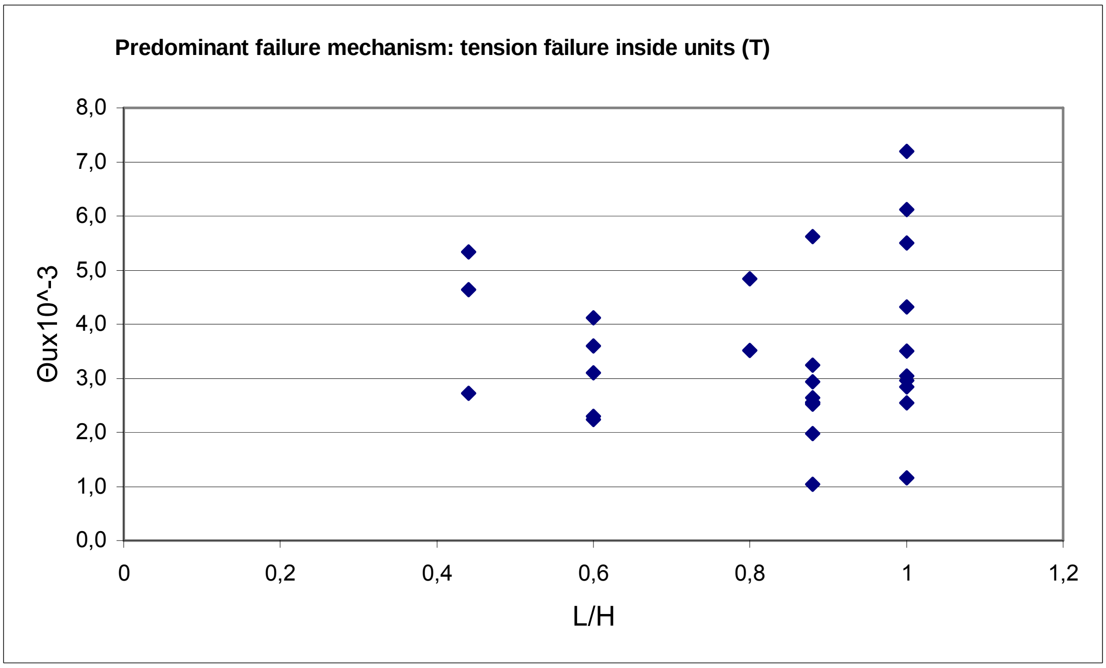


Fig. 6b: Interstorey drift angle as a function of aspect ratio for specimens with predominant tension failure inside units

