In-situ Testing at building unit Zijlvest 25 in Loppersum

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(European Centre for Training and Research in Earthquake Engineering)

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General Introduction

To support the modelling of the seismic response of buildings, an extensive program of measurement of the properties of building materials typically used in the construction of the buildings in Groningen is executed. This program includes measurements on new and old building materials both in the laboratory and in existing buildings. The material properties have been summarized in the Material Characterisation Report (1).

This document describes the experiments carried out in the existing terraced house at Zijlvest 25 in Loppersum. This unreinforced masonry (URM) house was built in the 1970's. The testing procedures developed by Eucentre in Pavia have been attached as an appendix to this report.

The laboratory experiments are described in the report "Characterisation of Original Groningen Masonry" (Ref. 2). More information on the modelling of URM walls and buildings can be found in the "URM Modelling and Analysis Cross Validation Report" (Ref. 3).

References

1. Summary report for the characterisation of original Groningen masonry, TU Delft (S. Safari, J. Rots), 18th December 2015.
2. Material Characterisation – Version 1.3, Eucentre, P&P, TU-Delft, TU-Eindhoven, October 2015.
3. URM Modelling and Analysis Cross Validation – Arup, Eucentre, TU Delft, Reference 229746_032.0_REP127_Rev.0.03 April 2015.

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Results of in-situ tests

Building unit: Loppersum, Zijlvest 25

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1. Introduction

This document provides information on the results of in-situ tests performed in the terraced house located in Zijlvest 25, Loppersum, NL.
The tests were conducted by Eng. Filippo Dacarro, Eng. Simone Girello, Mr.Roberto Pistore, Dr. Simone Peloso and Dr. Francesco Graziotti. The document was written with the contributions of Eng. Andrea Rossi, Dr. Ilaria Senaldi and Dr. Simone Peloso.
The locations of the tests conducted on the masonry walls of the house are shown in the following figures. The complete description of the methods and procedures of the various tests are described in the attached documents: Annex_1_procedure_flatjacks.pdf, Annex_2_Report PNT_G tests.pdf, Annex_3_6239_Boviar_782.pdf (40, 41).

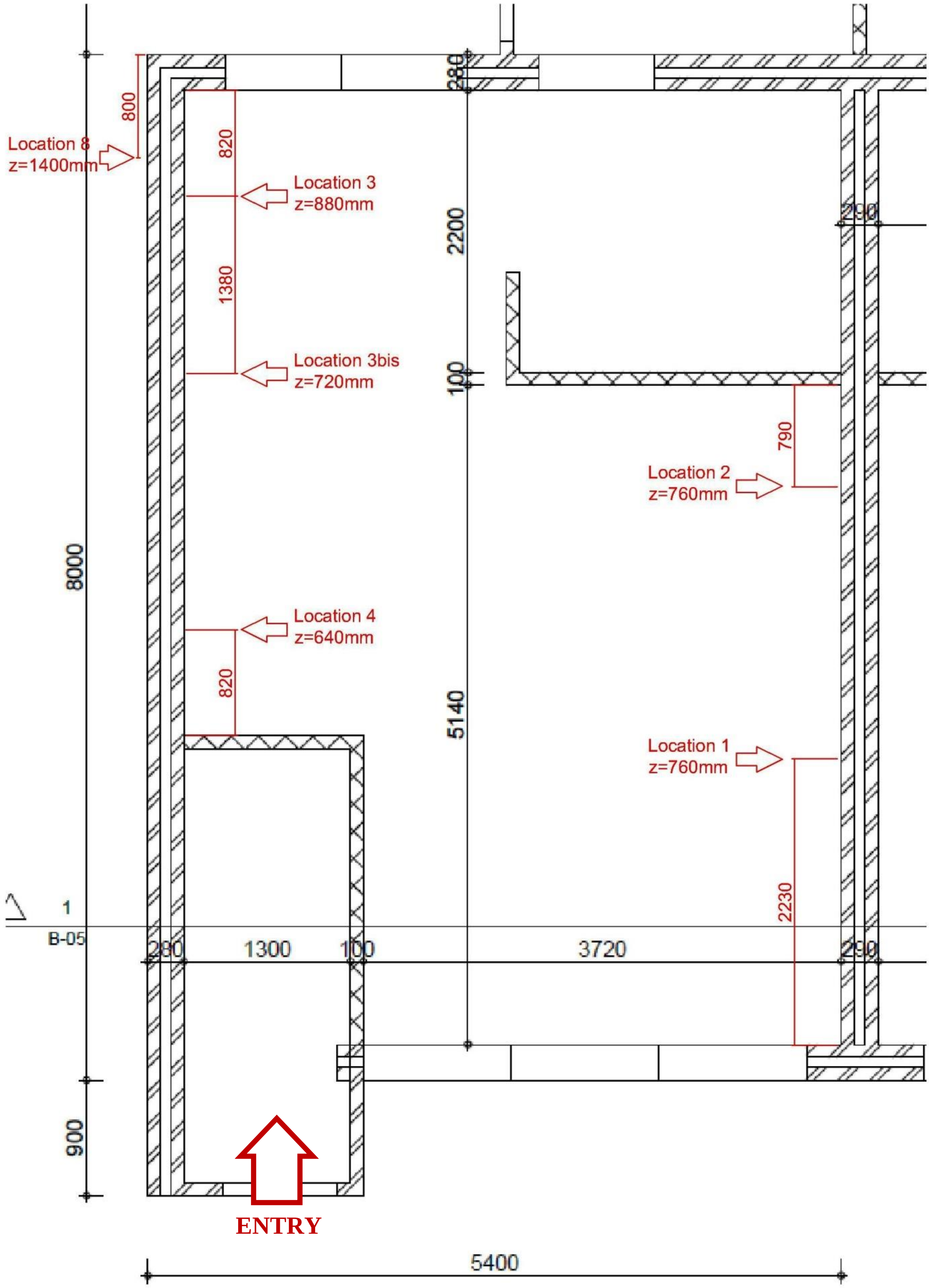


Figure 1 - Plane view of the ground floor and position of the centre of each test (measures in mm)

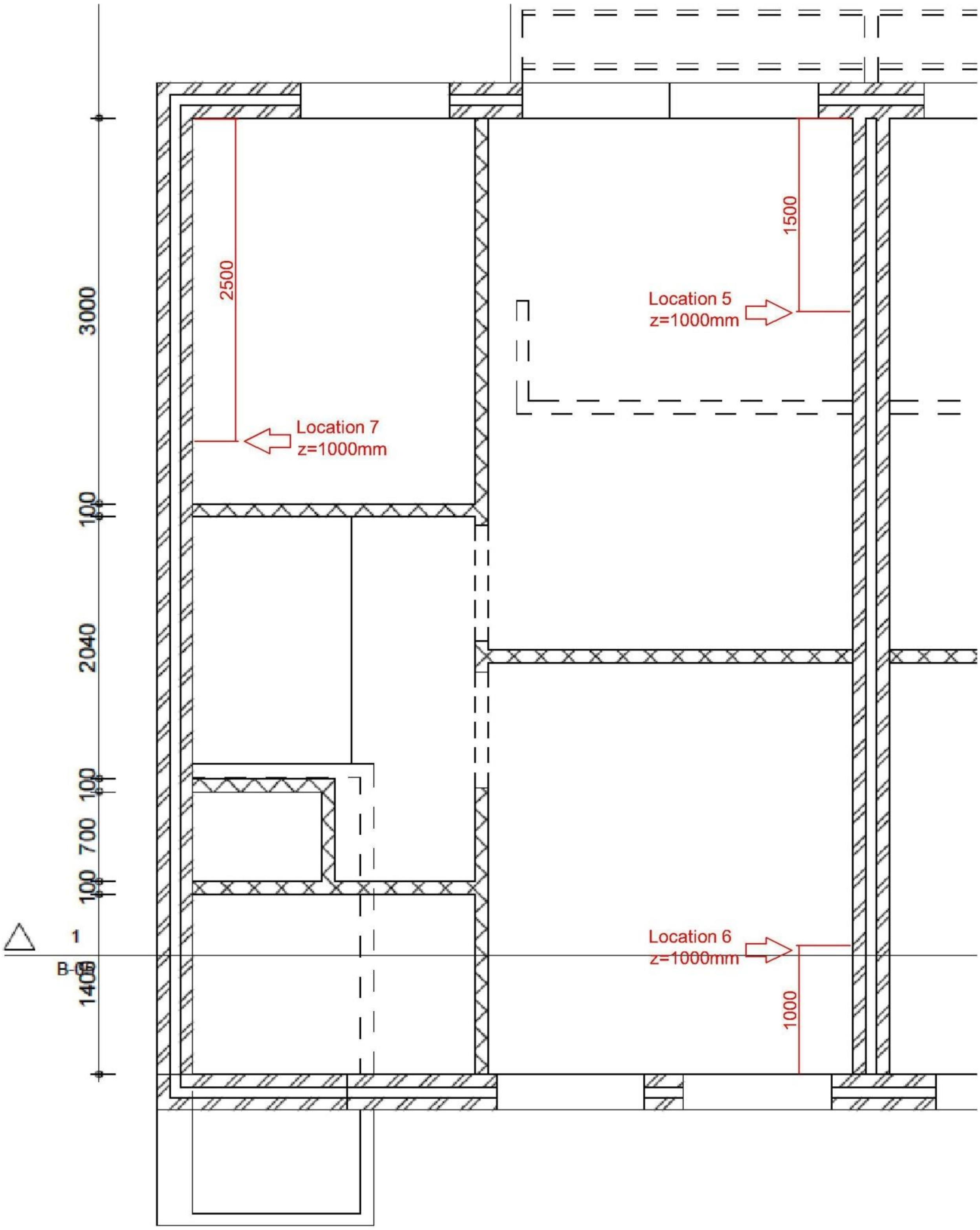


Figure 2 - Plane view of the first and position of the centre of each test (measures in mm)

2. Single Flat jack test Results

The purpose of the Single Flatjack test is to evaluate, in situ, the average compressive stress in a specific zone of an unreinforced masonry wall.
In order to measure the stress acting in the masonry wall, a flatjack inserted in a slot cut between two courses of blocks was used. When a slot is cut in a masonry wall, the release of the compressive stresses/strains near the cut causes a reduction of the distance between the edges of the cut. The stress existing prior to the cut is estimated by increasing the pressure in the flatjack until the original distance between the points above and below the cut is restored. The stress state in the masonry is related to the flatjack pressure (multiplied by coefficients that take into account the characteristics of the jack, the surface of the jack in contact with the masonry and the area of the opening).

The methodology and the procedure of the test are explained in Annex 1.

The distance between four gage points were measured. The distance between the gage points was measured both manually and digitally (by mean of LVDTs), in order to have two different and independent measures.

A rectangular flatjack was used (dimensions: 350x95x4.1 mm, nominal area of 33250 mm^2).
For further and more precise information, see the attached calibration certificates (Annex 3).

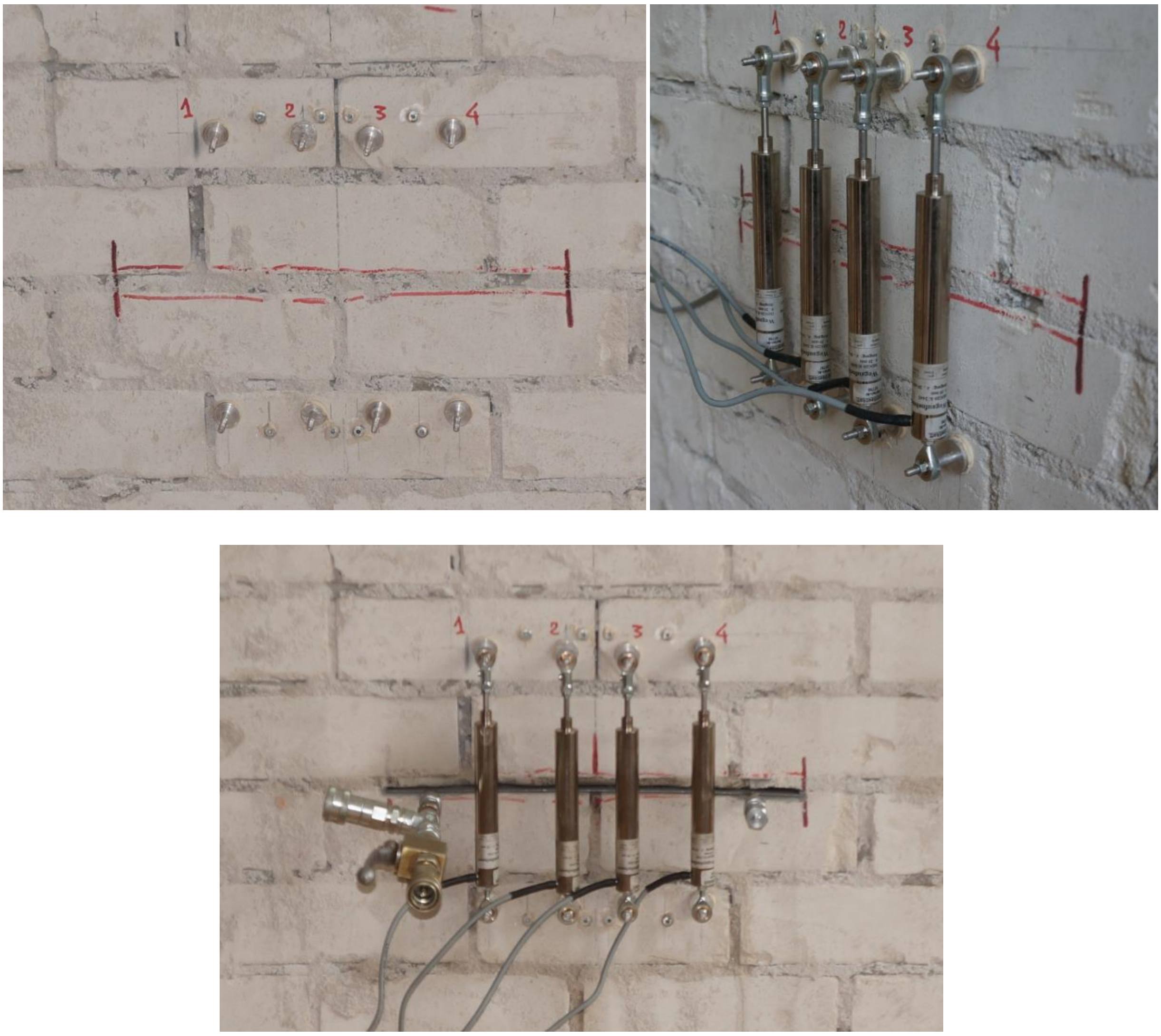


Figure 3 - Different views of the Setup for the Single Flatjack Test

The average compressive stress in the masonry, σ_j, in contact with the flatjacks is given as before by:

$$\sigma_j\ [kPa] = K_a\left(a[-]\cdot p[bar]\cdot 100[kPa/bar] + \frac{b}{A_{flj}}[kPa]\right) \qquad (1)$$

A_{flj} area of the flatjack (given by the producer) = 33250 mm^2= 0.03325 m^2;

K_a A_{flj} /area of the slot, $K_a \approx 1$ in this case;

p flatjack pressure required to restore the gage points to the distance initially measured between them within the allowed tolerance (bar).

In this case (data provided by the flatjack producer):

a = 0.778 (-)
b/A = 3.810 kPa

a,*b* taken as the mean of the values provided by the three calibration certificates (Annex 3).

Single Flat jack test Results, location 1

Manual measurements

In the following Table, the results of the test are reported.
The recorded measures are:

- The vertical distance between the gage points before the sawcut, this distance is the original/initial one that has to be restored;
- the relative distance of the gage points, after the sawcut (the flatjack is not present yet);
- the relative distance of the gage points, with the flatjack inserted and with an internal pressure of 5.5 bar, corresponding to the restoring of the original configuration: relative distances equal to the initial ones (before the sawcut).

Table 1 - Single Flatjack Test Results, manual measurements - location 1

Undamaged masonry					Base (μm)
gage points	distance measurements (μm)				5529
	1st	2nd	3rd	mean	
1	4827	4845	4855	**4842**	
2	5007	5012	5018	**5012**	
3	5576	5574	5569	**5573**	
4	5326	5327	5323	**5325**	

Flatjack pressure: 5.5 bar (1)					Base (μm)
gage points	distance measurements (μm)				5514
	1st	2nd	3rd	mean	Δ [μm]
1	4837	4840	4836	**4838**	-5
2	-	-	-		
3	-	-	-		
4	5313	5313	5317	**5314**	-11

After sawcut					Base (μm)
gage points	distance measurements (μm)				5529
	1st	2nd	3rd	mean	Δ [μm]
1	4816	4841	4845	**4834**	-8
2	4995	4996	4992	**4994**	-18
3	5467	5455	5519	**5480**	-93
4	5328	5313	5302	**5314**	-11

Flatjack pressure: 5.5 bar (2)					Base (μm)
gage points	distance measurements (μm)				5515
	1st	2nd	3rd	mean	Δ [μm]
1	4846	4842	4862	**4850**	8
2	5007	5007	5005	**5006**	-6
3	5565	5559	5563	**5562**	-11
4	5319	5321	5321	**5320**	-5

In the following Figure the results of the test are shown.

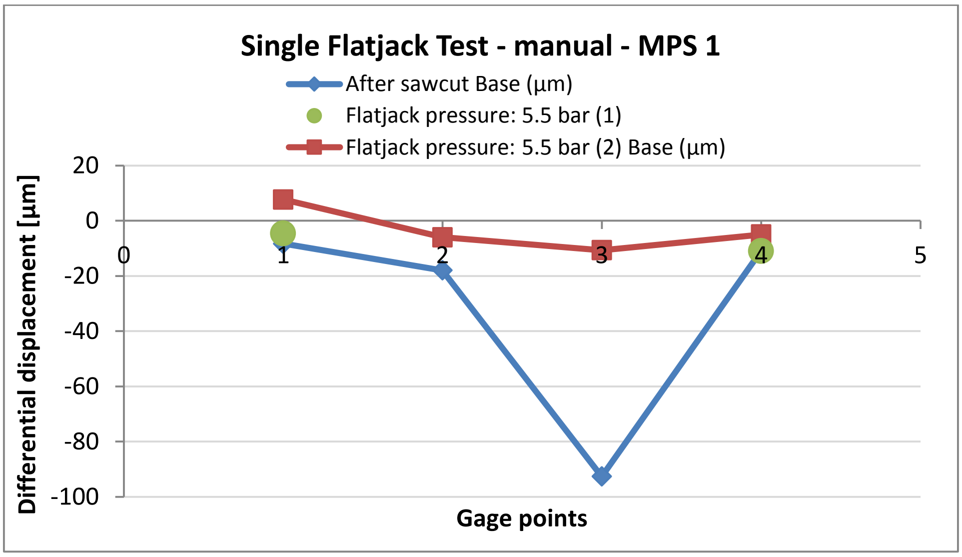


Figure 4 - Graph of the Single Flatjack Test Results, manual measurements - location 1

In the location 1, the restoring pressure in the masonry is equal to 0.432 MPa (manual measure).

Digital measurements (LVDT)

If the record of the gage point distance is taken digitally, using LVDT devices, the determination of the restoring pressure is easier. This because both distance and pressure measures are taken in real time and simultaneusly.
Four LVDT are used to measure the relative distance of gage points in the verical direction, and the differential displacement: $\Delta_i = d_i - d_{initial}$ per each LVDT (i=time step index).
The mean value of Δ_i was calculated. This value is equal to zero at the beginning (before the sawcut) and after the restoring of the initial condition, corresponding to a certain value of the pressure in the flatjack.
The restoring pressure in the flatjack corresponds to the situation in which the LVDT mean record of the differential displacements (Δ_i) is again equal to zero.
In the following Figure the records of the test and the identificaiton of the restoring situation are clearly shown.