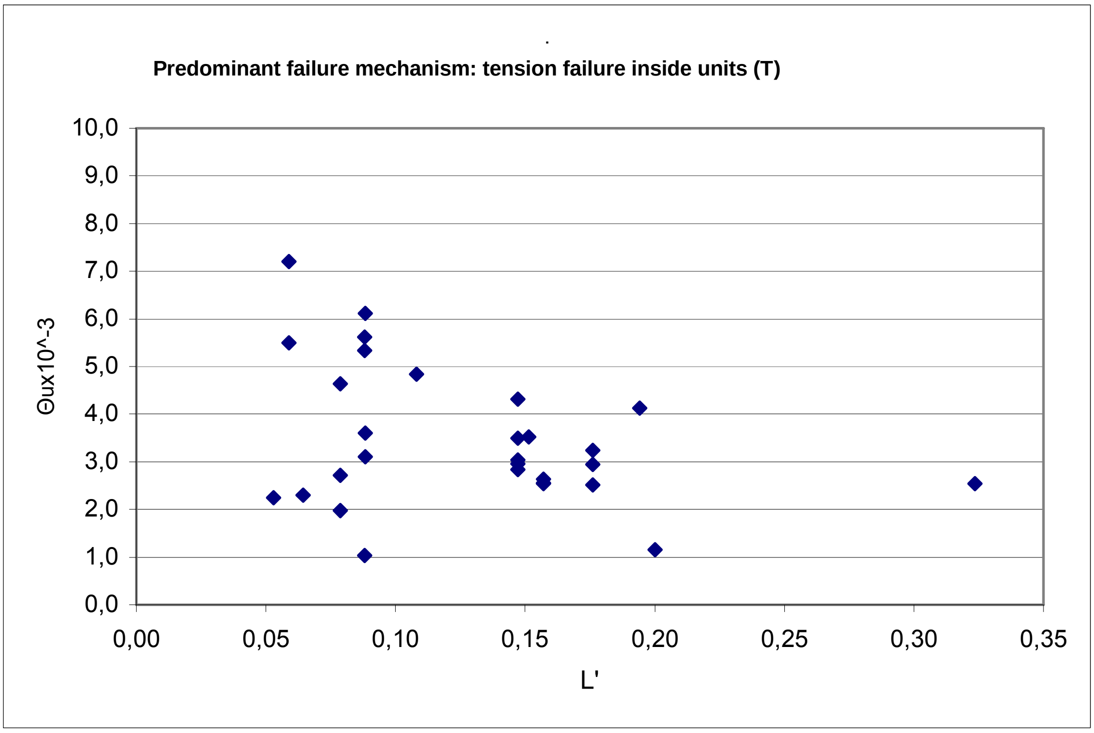


Fig. 6c: Interstorey drift angle as a function of l´ for specimens with predominant tension failure inside units

3. Behaviour Factor q

3.1. Estimation of behaviour factors from static cyclic tests

The definition of the q-factor is illustrated in Fig. 7. As already lined out in chapter 2.1, there is, however, not a simple, unique relationship between the displacement interstorey drift angle of a masonry wall and the behaviour factor q of the whole structure. Instead, the behaviour of the structure depends on many other circumstances as well.