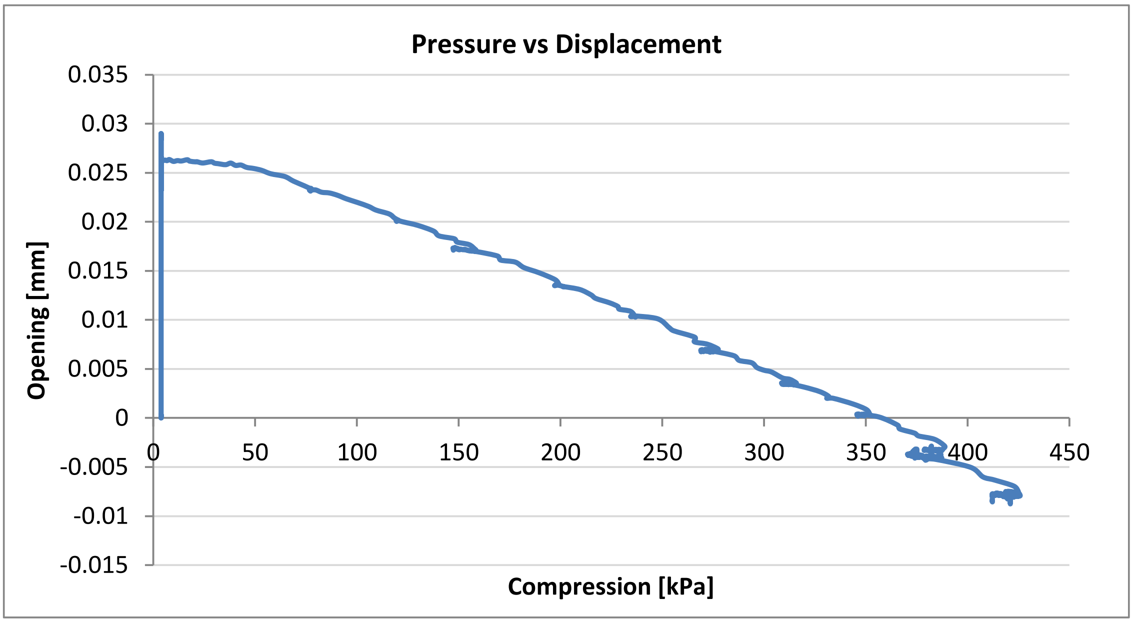


Figure 5 - LVDT's displacement vs. flatjack corrected pressure - location 1

In the **location 1**, the restoring pressure in the masonry is equal to **0.366 MPa** (digital measurements).

Single Flat jack test Results, location 2

Manual measurements

In the following Table, the results of the test are reported.

The recorded measures are:

- The vertical distance between the gage points before the sawcut, this distance is the original/initial one that has to be restored;
- the relative distance of the gage points, after the sawcut (the flatjack is not present yet);
- the relative distance of the gage points, with the flatjack inserted and with an internal pressure of 2.7 bar, corresponding to the restoring of the original configuration: relative distances equal to the initial ones (before the sawcut).

Table 2 - Single Flatjack Test Results, manual measurements - location 2

Undamaged masonry					Base (μm)
gage points	distance measurements (μm)				5521
	1st	2nd	3rd	mean	
1	6097	6097	6099	**6098**	
2	4467	4438	4450	**4452**	
3	8529	8529	8528	**8529**	
4	7647	7630	7638	**7638**	
After sawcut					**Base (μm)**
gage points	distance measurements (μm)				5523
	1st	2nd	3rd	mean	Δ [μm]
1	6066	6071	6071	**6069**	-28
2	4409	4414	4424	**4416**	-36
3	8491	8493	8503	**8496**	-33
4	7596	7572	7581	**7583**	-55
Flatjack pressure: 2.7 bar					**Base (μm)**
gage points	distance measurements (μm)				5519
	1st	2nd	3rd	mean	Δ [μm]
1	6092	6092	6094	**6093**	-5
2	4442	4447	4446	**4445**	-7
3	8535	8528	8528	**8530**	2
4	7611	7653	7655	**7640**	1

In the following Figure the results of the test are graphically shown.

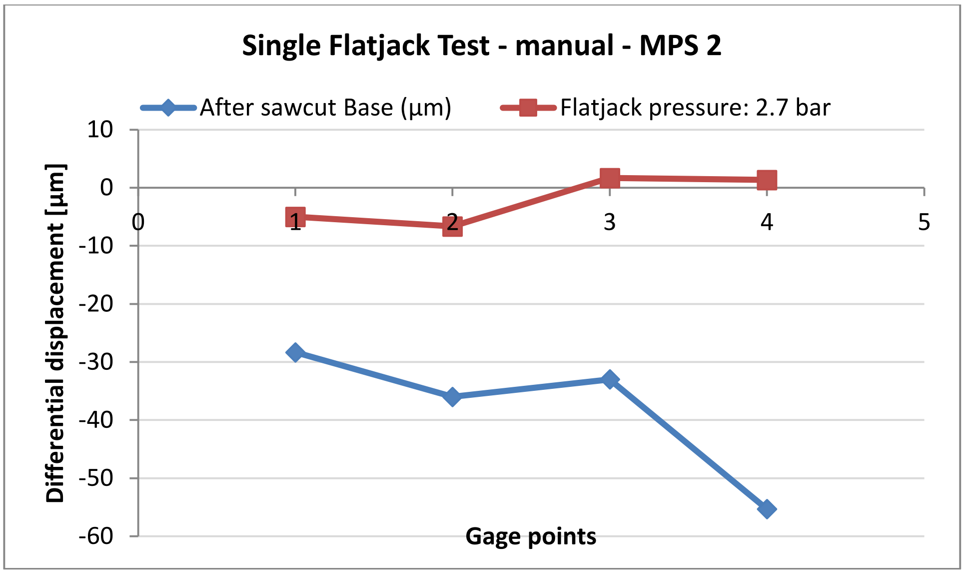


Figure 6 - Graph of the Single Flatjack Test Results, manual measurements - location 2

In the location 2, the restoring pressure in the masonry is equal to 0.214 MPa (manual measurements).

<u>Digital measurements (LVDT)</u>

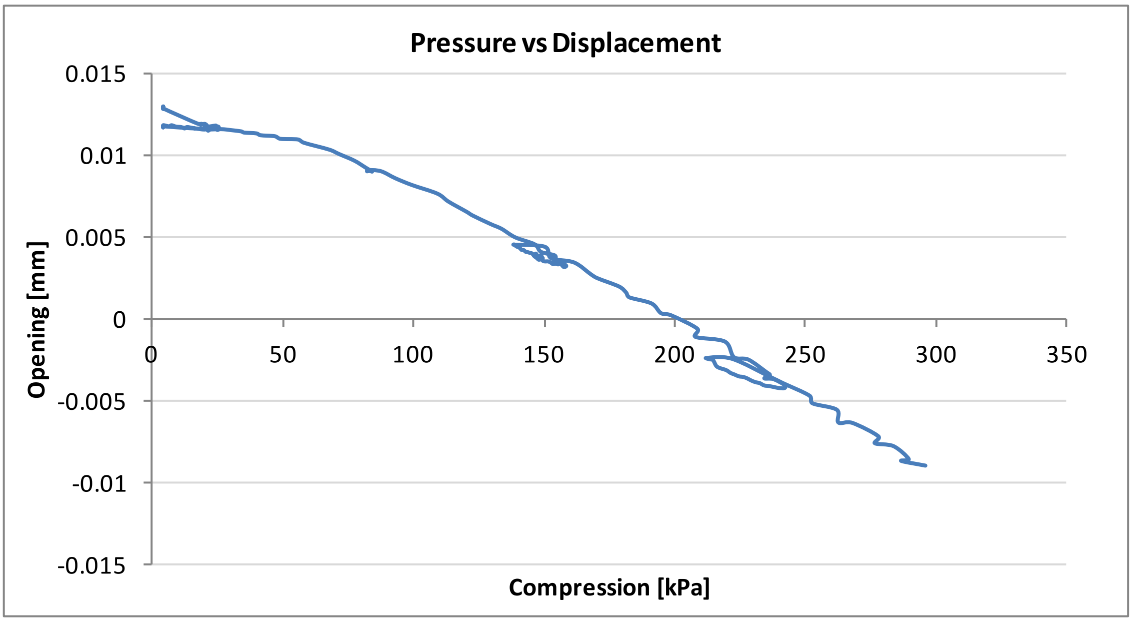


Figure 7 - LVDT's displacement vs. flatjack corrected pressure - location 2

In the **location 2**, the restoring pressure in the masonry is equal to **0.208 MPa** (digital measurements).

Single Flat jack test Results, location 3

Manual measurements

In the following Table, the results of the test are reported.

The recorded measures are:

- The vertical distance between the gage points before the sawcut, this distance is the original/initial one that has to be restored;
- the relative distance of the gage points, after the sawcut (the flatjack is not present yet);
- the relative distance of the gage points, with the flatjack inserted and with an internal pressure of 1.45 bar, corresponding to the restoring of the original configuration: relative distances equal to the initial ones (before the sawcut).

Table 3 - Single Flatjack Test Results, manual measurements - location 3

Undamaged masonry					Base (μm)
gage points	distance measurements (μm)				5523
	1st	2nd	3rd	mean	
1	5624	5617	5624	5622	
2	3945	3943	3948	3945	
3	6599	6610	6607	6605	
4	5889	5896	5892	5892	
After sawcut					**Base (μm)**
gage points	distance measurements (μm)				5524
	1st	2nd	3rd	mean	Δ [μm]
1	5605	5604	5612	5607	-15
2	3920	3926	3919	3922	-24
3	6582	6591	6612	6595	-10
4	5876	5881	5878	5878	-14
Flatjack pressure: 1.45 bar					**Base (μm)**
gage points	distance measurements (μm)				5521
	1st	2nd	3rd	mean	Δ [μm]
1	5624	5620	5622	5622	0
2	3940	3942	3937	3940	-6
3	6620	6620	6619	6620	14
4	5890	5892	5885	5889	-3

In the following Figure the results of the test are shown.

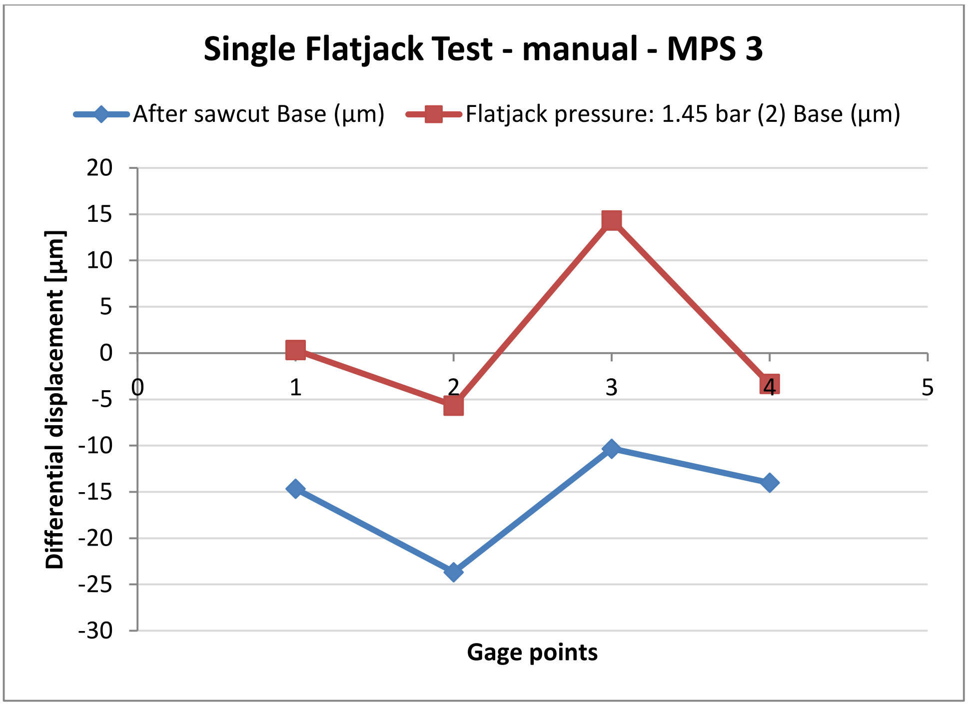


Figure 8 - Graph of the Single Flatjack Test Results, manual measurements - location 3

In the location 3, the restoring pressure in the masonry is equal to 0.117 MPa (manual measurements).

Digital measurements (LVDT)

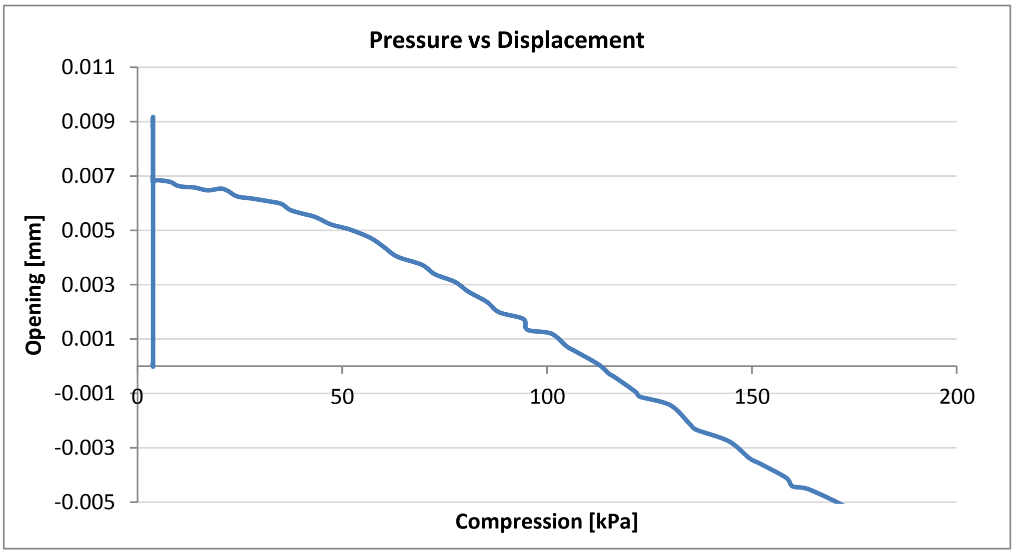


Figure 9 - LVDT's displacement vs. flatjack corrected pressure - location 3

In the **location 3**, the restoring pressure in the masonry is equal to **0.115 MPa** (digital measurements).

Single Flat jack test Results, location 4

Manual measurements

In the following Table, the results of the test are reported.

The recorded measures are:

- The vertical distance between the gage points before the sawcut, this distance is the original/initial one that has to be restored;
- the relative distance of the gage points, after the sawcut (the flatjack is not present yet);
- the relative distance of the gage points, with the flatjack inserted and with an internal pressure of 4.5 bar, corresponding to the restoring of the original configuration: relative distances equal to the initial ones (before the sawcut).

Table 4 - Single Flatjack Test Results, manual measurements - location 4

Undamaged masonry					Base (μm)
gage points	distance measurements (μm)				5521
	1st	2nd	3rd	mean	
1	4867	4867	4868	**4867**	
2	4737	4744	4738	**4740**	
3	5091	5092	5099	**5094**	
4	5104	5105	5104	**5104**	

After sawcut					Base (μm)
gage points	distance measurements (μm)				5521
	1st	2nd	3rd	mean	Δ [μm]
1	4840	4841	4845	**4842**	-25
2	4704	4708	4704	**4705**	-34
3	5067	5062	5063	**5064**	-30
4	5065	5071	5064	**5067**	-38

Flatjack pressure: 4.5 bar					Base (μm)
gage points	distance measurements (μm)				5520
	1st	2nd	3rd	mean	Δ [μm]
1	4867	4874	4867	**4869**	2
2	4742	4739	4737	**4739**	0
3	5145	5127	5137	**5136**	42
4	5106	5109	5107	**5107**	3

In the following Figure the results of the test are shown.

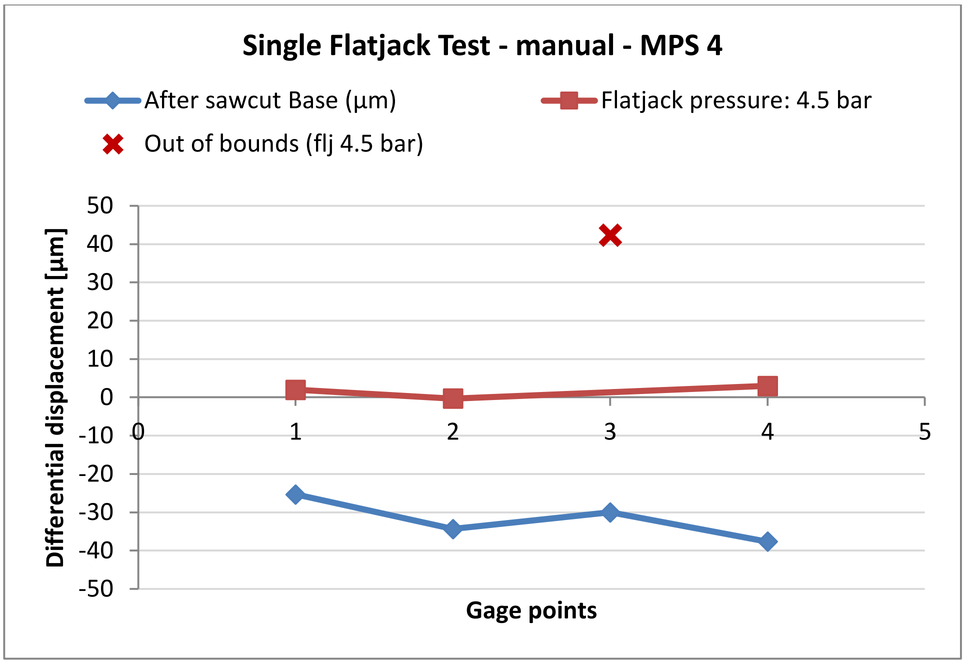


Figure 10 - Graph of the Single Flatjack Test Results, manual measurements - location 4

In the location 4, the restoring pressure in the masonry is equal to 0.354 MPa (manual measurements).

Digital measurements (LVDT)

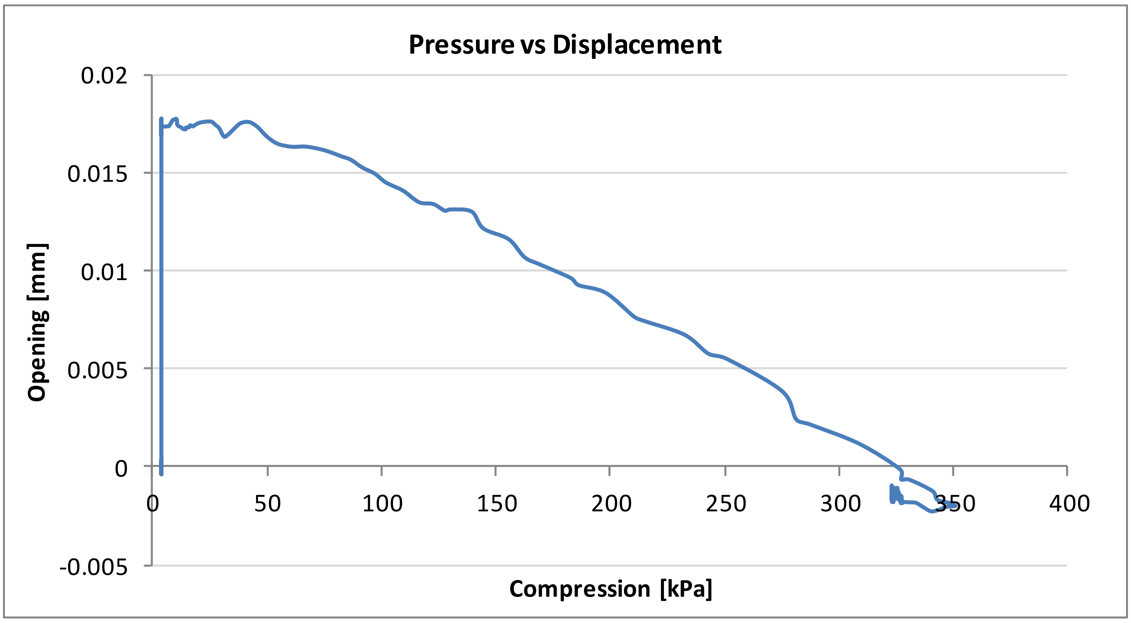


Figure 11 - LVDT's displacement vs. flatjack corrected pressure - location 4

In the **location 4**, the restoring pressure in the masonry is equal to **0.327 MPa** (digital measurements).

3. Double Flat jack test Results

The purpose of the in situ test is to evaluate the stiffness properties of a portion of an unreinforced masonry wall .
In order to measure the deformation characteristics, it is necessary to use two flatjacks in two different sawcuts. The pressure in the flatjacks is gradually increased inducing a compressive stress state in the portion of the wall between the two devices. By gradually increasing the pressure in the flatjacks and recording the deformation of the masonry between the slots it is possible to obtain the characteristics of interest in the form of a load/deformation relationship (stress/strain rel.).

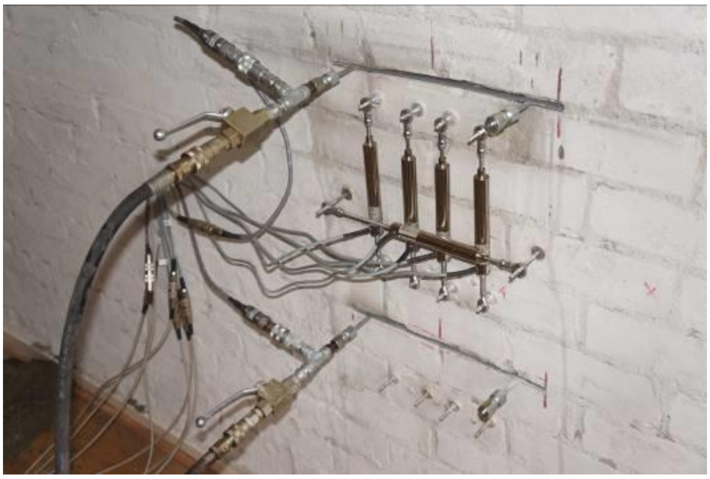


Figure 12 - Double Flatjack Test Setup

In Annex 1 it is possible to find a complete explanation of the test methodology.
The two flatjacks are of the same type of the one used for the Single Flatjack test: rectangular flatjack, 350x95x4.1 mm, with a nominal area of 33250 mm^2. For further and more precise information see the Certificates attached to this document (Annex 3).
Five LVDTs were used: four of them recorded the vertical displacements, one of them recorded the horizontal displacement, as is possible to see in the previous Figure.

The average compressive stress in the masonry, σ_j, in contact with the flatjacks is given as before by:

$$\sigma_j = K_a\left(a \cdot p + \frac{b}{A_{flj}}\right)$$

The deformation ε of the part of masonry between the two flatjack, at the time step *i*, is given by:

$$\varepsilon_i = \frac{\Delta_i}{L}$$

where: Δ_i differential displacement: $\Delta_i = d_i - d_{initial}$ for each LVDT (*i*=time step index)
L = reference distance of the specific used LVDT.

Double Flat jack test Results, location 1

The following graph shows the mean vertical deformation vs. the stress in the masonry.
Ten cycles of loading and unloading have been done in this test. The pressure in the flatjacks is increased to a certain level of pressure and then reduced back to zero (up to 8 bar).

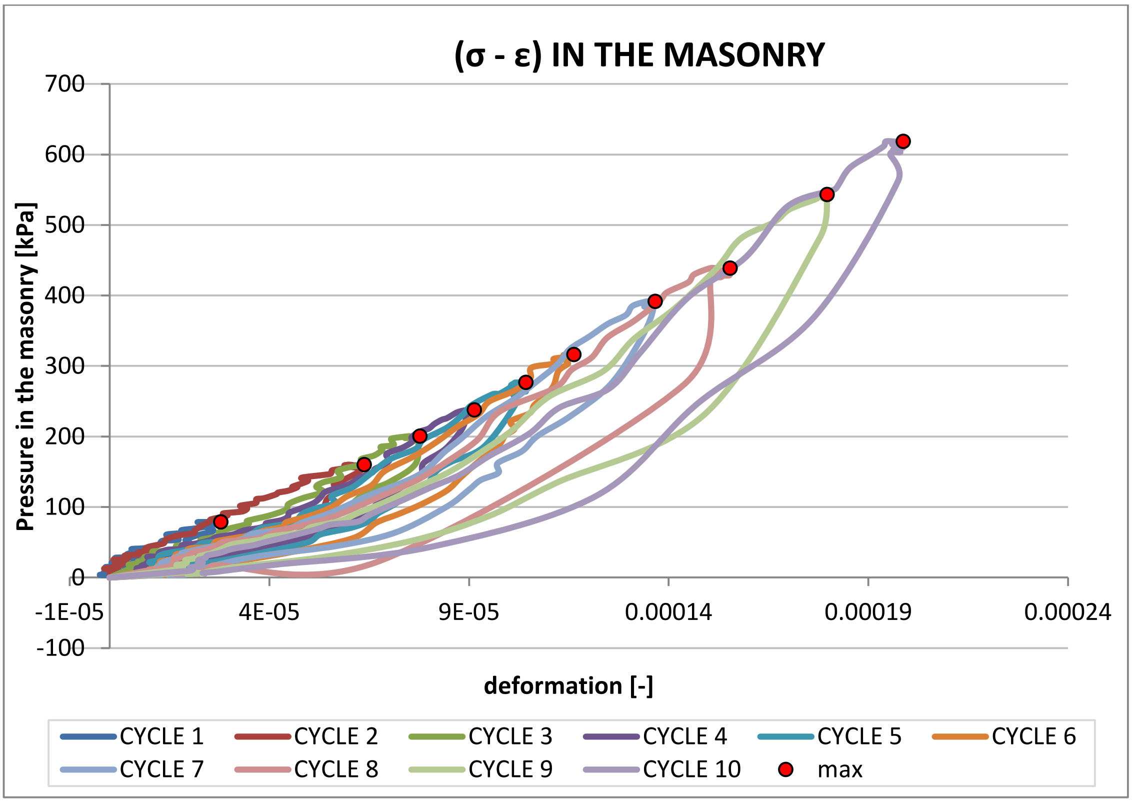


Figure 13 - Mean vertical deformation vs pressure in the masonry - location 1

The Elastic Young Modulus was conventionally determined using the following method:

E, the slope of the line passing through the point (ε_{max},σ_{max}) of the 3rd cycle and the point (ε_{max},σ_{max}) of the 8th cycle:

$$\boldsymbol{E} = \frac{\sigma_{max-cycle8} - \sigma_{max-cycle3}}{\varepsilon_{max-cycle8} - \varepsilon_{max-cycle3}} = \boldsymbol{3065\ MPa}$$

The Young Modulus is calculated in this way, in order to have a results more representative of all the cylces, except the first two, that give more scattered data.

Double Flat jack test Results, location 2

The following graph shows the mean vertical deformation vs. the stress in the masonry.
Eight cycles of loading and unloading have been done in this test. As before, the pressure in the flatjacks is increased to a certain level of pressure and then reduced back to zero.

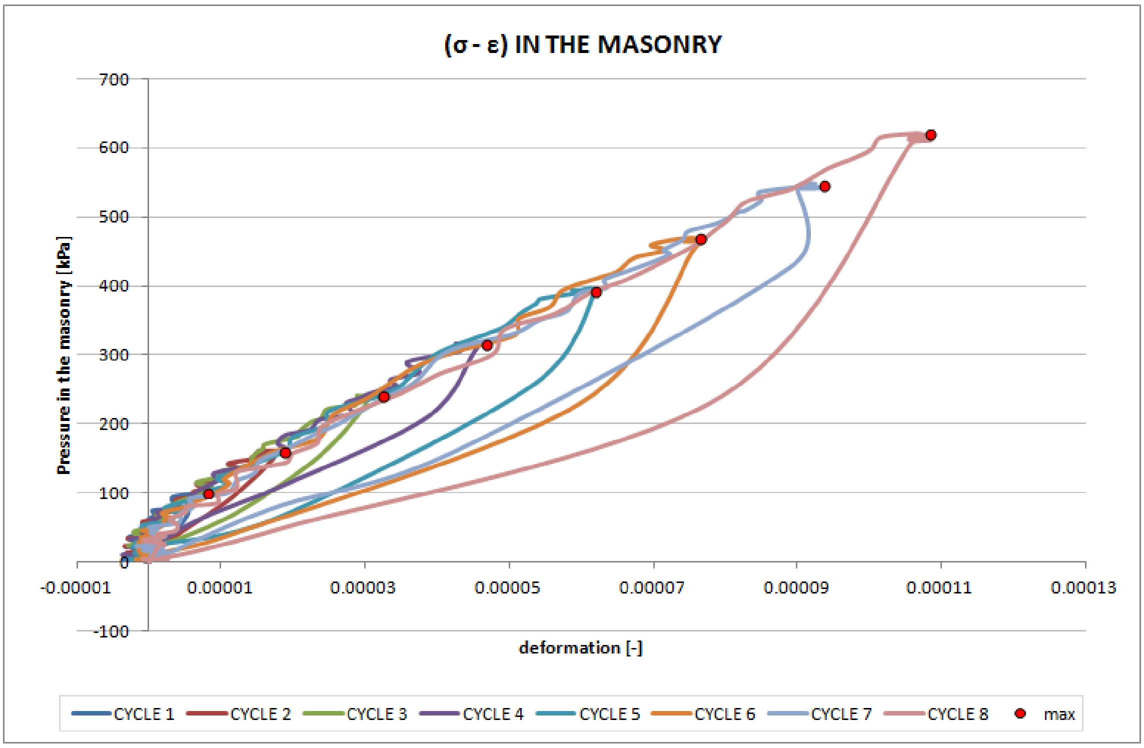


Figure 14 - Mean vertical deformation vs pressure in the masonry - location 2

The Elastic Young Modulus was determined using the following method:

E, the slope of the line passing through the point (ε_{max},σ_{max}) of the 3rd cycle and the point (ε_{max},σ_{max}) of the 8th cycle:

$$\boldsymbol{E} = \frac{\sigma_{max-cycle8} - \sigma_{max-cycle3}}{\varepsilon_{max-cycle8} - \varepsilon_{max-cycle3}} = \mathbf{4992}\ \boldsymbol{MPa}$$

The Young Modulus is calculated in this way, in order to have a results more representative of all the cylces, except the first two, that give more scattered data.

Double Flat jack test Results, location 3

The following graph shows the mean vertical deformation vs the stress in the masonry.
Eight cycles of loading and unloading have been done in this test. The pressure in the flatjacks is increased to a certain level of pressure and then reduced back to zero.

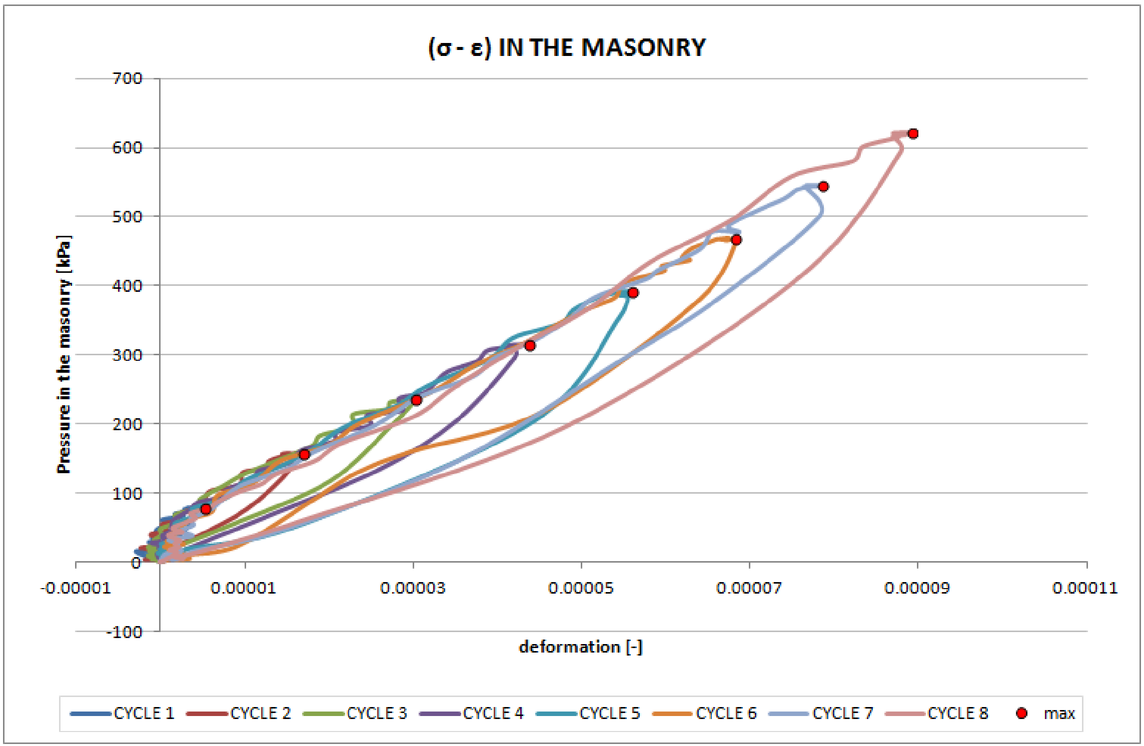


Figure 15 - Mean vertical deformation vs pressure in the masonry - location 3

The test was conducted using an external gasoline electrical power unit. This because an house circuit electrical interference was noticed in this position.
The Elastic Young Modulus was determined using the following method:

E, the slope of the line passing through the point (ε_{max},σ_{max}) of the 3rd cycle and the point (ε_{max},σ_{max}) of the 8th cycle:

$$\boldsymbol{E} = \frac{\sigma_{max-cycle8} - \sigma_{max-cycle3}}{\varepsilon_{max-cycle8} - \varepsilon_{max-cycle3}} = \mathbf{6517}\ \boldsymbol{MPa}$$

The Young Modulus is calculated in this way, in order to have a results more representative of all the cylces, except the first two, that give more scattered data.

Double Flat jack test Results, location 4

The following graph shows the mean vertical deformation vs. the stress in the masonry.
Eight cycles of loading and unloading have been done in this test. The pressure in the flatjacks is increased to a certain level of pressure and then reduced back to zero.

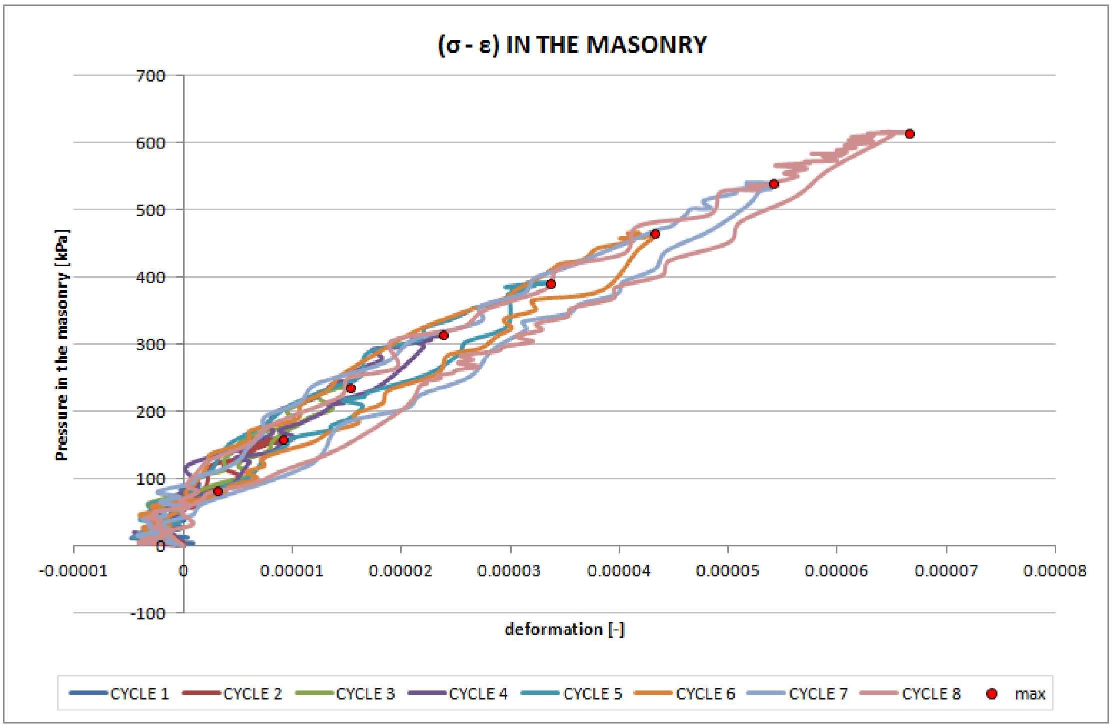


Figure 16 - Mean vertical deformation vs pressure in the masonry - location 4

The Elastic Young Modulus was determined using the following method:

E, the slope of the line passing through the point (ε_{max},σ_{max}) of the 3rd cycle and the point (ε_{max},σ_{max}) of the 8th cycle:

$$\boldsymbol{E} = \frac{\sigma_{max-cycle8} - \sigma_{max-cycle3}}{\varepsilon_{max-cycle8} - \varepsilon_{max-cycle3}} = \mathbf{7391}\ \boldsymbol{MPa}$$

The Young Modulus is calculated in this way, in order to have a results more representative of all the cylces, except the first two, that give more scattered data.

Double Flat jack test Results, location 3 bis

The following graph shows the mean vertical deformation vs the stress in the masonry.
Eight cycles of loading and unloading have been done in this test, the pressure in the flatjacks is increased to a certain level of pressure and then reduced back to zero.

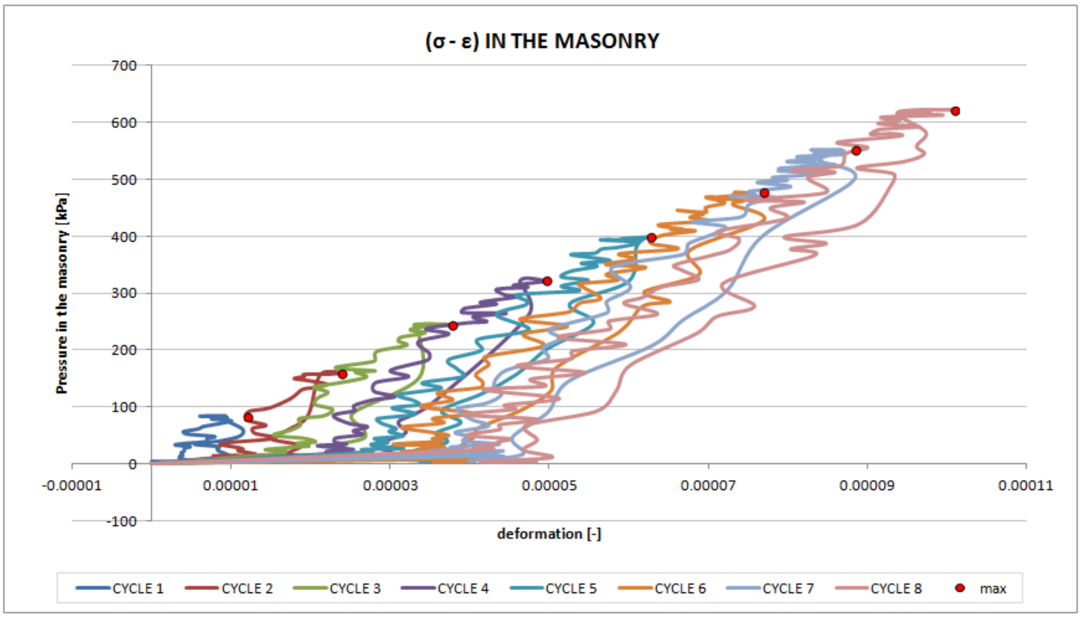


Figure 17 - Mean vertical deformation vs pressure in the masonry - location 3 bis

The signals recorded during this test resulted to have more noise compared to other position. This is likely due to some electrical field in the wall. Position 3 had the same problem, but more evident. For this reason in that position an external electrical power unit was used.
The Elastic Young Modulus was determined using the following method:

E, slope of the line passing through the point (ε_{max}, σ_{max}) of the 3rd cycle and the point (ε_{max}, σ_{max}) of the 8th cycle:

$$\boldsymbol{E} = \frac{\sigma_{max-cycle8} - \sigma_{max-cycle3}}{\varepsilon_{max-cycle8} - \varepsilon_{max-cycle3}} = \mathbf{5951}\ \boldsymbol{MPa}$$

The Young Modulus is calculated in this way, in order to have a results more representative of all the cylces, except the first two, that give more scattered data.

4. Shove Test Results

The purpose of this in situ test is to evaluate the shear resistance of the two horizontal bedjoints above and below a single unit block. Two flatjacks control the vertical load, and one small cylindrical jack controls the horizontal load.
In the following Figure it is possible to see the setup of the Shove test.
For further and more precise information about the procedure, see Annex 1.