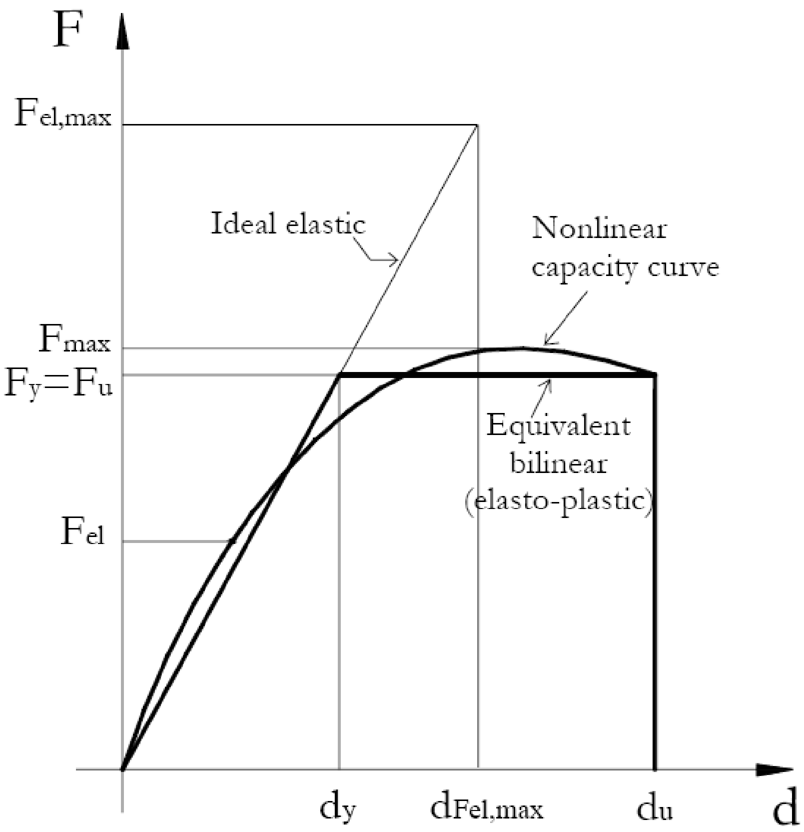


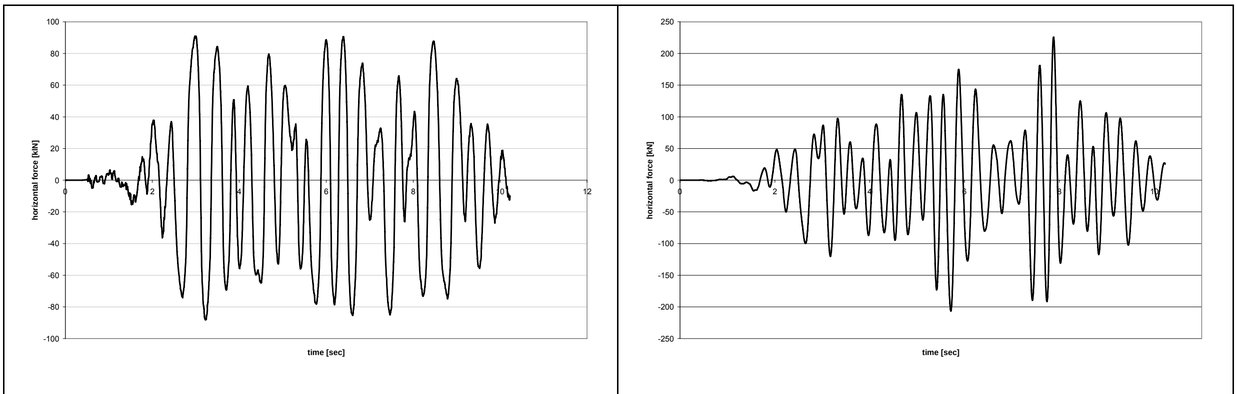
Fig. 7 Parameters for the definition of behaviour factor q (taken from [10])

3.2. Evaluation of Pseudo dynamic Wall Tests

For the pseudodynamic tests performed at Kassel University the behaviour factors were evaluated as follows. Therefore, the quotient of the maximum horizontal force of the test H_{max} (or F_y in fig. 7) and the elastic horizontal force H_{el} (or $F_{el,max}$) – calculated with the initial stiffness of the specimen – has to be determined according to equation (3).

$$q = \frac{H_{el}}{H_{max}} \tag{3}$$

The force H_{el} was calculated by the initial stiffness of the specimen and the given acceleration-time-history. Figure 8 shows exemplarily the force-time-behaviour of an AAC wall at a maximum acceleration of 0.28 · g (a) and the elastic calculated-force-time behaviour (b).



a) b)

Fig. 8: force-time-behaviour of wall No. 43 (see Annex) at 0.28 · g, a) test, b) elastic

It has to be kept in mind that the behaviour factor determined in this way depends on the execution of the tests. In the kind of test as performed at University Kassel it is expected that the wall in the upper story behaves in the same way as the tested wall. This may lead to an overestimation of the energy dissipation of the system. So the q-factors might be a little bit lower as calculated above.

In figure 9 the behaviour factors of all pseudodynamic wall tests carried out at University Kassel are displayed versus l′ as defined previously. The test carried out with calcium silicate units and the lowest l’ failed due to an error in the testing procedure, resulting in a low q-factor. Without this failure in the testing procedure a higher behaviour factor q could be expected.