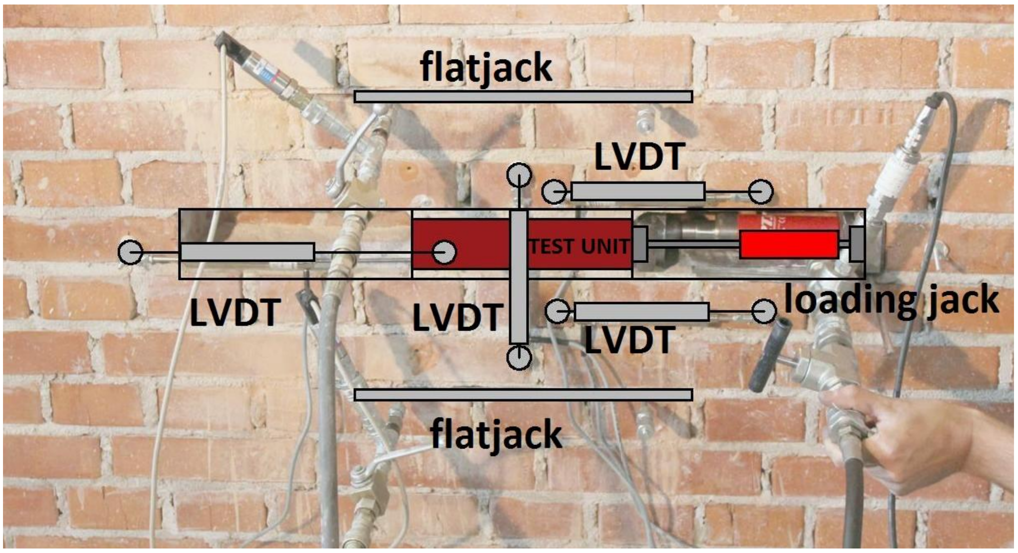


Figure 18 - Shove Test instrument configuration

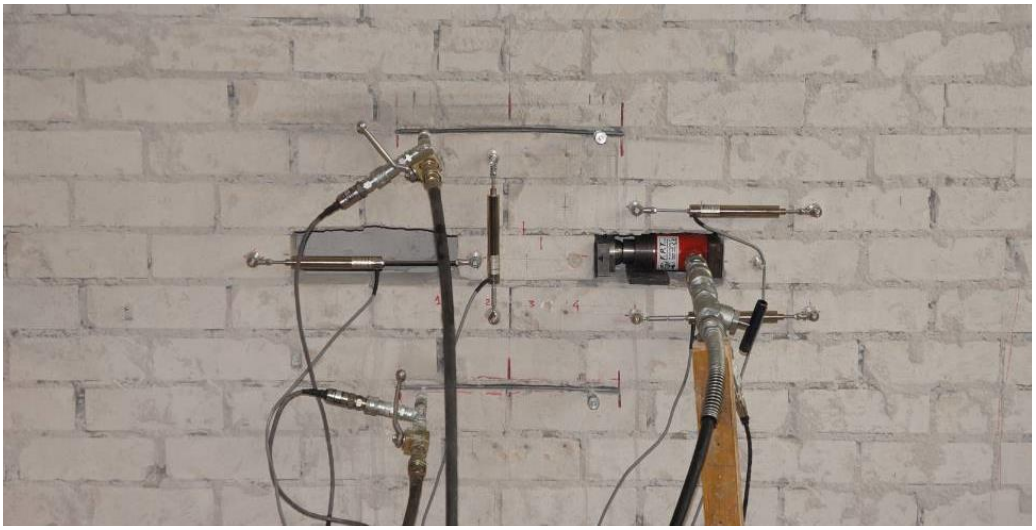


Figure 19 - Shove Test setup

The average compressive stress in the masonry, σ_j, in contact with the flatjacks is given as before by:

$$\sigma_j = K_a \left(a \cdot p + \frac{b}{A_{flj}} \right)$$

In the shove test, the output given by the instrumentation set at each level of vertical compression ensured by the two flatjacks, are essentially:

- LVDTs records (displacements)
- Flatjacks and cylindrical jack records (pressures)

The horizontal displacement, *D*, of the test unit at each horizontal load step, *i*, is:

$$D_i^{hor} = d_i^{hor} - d_{i-1}^{hor}$$

d_i^{hor} horizontal LVDT record at the load increment *i*

For each vertical pressure, it is possible to plot the horizontal load of the loading jack versus the horizontal displacement of the test unit. This will identify the point corresponding to the peak phase.

The peak phase can be identified in correspondence of a sudden decrease of the pressure in the horizontal loading jack and a considerable change of slope of the curve of the horizontal pressure vs. the horizontal displacement. The following Figures show clearly this phenomenon.

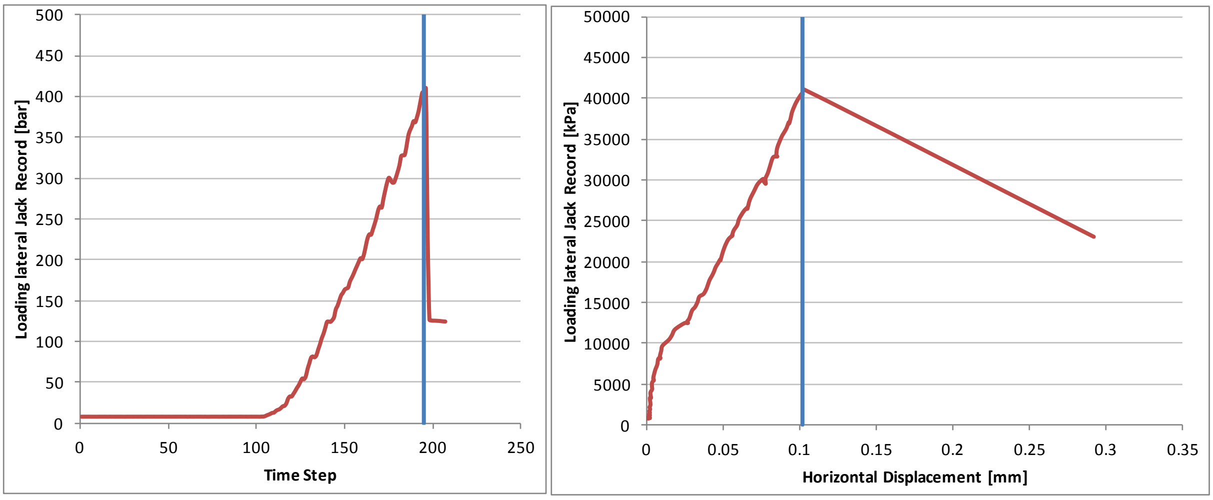


Figure 20 - Identification of the pick phase

For each level of vertical compression, two data were recorded:

σ_j normal compressive stress, in the masonry in contact with the flatjacks;
τ_i average mortar joint shear strength index; τ_i, for each level of f_m, is equal to:

$$\tau_i = \frac{P_{hi}}{A_j}$$

P_{hi} maximum horizontal force on the test unit at the i^{th} level of σ_j, (P_{hi} = pick phase in the current case);
A_j gross area of upper and lower bed joints (added) in the case of solid unit masonry.

Using a linear regression it is possible to obtain the joint shear strength index, τ, as a function of the normal compressive stress, in the masonry in contact with the flatjacks, σ_j:

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j$$

- The coefficient $\tilde{\mu}$ is the slope of the line relative to the linear regression of all the points (σ_j, τ), except the first. This because the first point is the one representative of the adhesion stress contribution to shear resistance, at low level of compressive stress.

- The joint shear stress index, $\tilde{\tau_0}$, at σ_j =0, is the Y-intercept of the linear function with: the angular coefficient equal to $\tilde{\mu}$, and passing through the point (σ_{j1}, τ_1)

Due to the very limited information available in the literature about the interpretation of this type of test (especially referred to single leaf masonry), Eucentre is now conducting a laboratory and numerical experimental campaign in order to riformulate the generic result $\tau(\sigma_j)$ with a more useful $\tau(\sigma_b)$ (where σ_b is the compressive stress of the interested brick):

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j \rightarrow \tau = \tau_0 + \mu(\sigma_b)$$

Shove test Results, location 1

The test has been conducted in this way:

- Set the pressure in the flatjacks at a low pressure (0.48 bar);
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Increase the pressure in the flatjacks up to approximately the one recorded in the single flatjack test (about 5 bar), and maintain it constant. The value imposed in this case was 4.3 bar;
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Increase the pressure in the flatjacks up to approximately 1.5 time the one recorded in the single flatjack test (5x15=7 bar), and maintain it constant. The value imposed in this case is 7.55 bar;
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, until the post peak phase (relative to horizontal direction).

In the following Table the results of the first Shove Test are reported:

σ_j: normal compressive stress in the masonry in contact with the flatjacks
σ_H : horizontal pressure (and force) in the loading lateral jack
τ: average mortar joint shear strength index (calculated as explained before)

Table 5 - Shove Test Results - location 1

σ_j MPa	$\sigma_{H\text{ - jack}}$ MPa	$\sigma_{H\text{ - jack}}$ bar	$\sigma_{H\text{ - jack}}$ ton	τ MPa	
0.04	**65.48**	**654.80**	**4.68**	**1.09**	**→ [□]**
0.339	**8.168**	**81.68**	**1.17**	**0.27**	**[□] ←**
0.585	**13.281**	**132.81**	**1.90**	**0.44**	**[□] ←**

The following Figure shows the points (σ_j, τ), coming from the results of the test, and the relation of shear strength index, τ, in function of σ_j

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j$$

In this case:

$\tilde{\tau}_0$= 1.057 MPa
$\tilde{\mu}$ = 0.766

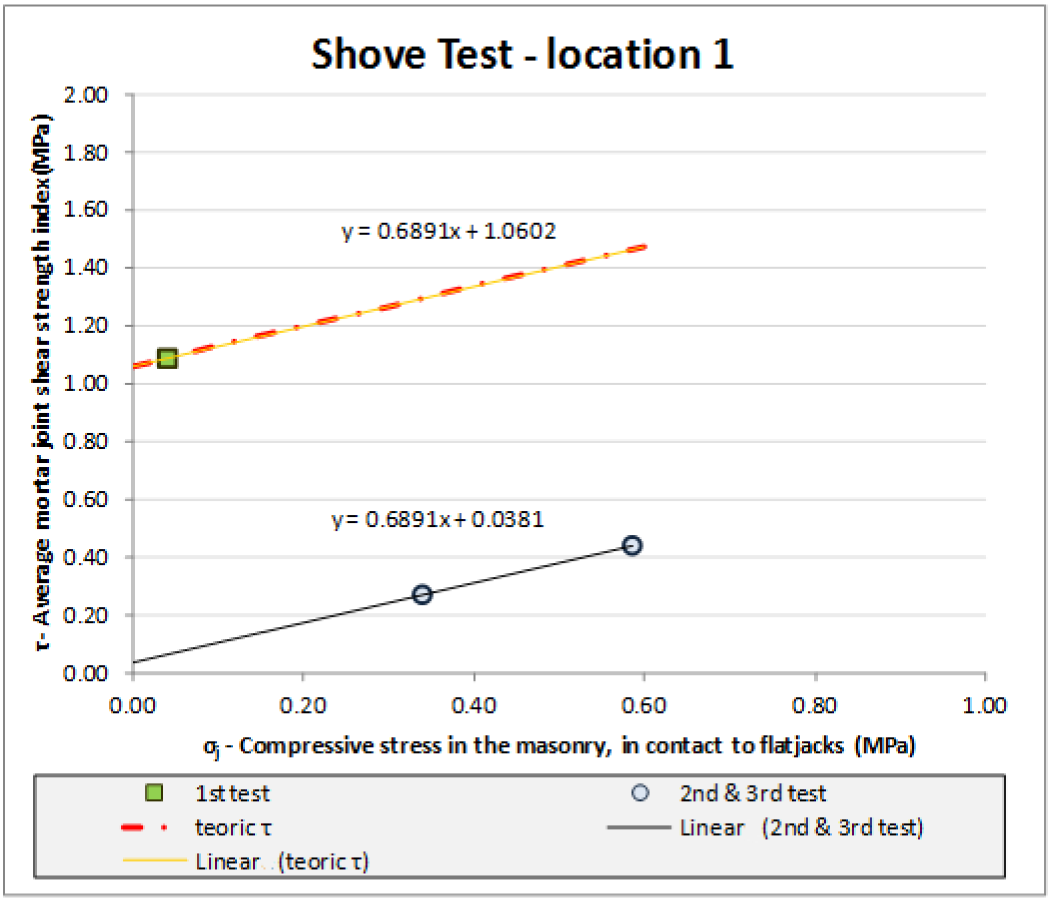


Figure 21 - Shove Test Results - τ (σ_j) graph - location 1

Shove test Results, location 2

The test has been conducted in this way:

- Set the pressure in the flatjacks at a low pressure (0.48 bar);
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Increase the pressure in the flatjacks up to 2 bar, and maintain it constant;
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Repeat the procedure increasing each time the pressure in the flatjacks. The maximum value imposed in this case was 7.55 bar;
- Reverse the load changing the position of the cylindrical jack and repeat the procedure.

In the following Table the results of the Shove Test are reported:

Table 6 - Shove Test Results - location 2

f_m MPa	$\sigma_{H\text{ - jack}}$ MPa	$\sigma_{H\text{ - jack}}$ bar	$\sigma_{H\text{ - jack}}$ ton	f_v MPa	
0.242	38.835	388.35	5.55	1.29	→ [□]
0.212	7.078	70.78	1.01	0.24	[□] ←
0.275	7.567	75.67	1.08	0.25	[□] ←
0.350	8.680	86.80	1.24	0.29	[□] ←
0.429	10.468	104.68	1.50	0.35	[□] ←
0.499	12.119	121.19	1.73	0.40	[□] ←
0.575	14.522	145.22	2.07	0.48	[□] ←
0.217	6.733	67.33	0.96	0.22	→ [□]
0.286	8.236	82.36	1.18	0.27	→ [□]
0.350	9.713	97.13	1.39	0.32	→ [□]
0.422	11.670	116.70	1.67	0.39	→ [□]
0.496	13.605	136.05	1.94	0.45	→ [□]
0.571	15.776	157.76	2.25	0.52	→ [□]

The following Figure shows the points (σ_j, τ), coming from the results of the test, and the relation of shear strength index, τ, in function of σ_j

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j$$

In this case:

$\tilde{\tau}_0$= 1.105 MPa
$\tilde{\mu}$ = 0.765

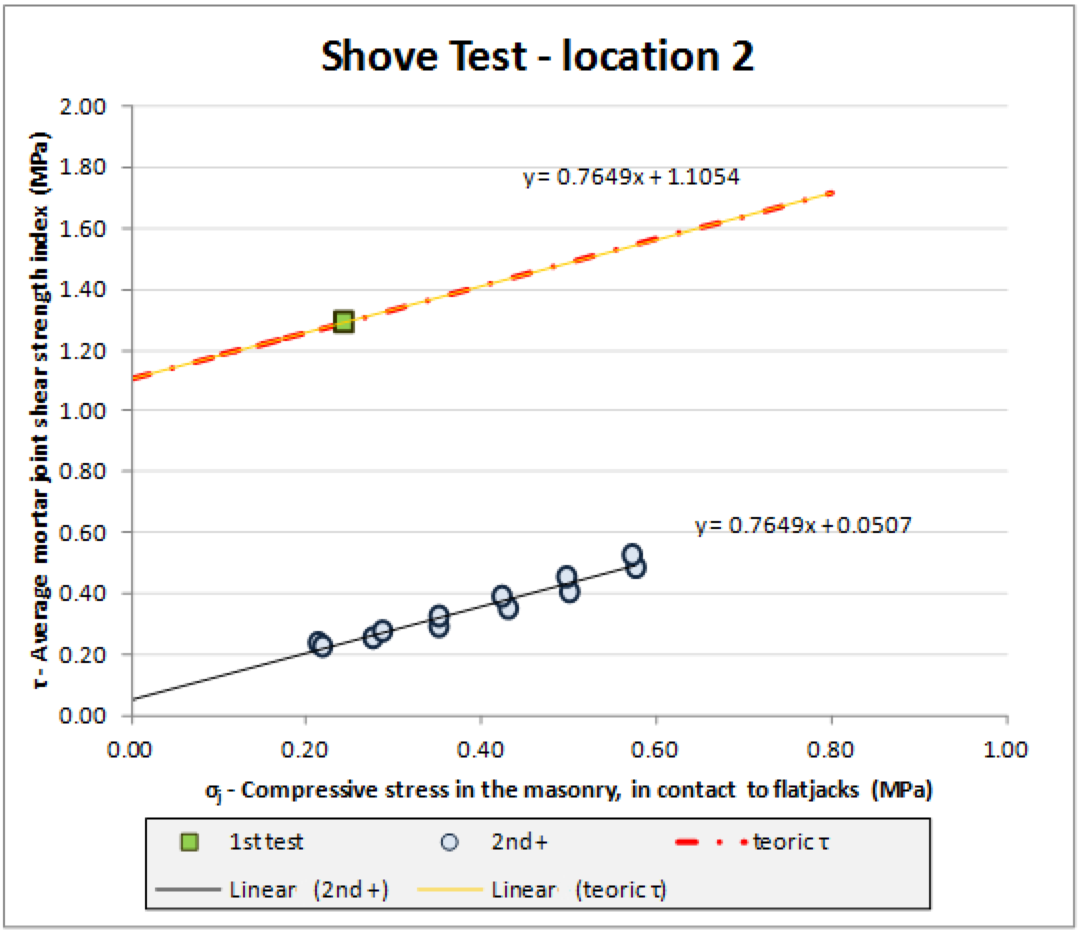


Figure 22 - Shove Test Results - τ (σ_j) graph - location 2

Shove test Results, location 3

During the test, before reaching the maximum force in the mortar, diffused cracks appeared in the masonry bricks near the test unit. Then the test was interrupted and it was decided to conduct it in a different location, 3 bis, on the same wall but sufficiently distant to not be disturbed by the first test attempt.

Shove test Results, location 4

The test has been conducted in this way:

- Set the pressure in the flatjacks at a low pressure (0.7 bar);
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Increase the pressure in the flatjacks up to 2 bar, and maintain it constant;
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Repeat the procedure increasing each time the pressure in the flatjacks. The maximum value imposed in this case was 7.91 bar;
- Reverse the load changing the position of the cylindrical jack and repeat the procedure.

In the following Table the results of the Shove Test are reported:

Table 7 - Shove Test Results - location 4

f_m MPa	$\sigma_{H\text{ - jack}}$ MPa	$\sigma_{H\text{ - jack}}$ bar	$\sigma_{H\text{ - jack}}$ ton	f_v MPa	
0.055	40.377	403.77	5.77	1.34	→ [□]
0.157	4.992	49.92	0.71	0.17	[□] ←
0.156	6.490	64.90	0.93	0.22	[□] ←
0.234	9.163	91.63	1.31	0.30	[□] ←
0.309	12.121	121.21	1.73	0.40	[□] ←
0.399	13.657	136.57	1.95	0.45	[□] ←
0.479	15.025	150.25	2.15	0.50	[□] ←
0.571	17.120	171.20	2.45	0.57	[□] ←
0.652	18.916	189.16	2.70	0.63	[□] ←
0.150	6.962	69.62	0.99	0.23	→ [□]
0.228	9.213	92.13	1.32	0.31	→ [□]
0.300	11.774	117.74	1.68	0.39	→ [□]
0.376	14.256	142.56	2.04	0.47	→ [□]
0.449	16.738	167.38	2.39	0.56	→ [□]
0.524	19.145	191.45	2.73	0.64	→ [□]
0.586	21.584	215.84	3.08	0.72	→ [□]

The following Figure shows the points (σ_j, τ), coming from the results of the test, and the relation of shear strength index, τ, in function of σ_j

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j$$

In this case:
$\tilde{\tau_0}$ = 1.2886 MPa
$\tilde{\mu}$ = 0.961

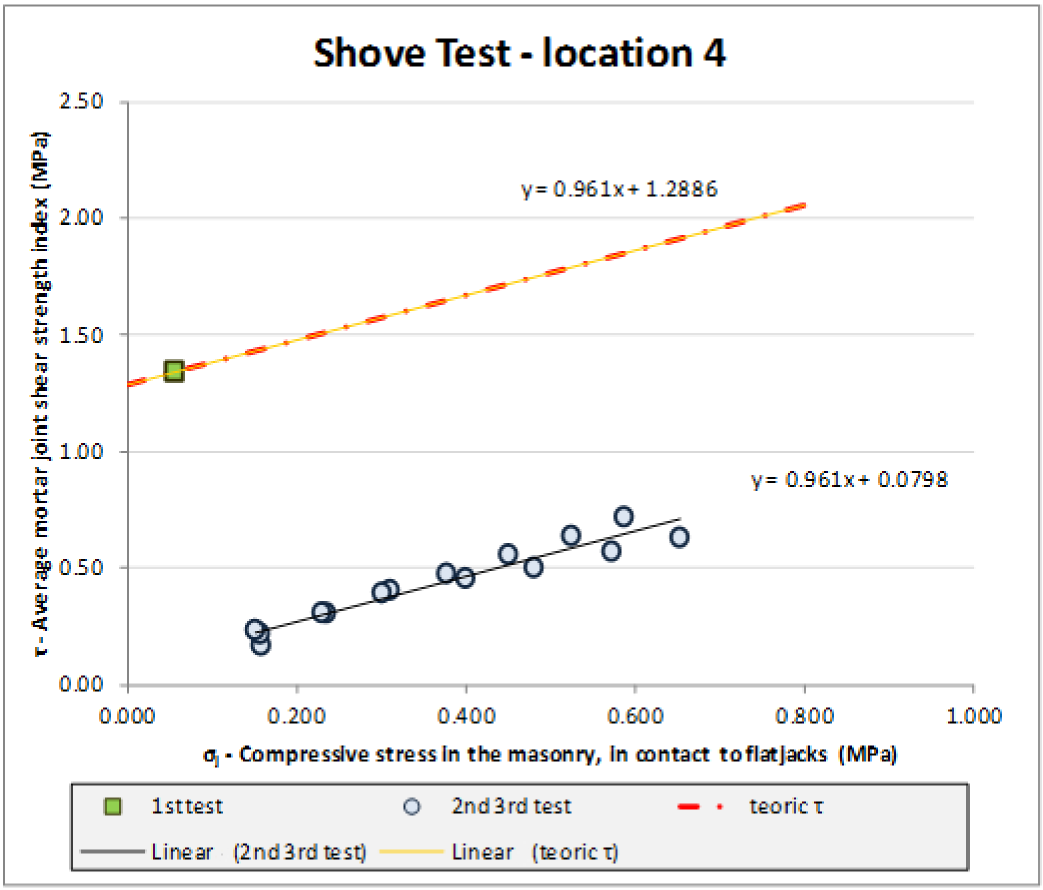


Figure 23 - Shove Test Results - τ (σ_j) graph - location 4

Shove test Results, location 3 bis

The test has been conducted in this way:

- Set the pressure in the flatjacks at a low pressure (0.58 bar);
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Increase the pressure in the flatjacks up to 2 bar, and maintain it constant;
- Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction);
- Repeat the procedure increasing each time the pressure in the flatjacks. The maximum value imposed in this case was 7.56 bar;
- Reverse the load changing the position of the cylindrical jack and repeat the procedure.