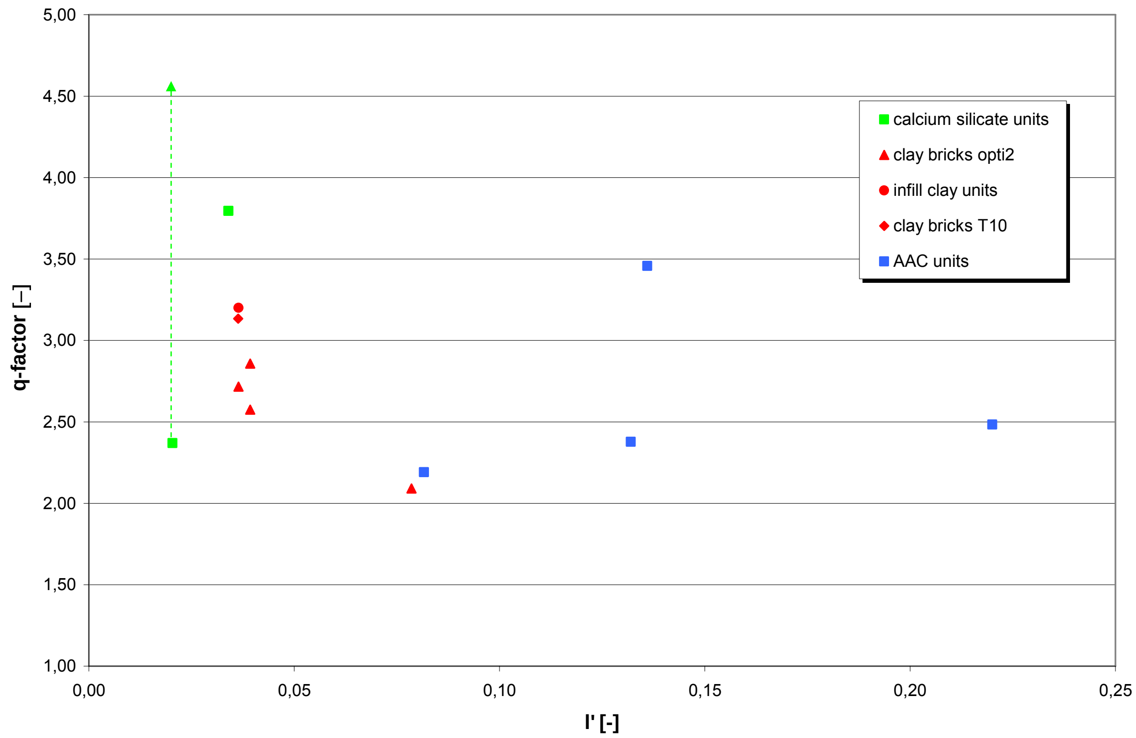


Fig. 9: q-factors of pseudodynamic tests (from University Kassel)

3.3. Evaluation of Shaking Table tests

In deliverable 7.2c a first proposal to estimate the behaviour factors from the test data has been proposed by the authors of this deliverable. This proposal considers the ratio of the ultimate ground acceleration possible for each test specimen in relation to the codified ground acceleration according to the German National code DIN 4149. Although the specimens were designed to study a typical layout of a terraced house in Germany assuming a minimum possible wall length of the main shear wall in such as building, the tests have not been full scale models of the real building.

The design of the test specimen has followed the aim to simulate the vertical stresses in the shear wall of a real house. However, the ground plan area of the test specimen had to be limited and is much smaller than that of the real house in full scale. In turn, the horizontal inertia forces of a building are also smaller, approximately following the same ratio, when the same level of horizontal accelerations is considered.

Thus, it may be misleading to base the calculation of the behaviour factor q on the ratio of experimental ultimate accelerations to the maximum ground acceleration in the most unfavourable earthquake zone according to a code regulation.

Instead, it is proposed to express the q-factor as the ratio of lateral forces obtained form the test in relation to the lateral force obtained for the same acceleration input using linear elastic analysis. Using this definition of q, the following behaviour factors are obtained for the dynamic tests performed on the shaking table of the National technical University Athens:

Table 1: q-factors due to Shaking Table Tests

Speci-	***Type***	***ue+***	***ue-***	***Fy+***	***Fy-***	***Fe*+***	***Fe*-***	***ke+***	***ke-***	***q+***	***q-***	$q_{average}$
men		*[mm]*	*[mm]*	*[kN]*	*[kN]*	*[kN]*	*[kN]*	*[kN/*	*mm]*			
A1	*CS optimized*	7,3	8,3	36,0	24,8	64,1	47,9	4,97	2,99	1,78	1,93	**1,86**
A2	*CS opt +perf.*	8,0	4,2	37,5	21,3	64,9	39,9	4,69	5,02	1,73	1,87	**1,80**
A3	*CS reinforced*	19,1	16,7	61,4	67,3	98,1	98,1	3,22	4,04	1,60	1,46	**1,53**
B1	*Clay optimized*	10,5	8,9	37,7	23,9	61,3	40,0	3,58	2,68	1,63	1,67	**1,65**
B2	*Clay infill blocks*	5,5	5,7	33,3	26,1	46,3	34,3	6,00	4,56	1,39	1,31	**1,35**
B3	*Clay infill reinf.*	15,8	16,5	57,2	64,1	100,2	84,0	3,63	3,89	1,75	1,31	**1,53**
B4	*LAC*	9,1	14,8	42,9	26,3	83,2	49,0	4,74	1,78	1,94	1,87	**1,90**

In table 1, u_e means the elastic limit deformation, F_y the yield force, and F_e the force from elastic solution. The values are taken from deliverable 7.2c [6]. k_e is the elastic stiffness. The symbols + and – denote the actual values for positive and negative direction. $q_{average}$ is the mean value of q+ and q-. These values correspond to q_d as calculated in deliverable 7.2c.

For a structural designer, one of the main governing parameters is the assumption of the initial stiffness in the analysis. The initial stiffness determines the fundamental period, and thus, the spectral value due to the response spectrum.

The elastic stiffness as obtained from the evaluation of the Interstorey drift angle, however, in general is not identical with the initial stiffness of the specimen. Assuming, a good structural analysis would be able to predict the correct value of the initial stiffness, another behaviour factor q would be necessary to match the required results since the spectral value depends on the linear elastic response spectrum

The linear elastic response spectrum according to EC 8 as well as the design spectrum, which contains the reduction by the q-factor, consist of an ascending branch for small periods $T < T_B$, a plateau for $T_B < T < T_C$ and a descending branch for $T > T_C$. This descending branch descends inversely proportional to T (see e.g. Fig. 22 in deliverable 7.2c)

The value of T_C assumed for the response spectrum amounts to T_C = 0,4 s corresponding to f = 2,5 Hz. Using the measured fundamental frequencies f_0 from deliverable 7.2c, the pertinent periods T_0, the values of the design accelerations can be calculated in comparison to the accelerations for the periods T_e calculated using the equivalent stiffness k_e. As far as both periods lie within the limits of the plateau $T_B < T < T_C$, there is no difference in the calculated linear response accelerations. Otherwise, the response acceleration obtained for a larger value $a(T_0) > a(T_e)$ would form the basis for a structural analysis. Considering this, the q-factors evaluated form the experimental data might be modified to q´ accordingly as shown in Table 2. Modifications may become necessary for the tests A1, A2, and B4.

Table 2: Modified behaviour factors q´ for design purposes

Spec.	T_0/T_e+	T_0/T_e-	T_{0exp}	T_e+	T_e-	a_e+/a_0	a_e-/a_0	q+´	q-´	q´$_{av}$
A1	0,56	0,43	0,27	0,49	0,63	0,82	0,64	2,16	3,02	**2,59**
A2	0,59	0,61	0,26	0,44	0,43	0,90	0,93	1,92	2,01	**1,96**
A3	0,90	1,00	0,20	0,22	0,20	1,00	1,00	1,60	1,46	**1,53**
B1	0,67	0,58	0,24	0,36	0,41	1,00	0,97	1,63	1,72	**1,67**
B2	0,67	0,76	0,23	0,34	0,30	1,00	1,00	1,39	1,31	**1,35**
B3	0,95	0,99	0,24	0,25	0,24	1,00	1,00	1,75	1,31	**1,53**
B4	0,60	0,37	0,22	0,37	0,61	1,00	0,66	1,94	2,84	**2,39**

For specimen A3 and B3 the results show rather low behaviour factors q´ although these specimens have used vertical reinforcement at the edges. In fact, this may appear as negative and surprising, but this result is a consequence of the fact that the "yield"-force level on the resistance side is much higher than for the unreinforced specimens. Hence, the Interstorey drift angle demand was lower which may explain the low q-factor.

3.4. Evaluation of Pseudodynamic Full Scale Tests at JRC Ispra

For the tests at JRC Ispra, behaviour factors q can be estimated on the basis of the nonlinear relation between input acceleration and total lateral force response. For both the buildings with Calcium Silicate units (specimen K) and with Clay units (specimen M), Fig. 10 and Fig. 11 depict these relationships.