In the following Table the results of the Shove Test are reported:

Table 8 - Shove Test Results - location 3 bis

f_m MPa	$\sigma_{H\text{ - jack}}$ MPa	$\sigma_{H\text{ - jack}}$ bar	$\sigma_{H\text{ - jack}}$ ton	f_v MPa	
0.045	32.361	323.61	4.62	1.08	→ [□]
0.158	9.523	95.23	1.36	0.32	[□] ←
0.234	11.717	117.17	1.67	0.39	[□] ←
0.312	13.926	139.26	1.99	0.46	[□] ←
0.375	15.827	158.27	2.26	0.53	[□] ←
0.468	18.551	185.51	2.65	0.62	[□] ←
0.532	20.124	201.24	2.87	0.67	[□] ←
0.583	21.693	216.93	3.10	0.72	[□] ←

The following Figure shows the points (σ_j, τ), coming from the results of the test, and the relation of shear strength index, τ, in function of σ_j

$$\tau = \tilde{\tau}_0 + \tilde{\mu} \cdot \sigma_j$$

In this case:

$\tilde{\tau}_0$= 1.0324 MPa
$\tilde{\mu}$ = 0.9491

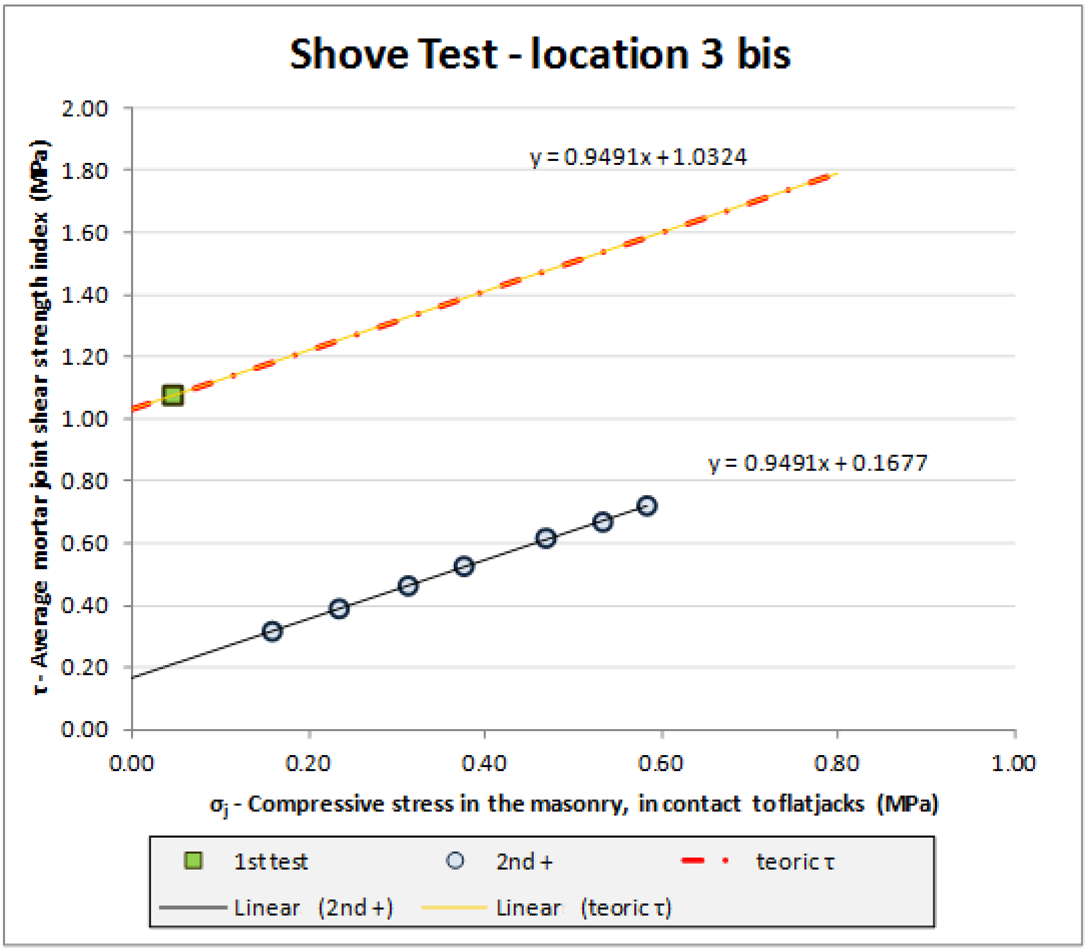


Figure 24 - Shove Test Results - τ (σ_j) graph - location 3 bis

Shove Test interpretation

The data obtained from the Shove Test have to be elaborated to obtain the relation between the shear strength of the mortar and the compression applied on it.

This chapter describes the procedure and the calculations done to achieve the relation between the shear strength of the mortar joint as a function of the effective vertical compression.

ASTM guideline: critical aspects

The current standard regarding the shove test interpretation is ASTM C1521-09: "Standard Test Measurement of Masonry Mortar Joint Shear Strength Index". This guideline gives a procedure for the test (substantially identical to the one used in Zijlvest) and an interpretation of the results.
The crucial issue interpreting a shove test is the evaluation of the effective normal compressive stress in the brick/test unit (σ_n in ASTM, σ_b in this document).
This stress is calculated by mean of a modification factor *j* that multiplies the stress applied by the flatjacks to the masonry σ_j. The document reports: *Analysis of an in situ shear test on a two-wythe brick masonry wall, has shown that the distribution of normal stress on the test unit s nonuniform, with the average stress equivalent to 1.7 times the applied flatjack stress. Hence, the modification factor j is equal to 1.7 for this case.*
The value of j is unique for this particular configuration … … Further analysis would have to be conducted to determine the actual state of normal stress acting on the test unit for other geometries and test configurations.

This interpretation procedure has some intrinsic critical aspects:

- The vertical compression in the wall (far field effect) is not taken in account. ASTM considers σ_b=0 if σ_j=0. Lab tests performed at Eucentre lab as well as nonlinear numerical models show that this is not the case, expecially for two-wythe masonry wall. Do not consider the compression of the brick with σ_j=0 (called σ_b^*) leads to unrealistic and overestimated results, expecially for vertically loaded two-wythe masonry wall;

- The authors of the studies and papers (dated 1990) on which the standard is based for the interpretation of the test results were directly contacted and they agree that the given interpretation is too simplistic and further calibration studies are needed.

For all these reasons, a new approach is proposed in this document, based on numerical and experimental studies carried out on purpose for this task. Among other aspects, the proposed approach accounts for σ_b^* (the compression of the brick with σ_j=0), this resulted to be imperative interpreting results for two-wythe masonry wall recently conducted at Eucentre Lab.

Eucentre Lab tests: useful observations

Several shove tests were performed in Eucentre lab in order to have a benchmark for the in situ shove tests performed in Holland. The test performed in lab were conducted on 2 two-wythe masonry walls, since they were built in order to simulate the first house that was originally planned to be tested by Eucentre (Molenweg 32). That building was tested by another company. The Eucentre laboratory

results can be used for the interpretation of results on double-wythe masonry, but they were also very useful in order to interpret the simpler test on a single-wythe wall (e.g. a leaf of a cavity wall).
The testing campaign consists of 8 lab-simulated shove tests performed on two walls and a series of triplets built with the same masonry (low strength mortar f_m=4.2 MPa and clay bricks). The test setup allowed to control the force on the top of the wall, in order to simulate the presence of a pre-existing load and study its influence on the results. Figure 25 shows two pictures, on the left one of the loaded walls and on the right a test on a triplet.

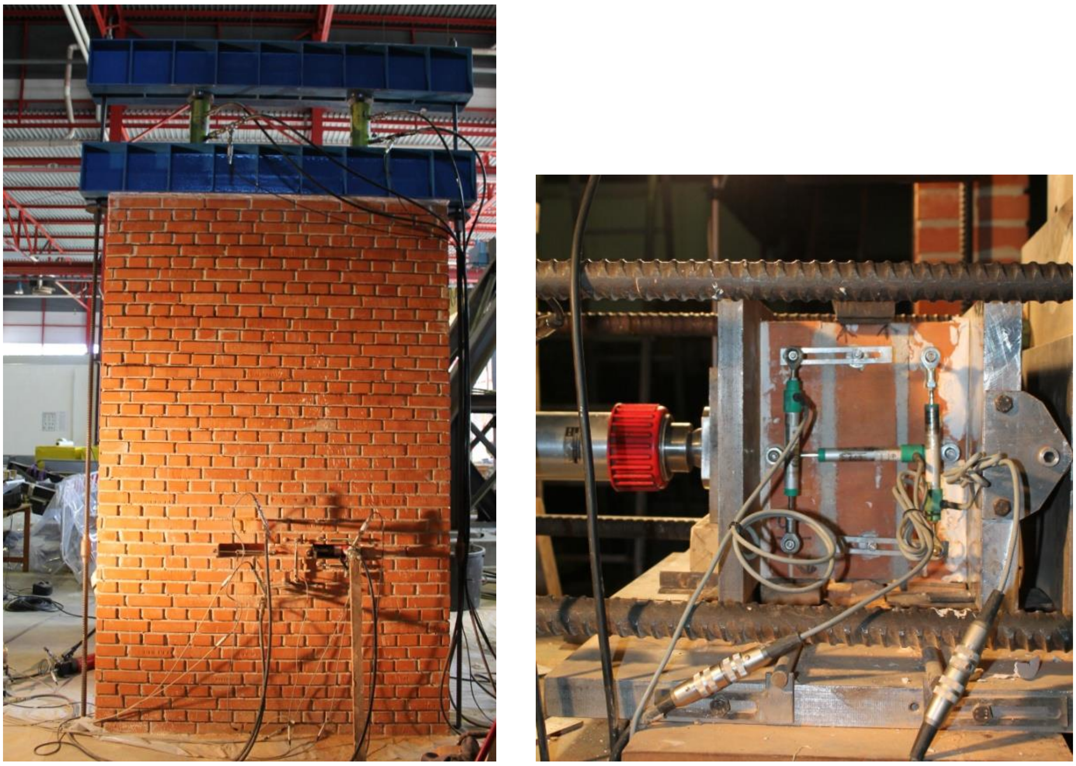


Figure 25 – Test setup for a loaded wall (left) and for a tripley (right)

The tests on triplets were aimed to have a sufficiently reliable measure of the bedjoint shear strengths under controlled normal stress, which was taken as a reference for the interpretation of the shove tests. Figure 26 presents the σ-τ strength envelope obtained from the triplet tests, showing how the usual Coulomb-type strength model is very suitable to interpret the results.

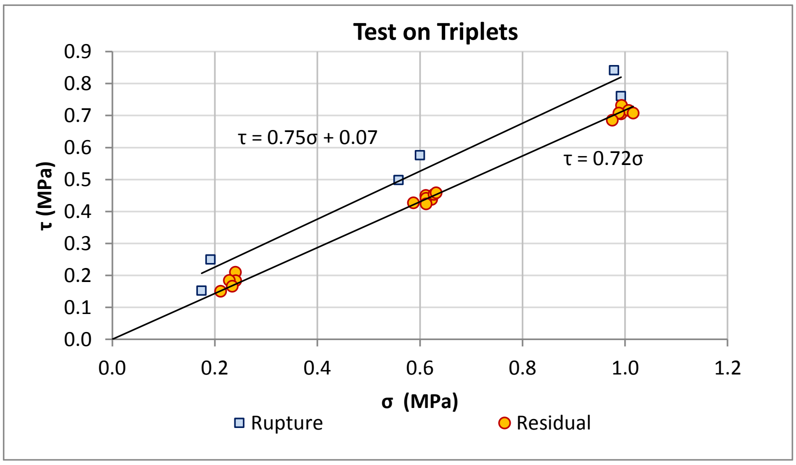


Figure 26 – σ-τ strength envelope obtained from the triplet tests

Regarding the lab simulated shove tests, one of the main conclusion was that the vertical compression in the brick resulted to be greater than 0 (σ_b>0) even for a stress in the masonry due to flatjacks equal to 0 (σ_j=0). This is due to the load on the top of the wall, and it is particularly evident when the test is performed only on one side of a thick wall. This is due to the complex stress distribution in the masonry in the proximity of the tested brick.
Figure 27 represents the phenomenon. The graph shows the result of a lab shove test (i.e. #6) with different values of vertical compression in the wall. The first point is the rupture point (peak strength, influenced by cohesion + friction), while the lines represent the friction coefficient found from the post-peak the residual strength. The residual shear strength was measured at three different levels of σ_j , and with three different levels of vertical load at the top of the wall (σ is the corresponding mean stress by simply dividing the load by the gross area of the wall). If the normal stress in the masonry in direct contact with the flatjack (σ_j) was a truthful measure of the normal stress in the brick that is being pushed in the shove test (σ_b), it should be expected that the line interpolating the residual strength points would pass very close to the origin of the axes (no cohesion).
The fact that this is not the case means that, if we consider as correct a Coulomb strength criterion (verified by the tests on triplets), the compression of the brick is in reality greater than zero also with no pressure in the flatjacks.

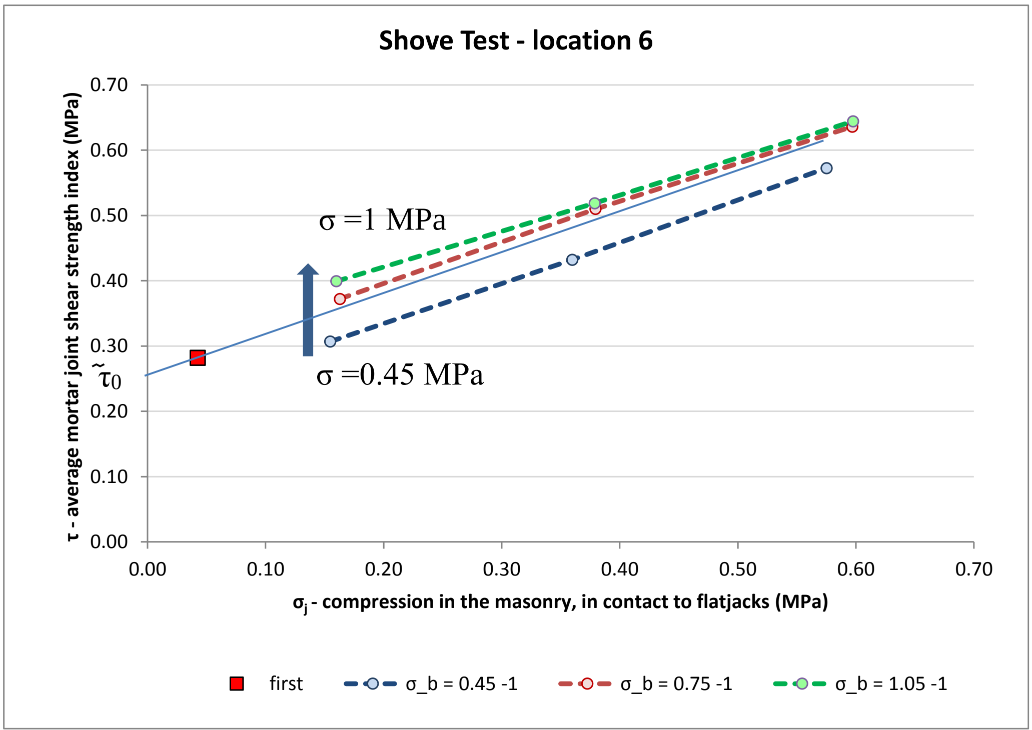


Figure 27 – Results of Shove Test # 6: influence of vertical load

This means that the points in the graph, if the effective compression in the brick σ_b is considered, will have a different abscissa (a greater value). As written before this normal stress shift is not negligible in case of walls with pre-existing stress coming from top loads or if the shove test is performed on the external brick of a thicker two-wythe.
In the following a way to estimate the compression of the brick when σ_j=0 (this value of σ_b is defined as σ_b^*) will be presented directly applied to Zijlvest tests. This procedure uses the results of the test performed after the first rupture to compute the friction coefficient.

By applying the proposed procedure it was possible to find a good agreement between the triplet tests and the simulated lab shove tests (Figure 28). As it is possible to observe the τ_0 is considerably lower than the one obtained by mean of uncorrected shove test data ($\tilde{\tau}_0$), and the regression line for the shove test data is very close to the regression line derived by triplet tests.

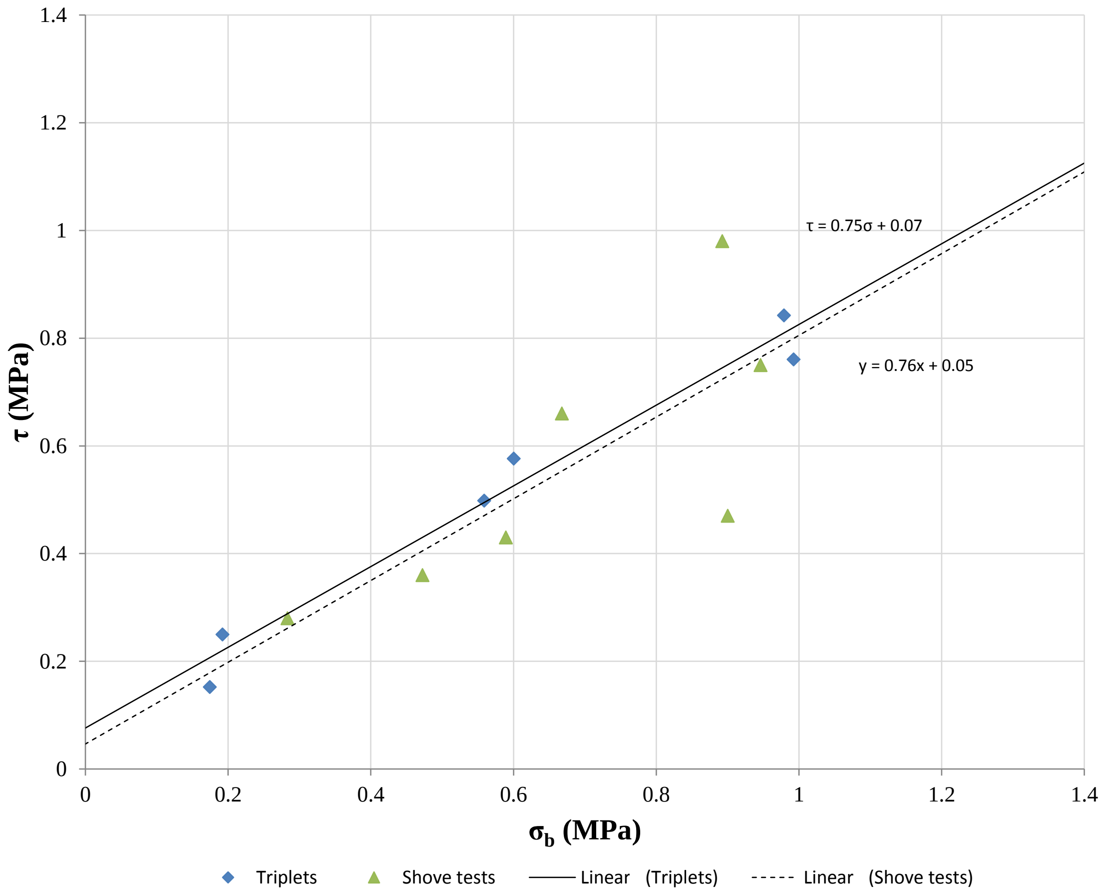


Figure 28 – Comparison between lab shove test and triplets

FEM models are also being developed to further support the proposed procedure for the case of thick walls (two wythes or more). The procedure was applied to Zijlvest test results by mean of a 2D nonlinear FEM, this was possible due to the particular geometry of the test in that thinner one-wythe wall.