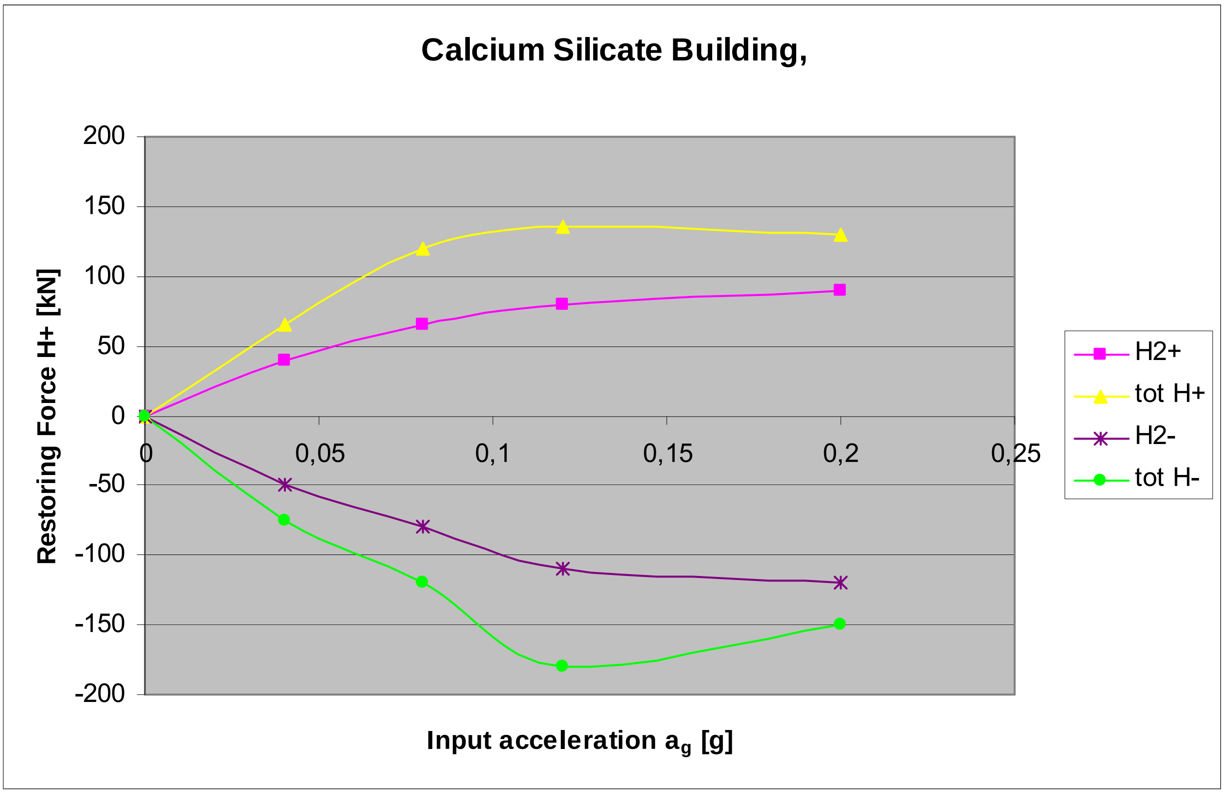


Fig. 10: Force vs. input acceleration relationship for specimen K (Calcium Silicate)