Interpretation Procedure

The relation between the normal compressive stress (σ_b) in the brick unit test, and the joint shear stress index (τ), can be described by Coulomb's law:

$$\tau = \tau_0 + \mu \cdot \sigma_b$$

τ_0: adhesion stress: joint shear stress index at zero normal compressive stress ($\sigma_b=0$)
μ: friction coefficient of the masonry

The problem is to determine the correct value of the normal compressive stress in the brick unit test. While the compressive stress in the masonry, in contact with the flatjacks can be evaluated from equation 1 at page 9, a direct measure of σ_b is not immediately estimable.
Factors that affect the normal compressive stress in the tested brick unit are:

- the normal compressive stress, in the masonry in contact with the flatjacks, σ_j
- the geometry of the Shove Test setup
- the vertical load on the test unit, due to the self weight of the wall and the load applied at the top of the wall.
- possible local effects such as the vertical dilatancy of the joint while it undergoes shear failure (which at this stage is assumed to be negligible compared to other factors)

It is assumed that σ_{bj}, i.e. the normal compressive stress in the test unit, due to flatjacks pressure, can be expressed as the product:

$$\sigma_{bj} = k_{bj} \cdot \sigma_j$$

where
k_{bj} jack to brick correction factor (nonlinear analysis)
σ_j normal compressive stress, in the masonry in contact with the flatjacks

σ_{bj} normal compressive stress, in the test unit, due to flatjacks pressure

Hence,

$$\tau = \tilde{\tau}_0 + \mu(k_{bj} \cdot \sigma_j) = \tilde{\tau}_0 + \mu(\sigma_{bj})$$

with:

τ joint shear strength index
μ the slope of the line, relative to the linear regression of all the residual strength points (σ_{bj}, τ),
$\tilde{\tau}_0$ the joint shear stress index, τ, at σ_{bj}=0

The coefficient, k_{bj}, takes into account the geometry of the test setup. This correction factor is calculated using a non linear model as described in next paragraphs.

Then two equations can be written:

$$\begin{cases} \text{"corrected"} & \tau = \tau_0 + \mu(\sigma_b) & (1a) \\ \text{"uncorrected"} & \tau = \tilde{\tau}_0 + \mu(\sigma_{bj}) & (2) \end{cases}$$

It is fundamental to find a relation between σ_b and σ_{bj}, in order to obtain (1).
The hypothesis is that the normal correct compressive stress on the test unit (σ_b) is a sum of two terms:

$$\sigma_b = \sigma_{bj} + \sigma_b{}^*$$

with:

σ_{bj} normal compressive stress, in the test unit, due to flatjacks pressure

σ_b* contribution to normal compressive stress in the test unit due to far field effect (e.g. the self weight of the wall and the load applied at the top of the wall)

The two equations become:

$$\begin{cases} \tau = \tau_0 + \mu(\sigma_{bj} + \sigma_b{}^*) & (1b) \\ \quad \tau = \tilde{\tau}_0 + \mu(\sigma_{bj}) & (2) \end{cases}$$

The residual shear strength with zero normal stress must be zero by definition. The uncorrected experimental data would show on the contrary a virtual residual stress $\tilde{\tau}_{0\text{-}res}$.

The σ_b* depends on the existing stress prior to the “cuts and slots”, and the geometry of the cuts and slots. It could be theoretically calculated by mean of a FEM model reproducing the geometry of the cuts and slots and the far field boundary condition . A boundary condition of this model would be the stress in the masonry measured by the single flat jack test (representative of the undisturbed “prior” stress). This would however produce results that would be affected by the uncertainty of the single flatjack test (whose results are may not be reliable in some cases). For this reasons it is proposed to introduce a method that estimates the σ_b* making using of the shove test residual strength results.
If the Coulomb model is valid, the value of virtual residual adhesion stress, $\tilde{\tau}_{0\text{-}res}$, is in reality due to σ_b*. All the residual strength points (σ_{bj}, τ), turn out to be well aligned along a linear fit, and the value of μ is determined with quite good precision. Then, by knowing μ it is possible to calculate indirectly the value of σ_b* as shown in the following graph:

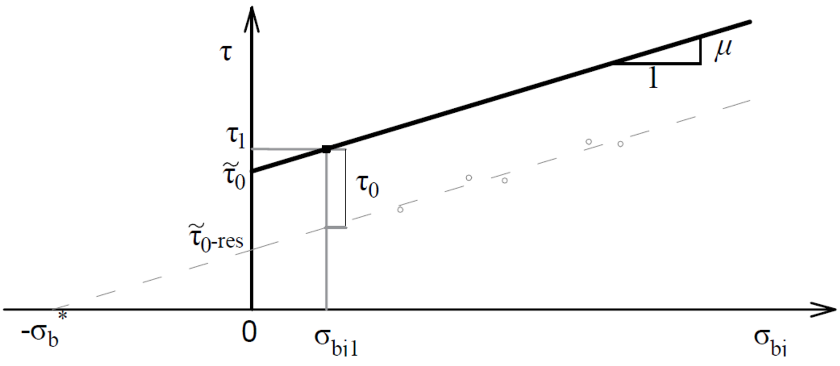


Figure 29 – Graphical representation of the shove test interpretation

from which:

$$\tilde{\tau}_{0-res} = \mu\sigma_b{}^* \quad (3)$$

and:

$$\sigma_b{}^* = \frac{\tilde{\tau}_{0-res}}{\mu} \quad (4)$$

At this stage, the graph could be re-plotted with a translation of the reference system: from σ_{bj}, τ to σ_b, τ: $(\sigma_b = \sigma_{bj} + \sigma_b{}^*)$

Notice that the ASTM guidelines suggests to conduct the first test with a low pressure in the flatjacks in order to have a better estimation of the cohesion coefficient. In case of heavily loaded or two-wythe masonry wall the effective compressive stress in the brick σ_b could be relatively high also for low flatjacks pressure. To not consider this effect leads to a non negligible overestimation of the cohesive strength τ_0.

At this stage the first point (σ_{bj1}, τ_1) is defined in the new reference system (σ_{b1}, τ_1). The value of μ remains the same. On the other hand, the value of τ_0 is determined as the interecept of the new reference system (σ_b, τ), with the τ axis (σ_b=0).

Definition of the jack-to-brick correction factor

The pressure in the test unit due only to the flatjack pressure, σ_{bj}, is equal to:

$$\sigma_{bj} = k_{bj} \cdot \sigma_j$$

With:

σ_j normal compressive stress, in the masonry in contact with the flatjacks

k_{bj} jack-to-brick correction factor

σ_{bj} normal compressive stress, in the test unit, due to flatjacks pressure

The coefficient, k_{bj}, takes into account the geometry of the test setup.
In order to determine this correction factor, non-linear F.E.M. model analyses have been carried out. The models were reproducing the shove test setup for each location. In the following Figure it is possible to see the discretization of a wall using Sap2000 program for the calculation of σ_{bj}.
In the present phase of the work, the constitutive law consisted of linear elastic behaviour in compression and zero tensile strength ("no-tension").
Also, at the top of the wall, a uniformly distributed load was imposed in order to attain the same stress at the flatjack height, measured in situ with the single flatjack test.

For each configuration (i.e. for each top wall load imposed), different values of pressure in the flatjack slots were applied, and then the average compression stress value in the unit test was calculated each time (section cut integration). In this way it is possible to have the value of the compression in the test unit brick due to flatjack pressure, and to calculate the k_{bj} correction factor.

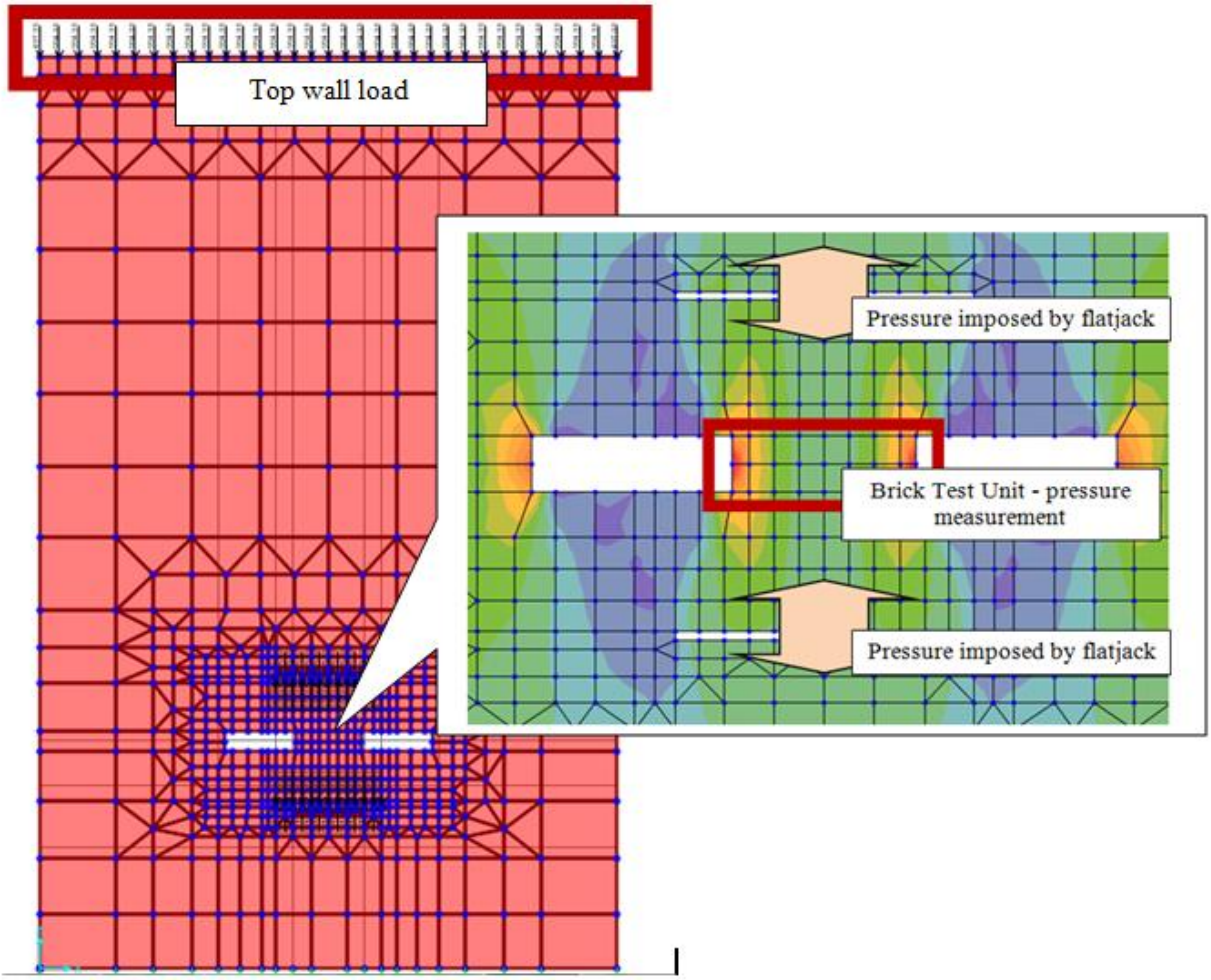


Figure 30 – Non linear FEM model

From the non linear models, the jack to brick correction factor for this particular geometry (consistent with the tests carried out in Zijlvest 25), due only to flatjack pressure, was found to be equal to:

$$k_{bj} = 1.28$$

Calculations and Results

<u>Location 1</u>

Table 9 - Shove test data, location 1

	k_{bj} = 1.28			From the trend line		$=\tilde{\tau}_{0\text{-res}}/\mu$	$= \sigma_{bj} + \sigma_b$*	$=\tilde{\tau}_0 - \tilde{\tau}_{0\text{-res}}$
	σ_j MPa	σ_{bj} MPa	τ MPa	$\tilde{\tau}_{0\text{-res}}$ MPa	μ -	σ_b* MPa	σ_b MPa	τ_0 MPa
→ [□]	0.040	0.051	1.088	0.038	0.538	0.071	0.122	1.022
[□] ←	0.339	0.433	0.271	0.038	0.538	0.071	0.504	1.022
[□] ←	0.585	0.749	0.441	0.038	0.538	0.071	0.820	1.022
			mean	0.038	**0.538**	0.071	0.482	**1.022**

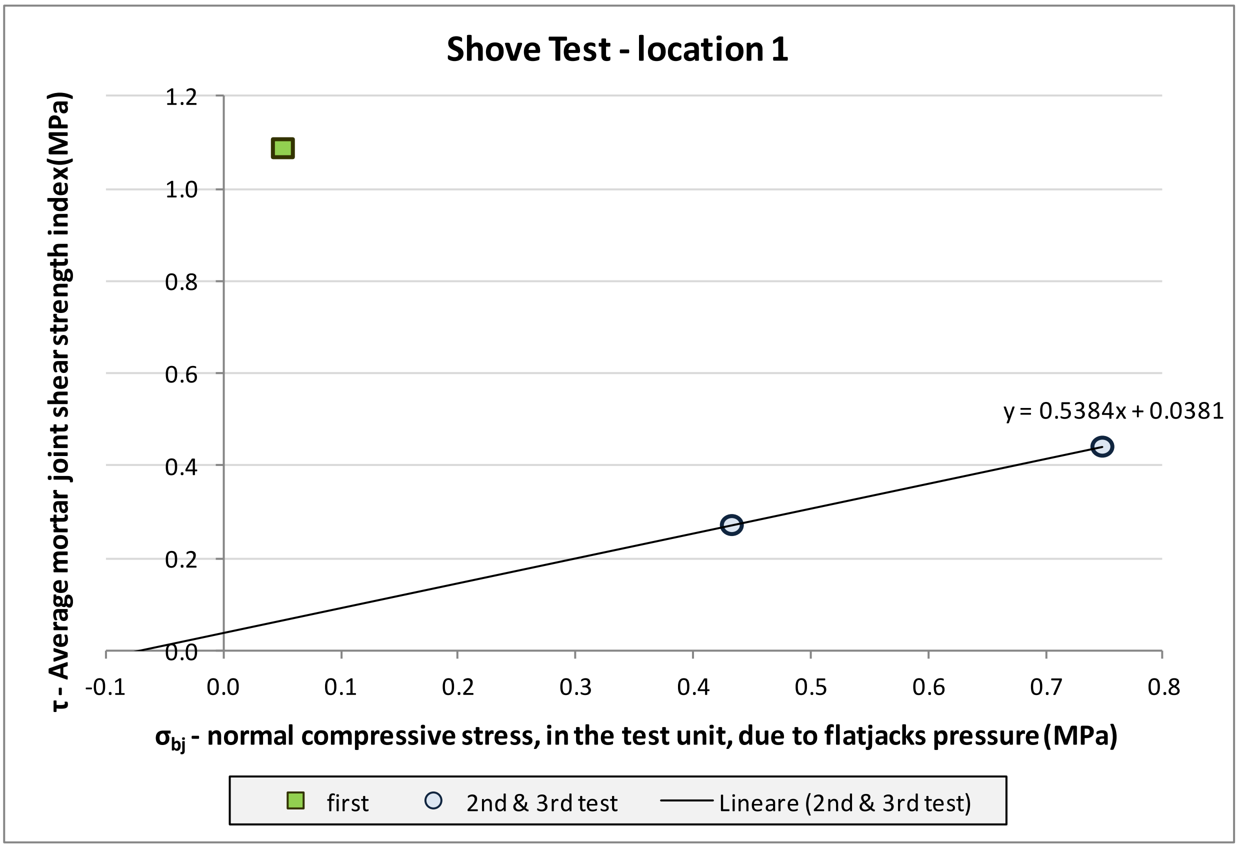


Figure 31 – Shove Test Results, σ_{bj},τ graph, location 1

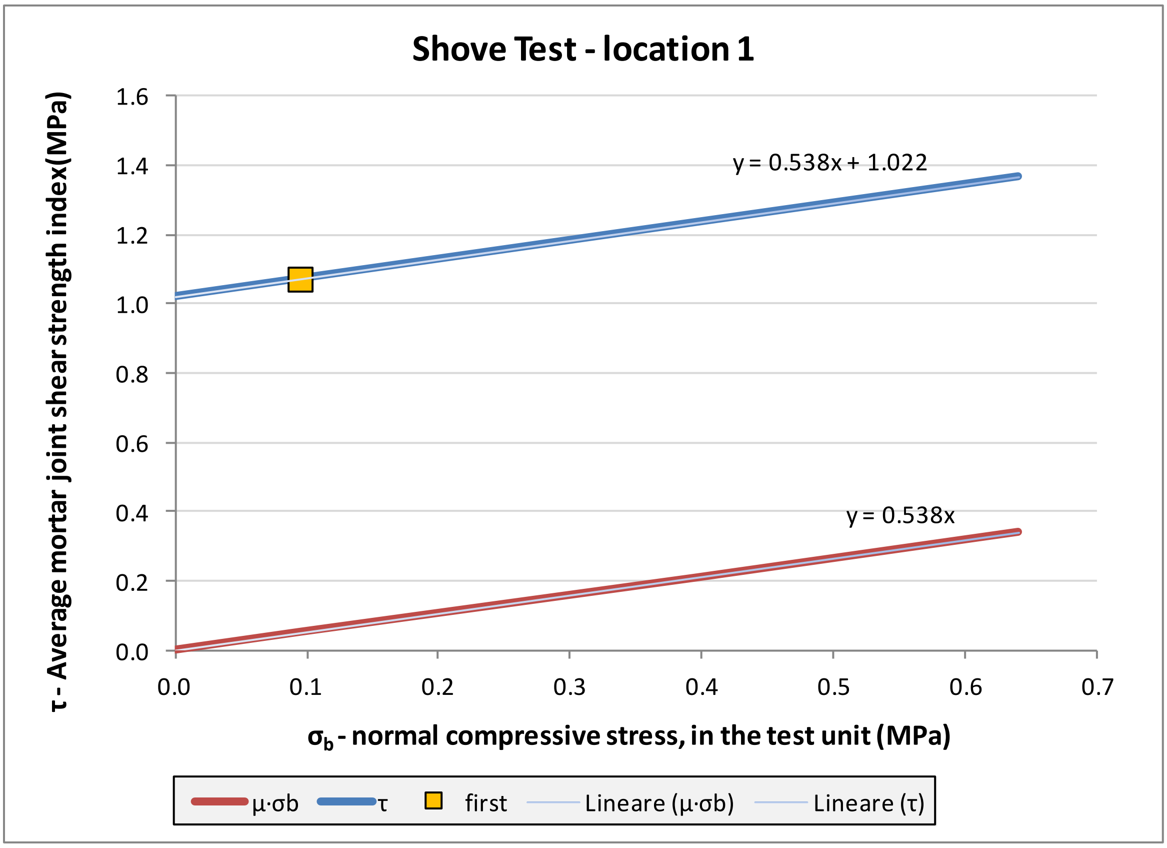


Figure 32 – Shove Test Interpretation of Results, σ_b,τ graph, location 1

Location 2

Table 10 - Shove test data, location 2

	k_{bj} =	1.28		From the trend line		$=\tilde{\tau}_{0\text{-res}}/\mu$	$= \sigma_{bj} + \sigma_b^*$	$=\tau_1-\mu\cdot\sigma_{bj1}$	$=\tilde{\tau}_0-\tilde{\tau}_{0\text{-res}}$
	σ_j MPa	σ_{bj} MPa	τ MPa	$\tilde{\tau}_{0\text{-res}}$ MPa	μ -	σ_b^* MPa	σ_b MPa	$\tilde{\tau}_0$ MPa	τ_0 MPa
→ [□]	0.242	0.309	1.290	0.051	0.598	0.085	0.394	1.105	1.055
[□] ←	0.212	0.272	0.235	0.051	0.598	0.085	0.357	1.105	1.055
[□] ←	0.275	0.352	0.251	0.051	0.598	0.085	0.437	1.105	1.055
[□] ←	0.350	0.448	0.288	0.051	0.598	0.085	0.533	1.105	1.055
[□] ←	0.429	0.550	0.348	0.051	0.598	0.085	0.634	1.105	1.055
[□] ←	0.499	0.639	0.403	0.051	0.598	0.085	0.724	1.105	1.055
[□] ←	0.575	0.736	0.482	0.051	0.598	0.085	0.821	1.105	1.055
→ [□]	0.217	0.278	0.224	0.051	0.598	0.085	0.363	1.105	1.055
→ [□]	0.286	0.366	0.274	0.051	0.598	0.085	0.451	1.105	1.055
→ [□]	0.350	0.448	0.323	0.051	0.598	0.085	0.533	1.105	1.055
→ [□]	0.422	0.540	0.388	0.051	0.598	0.085	0.625	1.105	1.055
→ [□]	0.496	0.635	0.452	0.051	0.598	0.085	0.720	1.105	1.055
→ [□]	0.571	0.731	0.524	0.051	0.598	0.085	0.816	1.105	1.055
			mean	0.051	**0.598**	0.085	0.570	1.105	**1.055**

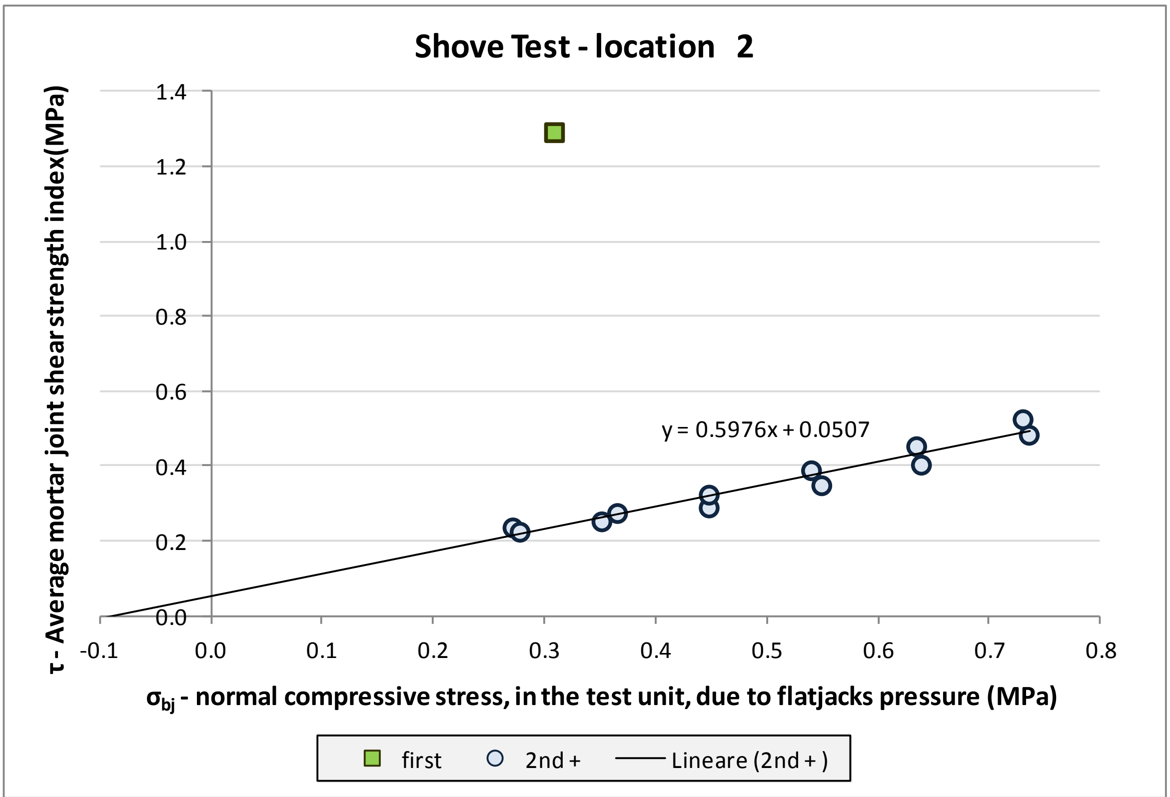


Figure 33 – Shove Test Results, σ_{bj},τ graph, location 2

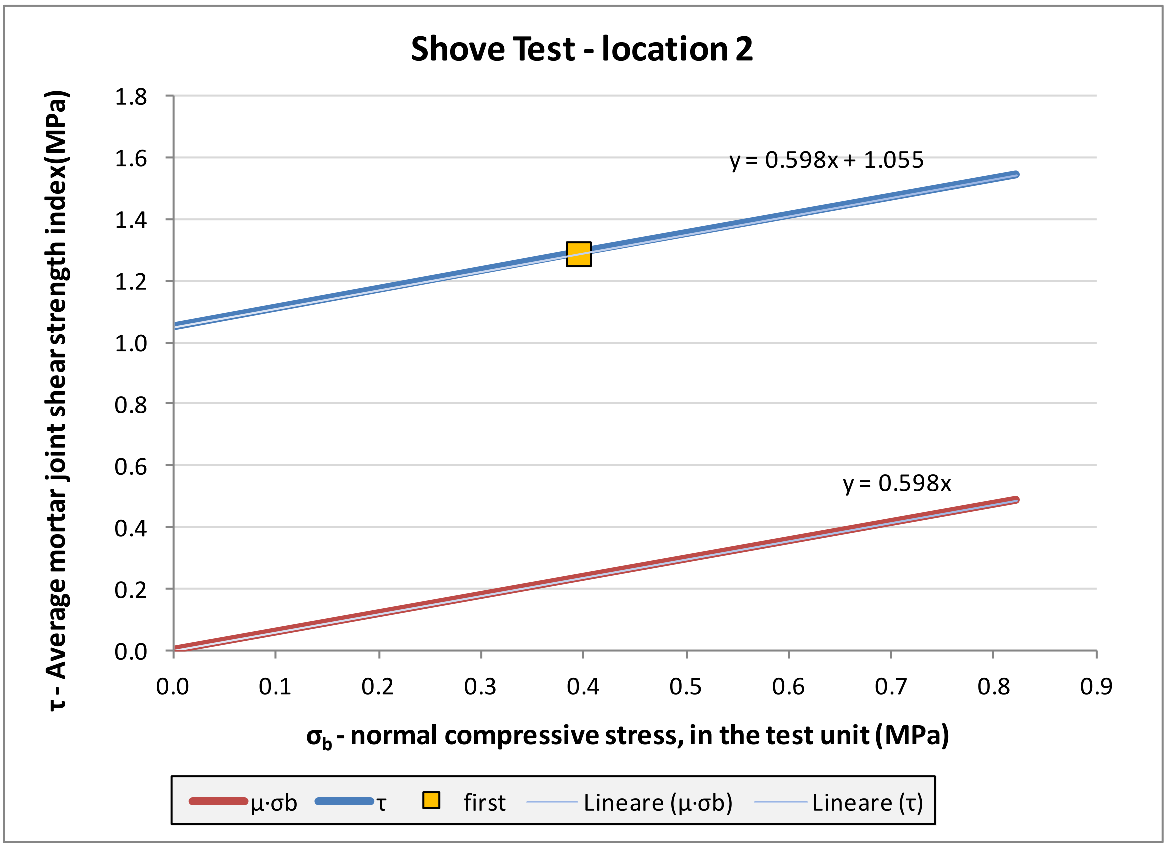


Figure 34 – Shove Test Interpretation of Results, σ_b,τ graph, location 2

Location 4

Table 11 - Shove test data, location 4

	k_{bj} =	1.28		From the trend line		$=\tilde{\tau}_{0-res}/\mu$	$=\sigma_{bj}+\sigma_b^*$	$=\tau_1-\mu\cdot\sigma_{bj1}$	$=\tilde{\tau}_0-\tilde{\tau}_{0-res}$
	σ_j	σ_{bj}	τ	$\tilde{\tau}_{0-res}$	μ	σ_b^*	σ_b	$\tilde{\tau}_0$	τ_0
	MPa	MPa	MPa	MPa	-	MPa	MPa	MPa	MPa
→ [□]	0.055	0.070	1.341	0.080	0.751	0.106	0.177	1.289	1.209
[□] ←	0.157	0.201	0.166	0.080	0.751	0.106	0.307	1.289	1.209
[□] ←	0.156	0.199	0.216	0.080	0.751	0.106	0.306	1.289	1.209
[□] ←	0.234	0.299	0.304	0.080	0.751	0.106	0.406	1.289	1.209
[□] ←	0.309	0.396	0.403	0.080	0.751	0.106	0.502	1.289	1.209
[□] ←	0.399	0.510	0.454	0.080	0.751	0.106	0.616	1.289	1.209
[□] ←	0.479	0.614	0.499	0.080	0.751	0.106	0.720	1.289	1.209
[□] ←	0.571	0.731	0.569	0.080	0.751	0.106	0.838	1.289	1.209
[□] ←	0.652	0.835	0.628	0.080	0.751	0.106	0.941	1.289	1.209
→ [□]	0.150	0.192	0.231	0.080	0.751	0.106	0.298	1.289	1.209
→ [□]	0.228	0.292	0.306	0.080	0.751	0.106	0.399	1.289	1.209
→ [□]	0.300	0.383	0.391	0.080	0.751	0.106	0.490	1.289	1.209
→ [□]	0.376	0.481	0.474	0.080	0.751	0.106	0.588	1.289	1.209
→ [□]	0.449	0.575	0.556	0.080	0.751	0.106	0.681	1.289	1.209
→ [□]	0.524	0.670	0.636	0.080	0.751	0.106	0.777	1.289	1.209
→ [□]	0.586	0.750	0.717	0.080	0.751	0.106	0.856	1.289	1.209
			mean	0.080	**0.751**	0.106	0.556	1.289	**1.209**

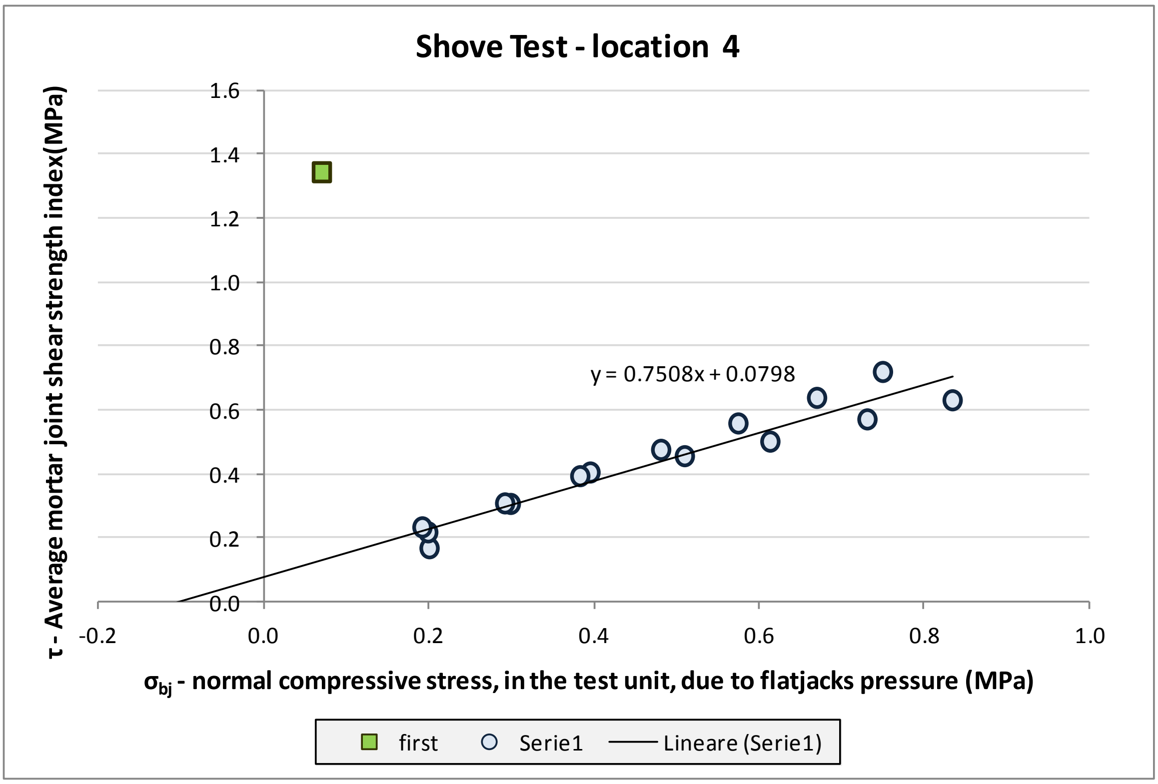


Figure 35 – Shove Test Results, σ_{bj},τ graph, location 4

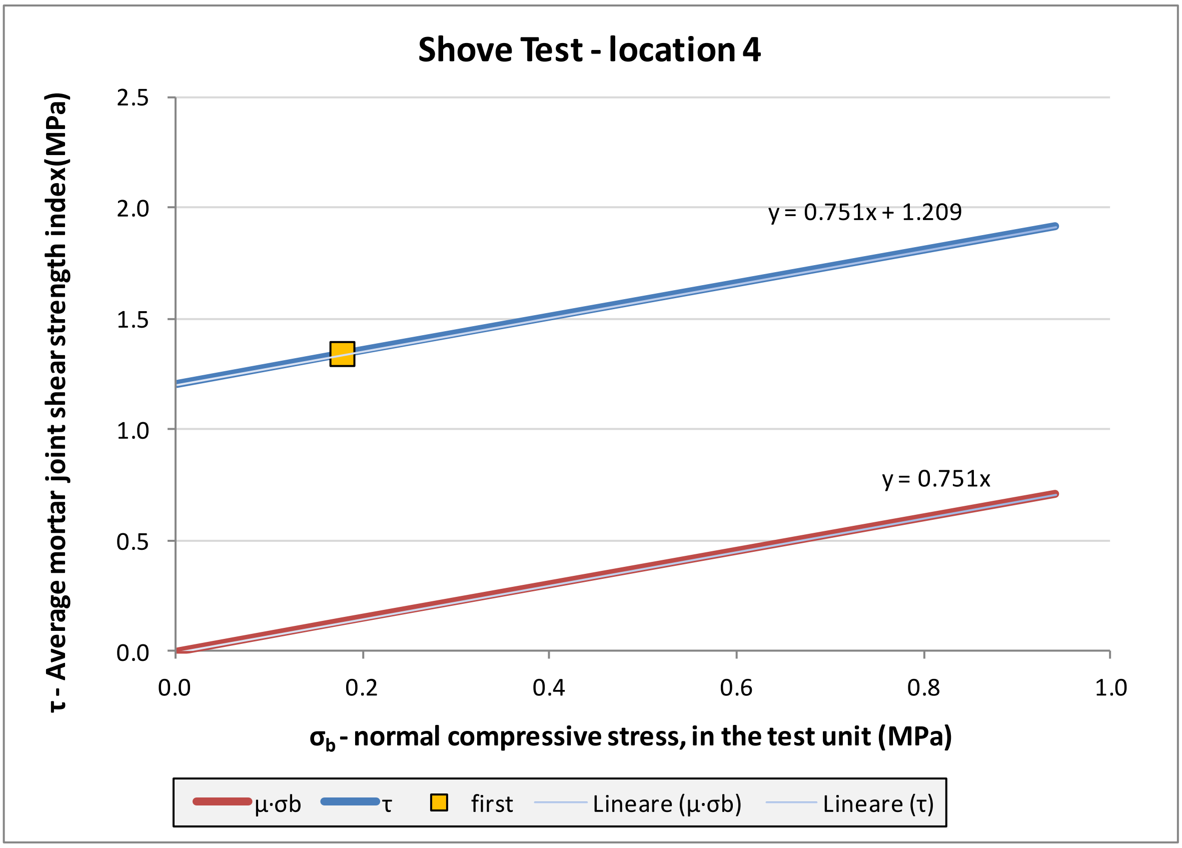


Figure 36 – Shove Test Interpretation of Results, σ_b,τ graph, location 4

Location 3 bis

Table 12 - Shove test data, location 3 bis

	k_{bj} =	1.28		From the trend line		$=\tilde{\tau}_{0\text{-res}}/\mu$	$=\sigma_{bj}+\sigma_b$*	$=\tau_1-\mu\cdot\sigma_{bj1}$	$=\tilde{\tau}_0-\tilde{\tau}_{0\text{-res}}$
	σ_j MPa	σ_{bj} MPa	τ MPa	$\tilde{\tau}_{0\text{-res}}$ MPa	μ -	σ_b* MPa	σ_b MPa	$\tilde{\tau}_0$ MPa	τ_0 MPa
→ [□]	0.045	0.058	1.075	0.168	0.742	0.226	0.284	1.032	0.865
[□] ←	0.158	0.202	0.316	0.168	0.742	0.226	0.428	1.032	0.865
[□] ←	0.234	0.299	0.389	0.168	0.742	0.226	0.525	1.032	0.865
[□] ←	0.312	0.399	0.463	0.168	0.742	0.226	0.625	1.032	0.865
[□] ←	0.375	0.480	0.526	0.168	0.742	0.226	0.707	1.032	0.865
[□] ←	0.468	0.599	0.616	0.168	0.742	0.226	0.825	1.032	0.865
[□] ←	0.532	0.681	0.669	0.168	0.742	0.226	0.908	1.032	0.865
[□] ←	0.583	0.746	0.721	0.168	0.742	0.226	0.972	1.032	0.865
			mean	0.168	**0.742**	0.226	0.659	1.032	**0.865**