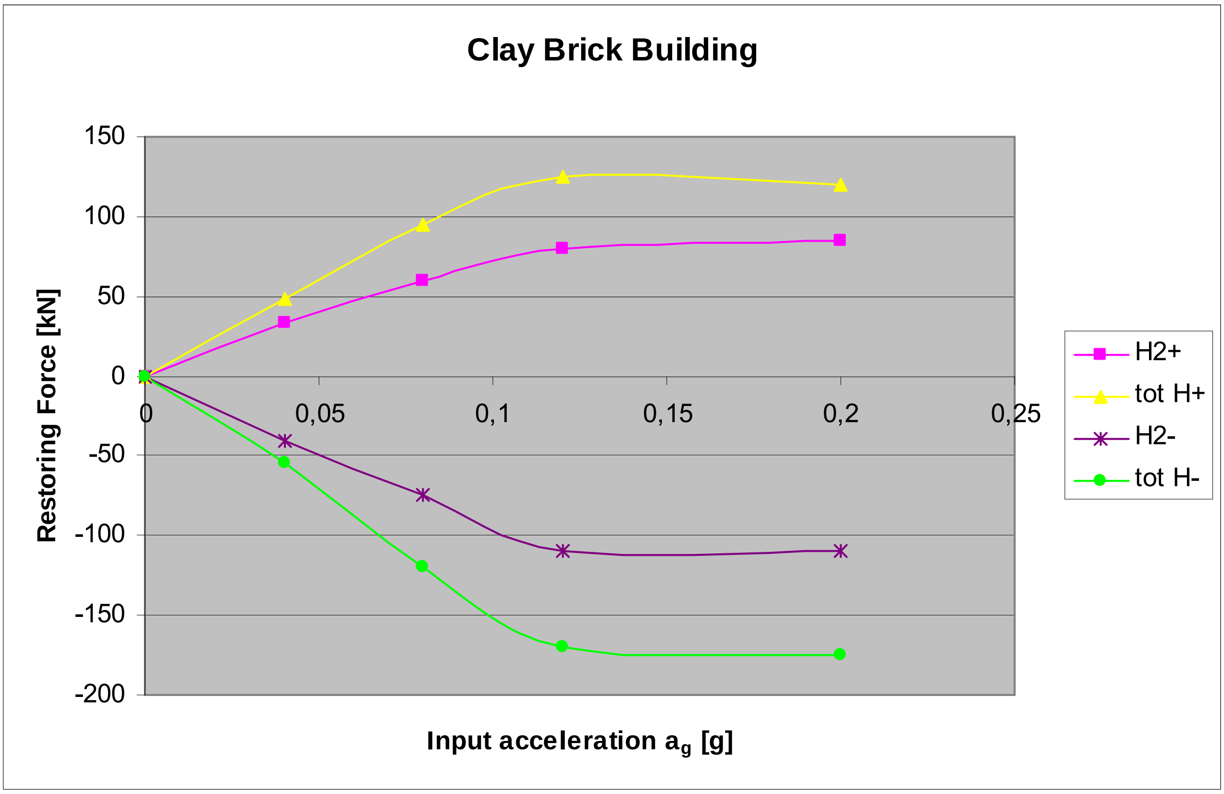


Fig. 11: Force vs. input acceleration relationship for specimen M (Clay Bricks)

Until about a_g = 0.08 g, the figures for both buildings show almost linear behaviour. Using the total restoring Force H_{tot} at a_g = 0.08 g for linear extrapolation until 0.2 g, the following data are obtained:

Table 3: Estimation of behaviour factors for the full scale tests at JRC Ispra

Specimen	H_{tot}+(0,08g)	H_{tot}-(0,08g)	H_{tot}+(0,2g)	H_{tot}-(0,2g)	H_{el}+ (0,2g)	H_{el}- (0,2g)	q+	q-
K	120	-120	130	-150	300,0	-300,0	2,3	
M	95	-120	120	-175	237,5	-300,0	2,0	
Specimen	H_{tot}+(0,08g)	H_{tot}-(0,08g)	H_{tot}+(0,2g)	H_{tot}-(0,2g)	H_{el}+ (0,2g)	H_{el}- (0,2g)	q+	q-
K	120	-120	130	-150	300,0	-300,0	2,3	
M	95	-120	120	-175	237,5	-300,0	2,0	

Specimen	H_{tot}+(0,08g)	H_{tot}-(0,08g)	H_{tot}+(0,2g)	H_{tot}-(0,2g)	H_{el}+ (0,2g)	H_{el}- (0,2g)
K (CS)	120 kN	-120 kN	130 kN	-150 kN	300 kN	-300 kN
M (Clay)	95 kN	-120 kN	120 kN	-175 kN	238 kN	-300 kN

It should be noted that the maximum force as used for this evaluation has been taken from the results for the excitation level of 0.2 g although the calcium silicate specimen (K) has reached an even higher value (about 180 kN) for the excitation of 0.12 g. This decrease of forces may be explained by a certain damage that seemingly has occurred during the load increase of the input. Therefore, the value of q = 2,2 for this specimen should be regarded as an upper limit value. When using the 180 kN instead of 150 kN for H_{tot}^{-}(-0,2g), a value of q = 2 would be obtained.

The results show slightly different behaviour factors for the positive and negative direction. This could be expected since the main shear walls act as unsymmetrical T – shaped cross sections. However, the overall level of q factors is above the actual value of q = 1.5 for unreinforced masonry in EN 1998-1.

Moreover, the capacity of the overall system is significantly higher than could be expected from design checks as they are usually performed. Even the resistance of those walls in the static cyclic and pseudodynamic wall tests which have identical geometry and similar vertical loading is much less than what has been observed in the tests on integral buildings. The comparative wall tests for the clay brick specimen in Pavia (static cyclic) and Kassel (pseudodynamic) showed in both cases a maximum horizontal force of about 50 kN for the wall with a length of 1.50 m and a maximum horizontal force of about 30 kN for the 1.00 m long exterior walls. So in sum a horizontal force of 50 kN + 2 * 30 kN = 110 kN could have been expected for the full scale specimen. In comparison to this, a remarkable increase of up to a total Force H_{tot} = 175 kN in the full scale tests could be observed.

This effect may be explained when the redistribution of normal forces between different walls in one storey is considered. For the ground plan layout of the terraced houses tested at JRC Ispra, a pronounced potential of redistribution of normal forces between lateral and transverse walls exists when uplift of transverse walls occurs.

It becomes clear, that the behaviour factor alone cannot explain increased seismic resistance as observed in the experiments. Quite in the contrary, it would be strongly misleading to attribute the increased earthquake resistance to the effect of ductile behaviour alone since this could not explain the high level of lateral resistance of the Ispra tests, expressed as horizontal force H_{tot}. In fact, the increase of normal force in the main shear walls seems to be at least as important as explanation for the very favourable behaviour.

4. Proposal for selection of behaviour factor

Figures 1 to 3 show a large variation of the ductility μ as well as of the interstorey drift capacity θ_u. However, for low values of the vertical strength exploitation α, and low values of the aspect ratio l_w/h_w there is a tendency to increased deformation capacity. A conservative lower bound of the ductility may, hence, be established. A regulation which is easy to handle can be proposed as follows:

For unreinforced masonry the behaviour factor may be assumed as q = 1.5 for values of the wall height to length ratio

$$h_w / l_w \leq 1.0$$

However, for a wall height to length ratio

$$H_w / l_w \geq 1.6$$

q = 2.0 may be used, under the condition that the stress exploitation is limited to

$$\alpha = \sigma_k / f_k \leq 0.15$$

Linear interpolation for intermediate h_w/l_w values is allowed.

The proposal reflects the observed more ductile behaviour under low values of vertical load and for slender walls.

An alternative approach depending on the dimensionless normal force n (n = α) is being presented in deliverables 9.3 and 9.4 [11]. A combination of the two proposals may be discussed.

It can be noted, that the evaluation of all test results did not show significant differences for the q-factors for different types of masonry. Thus, the proposals presented herein for the evaluation of the q-factor may be applied regardless of the type of masonry.

5. Conclusions

The experimental data of the ESECMaSE research comprises about 100 tests on masonry walls and entire structures. Although the determination of behaviour factors is made difficult by the large number of influencing parameters, some tendencies could be found. For a conservative lower bound, simple proposals have been made. They can be applied for all types of masonry as investigated within ESECMaSE.

The evaluation of the pseudodynamic tests on full scale structures at JRC Ispra as well as of the shaking table tests have enabled to determine behaviour factors of integral structures under seismic loading. It could be shown, that in addition to the overall ductility of a structure and its deformation capacity, other effects are at least as important as the value of the behaviour factor. As an example for this thesis, the increase of normal forces due to dynamic effects and due to redistribution of the vertical load flow is of importance.

Further research is necessary in order to obtain a better understanding of such effects. With respect to the ductility and the deformation capacity, a combination of experimental work and refined numerical modelling is required in order to enable a better quantitative prediction of ductility and deformation capacity.

The influence of seismic forces perpendicular to the wall plane has not been studied in detail within ESECMaSE. However, for medium and higher degrees of seismic, future research should also address this important aspect.

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