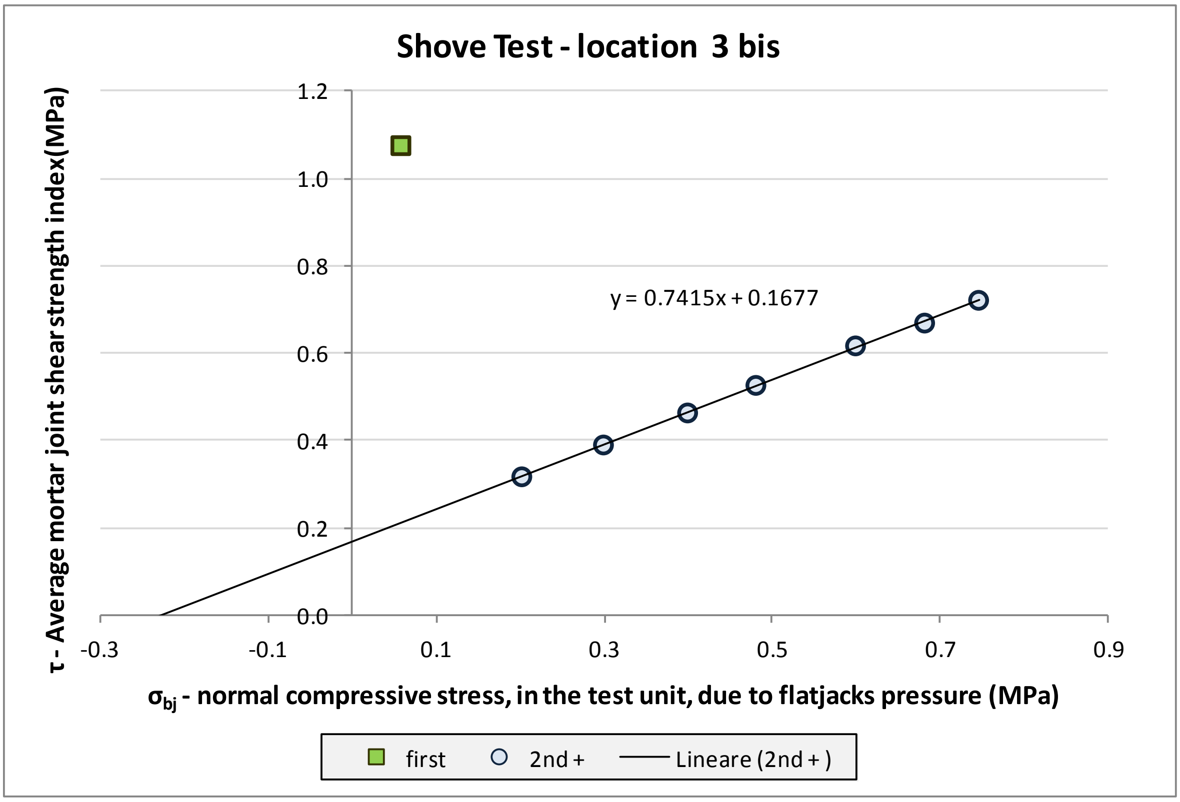


Figure 37 – Shove Test Results, σ_{bj},τ graph, location 3 bis

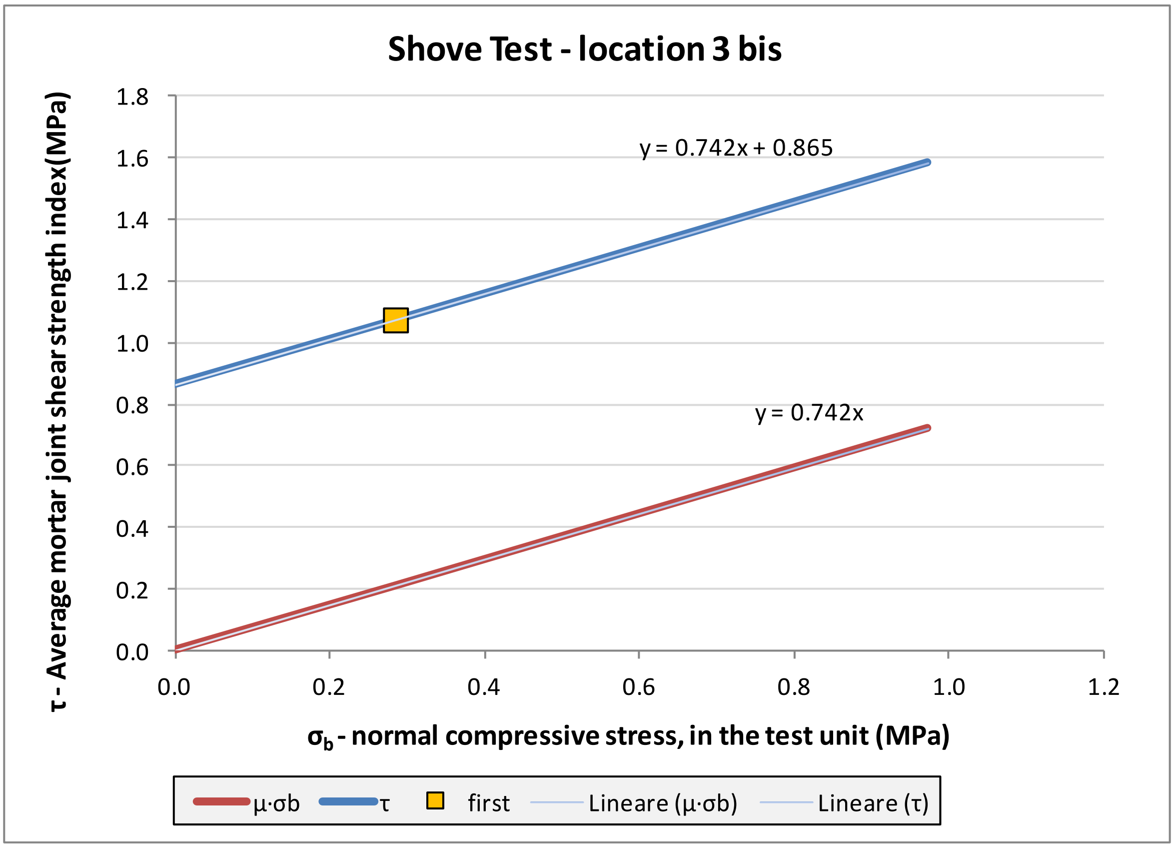


Figure 38 – Shove Test Interpretation of Results, σ_b,τ graph, location 3 bis

5. Ultrasonic Test

The ultrasonic test gives indications on the quality of the bricks and the masonry, by measuring the time travel of the wave pulse in the material, knowing also the distance between the transmitting and receiving device. This test is also useful to determine flaws, cracks and voids in the masonry.
Good results were obtained with a source frequency higher than 50 kHz (ultrasound).
The velocity of propagation of the ultrasonic wave (=distance/transit time) provides a qualitative indication of the quality of the mechanical characteristic of the calcium silicate (*e.g.* wave pass through a single brick) or of the all masonry (*e.g.* wave travels through more than one element).

The measurements can be:

- Direct if the transmitter and the receiver are aligned one opposite to the other, with the sample in between.
- Indirect if the transmitter and receiver are positioned on the same plane (*e.g.* same wall side).

The recorded measures were:

- Distance between the transmitter and the receiver
- Transit time necessary to the wave to travel from the transmitter to the receiver
- Wave velocity = distance / travel time

For each location different type of measures were conducted (see the following Figures):

- Direct measure on a single unit brick
- Indirect measure on a single unit brick
- Indirect measure along one single course of bricks (horizontal direction); the wave travel thorough more than one unit and mortar layers
- Indirect measure in the vertical direction; the wave travel through more than one unit and then some layers of mortar

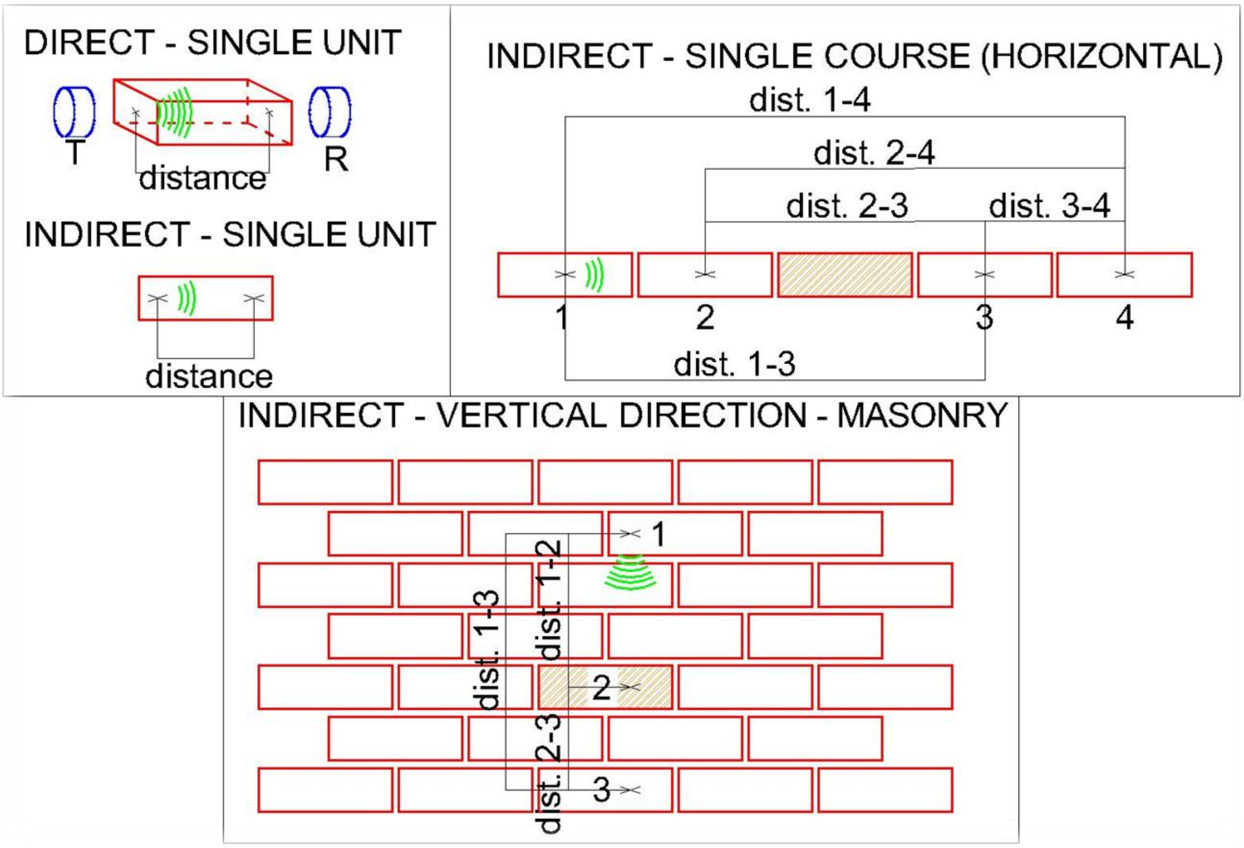


Figure 39 - Ultrasonic Test setup

In the following Table the results of the test are reported for each location.

Table 13 - Results of Ultrasonic Test

SINGLE UNIT

	Single Brick	
	Direct	Indirect
dist.(mm)	212	155
time (µs)	77	56
v (m/s)	2753	2768

LOCATION 1

	Single Brick	
	Direct	Indirect
dist.(mm)	212	155
time (µs)	82.2	59.2
v (m/s)	2579	2618

	Horizontal				
position	2_3 uncrack	2_4	1_3	1_4	2_3 crack
dist.(mm)	450	680	680	900	450
time (µs)	158.7	341.8	309.9	427.2	200.9
v (m/s)	2836	1989	2194	2107	2240

	Vertical			
position	1_2	1_3	2_3	1_2bis
dist.(mm)	275	450	165	275
time (µs)	122.1	180.3	66.7	116.4
v (m/s)	2252	2496	2474	2363

LOCATION 2

	Single Brick	
	Direct	Indirect
dist.(mm)	212	155
time (µs)	83.6	65.3
v (m/s)	2536	2374

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	680	680	900
time (µs)	174.6	281.7	368.5	525.8
v (m/s)	2577	2414	1845	1712

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	275	450	160
time (µs)	107	191.5	65.7
v (m/s)	2570	2350	2435

LOCATION 3

	Single Brick	
	Direct	Indirect
dist.(mm)		155
time (µs)		76.5
v (m/s)		2026

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	680	680	900
time (µs)	296	342.7	380	577.5
v (m/s)	1520	1984	1789	1558

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	250	420	165
time (µs)	110	228	80
v (m/s)	2273	1842	2063

LOCATION 4

	Single Brick	
	Direct	Indirect
dist.(mm)		155
time (µs)		63.8
v (m/s)		2429

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	680	680	900
time (µs)	195	352	338	516
v (m/s)	2308	1932	2012	1744

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	250	420	165
time (µs)	109	180	70
v (m/s)	2294	2333	2357

LOCATION 5

	Single Brick	
	Direct	Indirect
dist.(mm)		155
time (µs)		58.7
v (m/s)		2641

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	680	680	900
time (µs)	176	267	265	516
v (m/s)	2557	2547	2566	1744

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	250	420	165
time (µs)	141	246	68.1
v (m/s)	1773	1707	2423

LOCATION 6

	Single Brick	
	Direct	Indirect
dist.(mm)		155
time (µs)		59.6
v (m/s)		2601

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	680	600	835
time (µs)	225	226	344	502.3
v (m/s)	2000	3009	1744	1662

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	250	420	170
time (µs)	134	178	81
v (m/s)	1866	2360	2099

LOCATION 7

	Single Brick	
	Direct	Indirect
dist.(mm)		155
time (µs)		52.6
v (m/s)		2947

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	450	610	585	750
time (µs)	157	261	250	519.2
v (m/s)	2866	2337	2340	1445

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	250	420	165
time (µs)	93.9	158	57.3
v (m/s)	2662	2658	2880

LOCATION 8 OUT

	Single Brick	
	Direct	Indirect
dist.(mm)	96	155
time (µs)	26.3	43.7
v (m/s)	3650	3547

	Horizontal			
position	2_3	2_4	1_3	1_4
dist.(mm)	455	680	680	910
time (µs)	143	275	214	333
v (m/s)	3182	2473	3178	2733

	Vertical		
position	1_2	1_3	2_3
dist.(mm)	190	315	130
time (µs)	68.5	109	49.8
v (m/s)	2774	2890	2610

6. Rebound Hammer Test

The rebound hammer test gives indications on the quality of the bricks measuring the hardness of the material.
The used instrument is the Schmidt Rebound Hammer N type (impact energy = 2.207 Nm). The test is mainly useful to know if the masonry wall has uniform properties, and then, supported by others test methodologies, correlates the hardness of the unit block with the masonry mechanical properties.

The tests were performed for each of the locations 1, 2, 3, 4, 5, 6, 7 and 8.
For each location four areas were identified: A, B, C and D; in each of these a series of test were performed.

In the following Figure it is possible to see the relative position of the test zone in one of the eight locations.

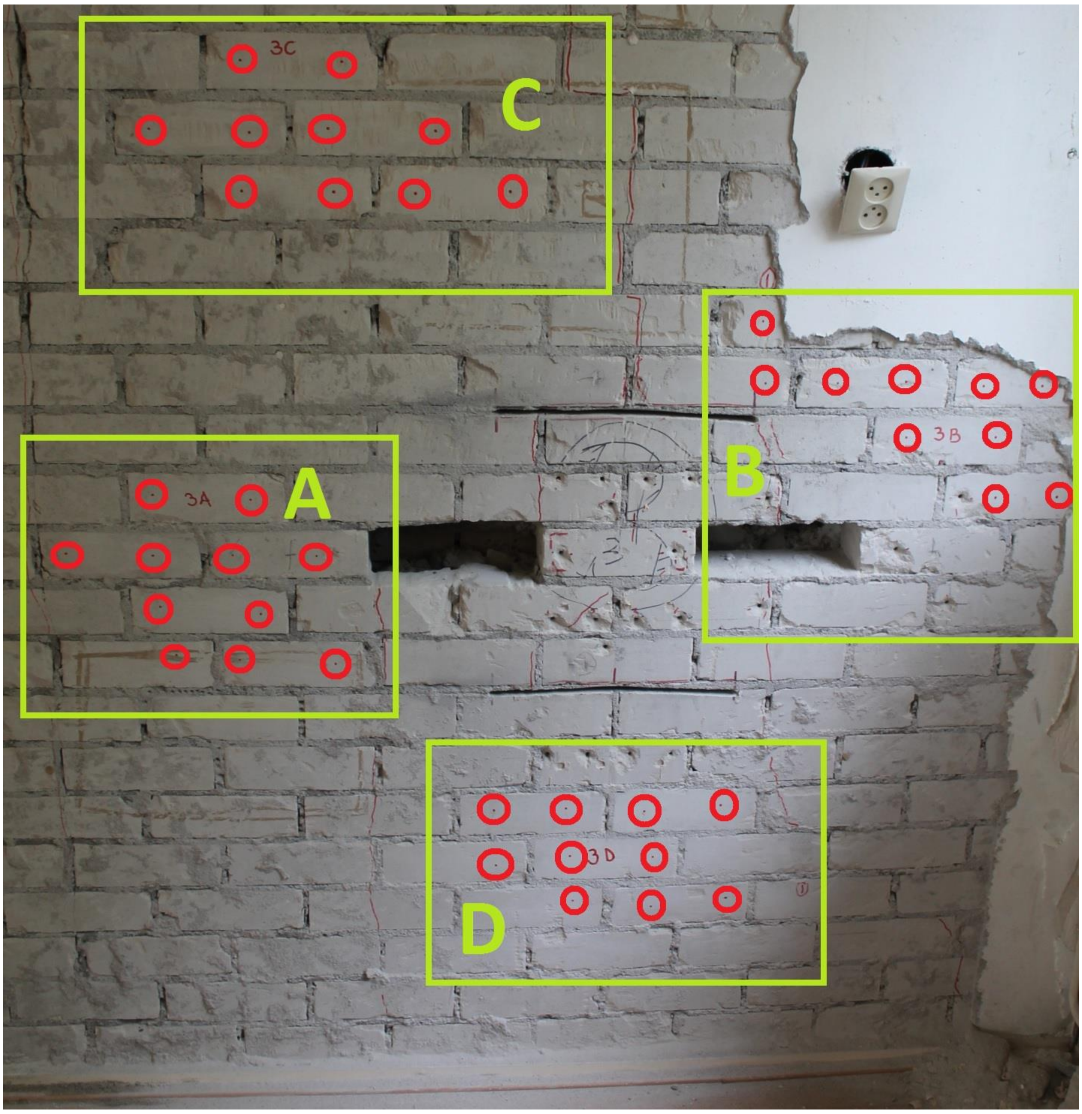


Figure 40 - Rebound Hammer Test setup

In the following Table it is possible to read the results of the test for each location.

Table 14 - Rebound Hamemr Test Results

Location	1	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	30	34	34	34	36	36	34	34	37	37	34.33	35.51
	B	34	36	34	34	32	32	36	36	34	36	34.40	
	C	38	34	34	38	38	35	36	38	38	40	36.90	
	D	38	36	36	34	36	36	36	38	38	36	36.40	
Location	2	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	36	36	32	32	34	34	38	38	38	36	35.40	35.48
	B	42	36	38	32	34	38	34	34	38	32	35.80	
	C	34	36	36	38	32	36	34	36	35	38	35.50	
	D	32	36	36	36	34	34	34	38	38	34	35.20	
Location	3	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	32	34	32	34	34	34	36	34	30	32	33.20	33.83
	B	30	34	32	34	36	34	33	36	32	34	33.50	
	C	32	34	28	36	36	34	34	36	36	36	34.20	
	D	36	32	34	34	34	34	34	34	36	36	34.40	
Location	4	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	34	38	36	34	38	38	32	32	34	34	35.00	35.48
	B	34	34	42	40	38	35	32	34	36	36	36.10	
	C	34	32	38	38	36	34	36	36	34	36	35.40	
	D	36	36	38	34	34	34	34	34	38	36	35.40	
Location	5	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	32	36	34	34	34	34	34	34	36	36	34.40	34.00
	B	34	34	34	34	30	32	36	34	34	34	33.60	
	C												
	D												
Location	6	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	36	38	40	40	38	36	36	34	34	32	36.40	35.90
	B	34	34	36	36	38	34	36	34	38	34	35.40	
	C												
	D												
Location	7	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	32	34	34	32	32	38	38	36	40	40	35.60	37.60
	B	40	38	40	42	40	40	36	36	40	44	39.60	
	C												
	D												
Location	8 - out	Rebound Hammer Number (reading from the instrument)										mean	mean-tot
Area	A	30	32	28	28	32	36	30	30	32	30	30.80	32.00
	B	30	32	28	32	32	32	26	34	32	28	30.60	
	C	34	34	32	34	32	34	32	34	32	34	33.20	
	D	32	34	32	30	32	34	32	34	32	32	32.40	
	E	34	34	32	34	34	32	28	34	34	34	33.00	

7. Penetrometric test on mortar

According to the document "Proposal of in situ testing: Zijilvest 25" (Eucentre, Pavia) penetrometic tests were performed for mechanical characterization of the mortar joints in the masonry building in Zijlvest 25. The used instrument is PNT-G penetrometer by Pizzi ltd company - Florence (Italy), as described in the annexed document (Annex 2).
Horizontal and vertical mortar joints of the inner face of the wall have been tested. Further tests have been executed on adjacent portions of plaster. On the external face only horizontal joints have been tested.

Location of the PNT-G tests
For each zone (1 to 8, no 3bis) were performed more samples (identified by the letters A, B and C. *V* subscription identifies the Vertical joints). The details of the various tests are given in the appropriate datasheet (Annex: 2. PNT-G datasheet).

Results
Statistically characterizing test data and applying the procedure described in the Annex 2 the average compressive strength of the mortar was obtained, in particular:

<u>Horizontal joint:</u>

Average:	$f_m = 8{,}06\ MPa$
Standard deviation:	$\sigma_{f_m} = 2{,}51\ MPa$
C.o.V.	0.31

<u>Vertical joint:</u>

Average:	$f_m = 2{,}56\ MPa$
Standard deviation:	$\sigma_{f_m} = 1{,}31\ MPa$
C.o.V.	0.51

The data concerned vertical joints are rather scattered because of their incomplete filling and/or the penetration of plaster inside them: the composition of the vertical joints is a mix of plaster and mortar.

<u>Plaster:</u>

Average:	$f_m = 1{,}52\ MPa$
Standard deviation:	$\sigma_{f_m} = 0{,}34\ MPa$
C.o.V.	0.22

External wall:

Average: $f_m = 3{,}06\ MPa$

Standard deviation: $\sigma_{f_m} = 1{,}04\ MPa$

C.o.V. 0.34

On the external faces only horizontal joints have been checked: the mortar of the vertical joints appears of the same mix design.

8. Dynamic identification

Dynamic monitoring and subsequent modal identification allows to deepen the knowledge of the dynamic properties of the building unit being considered. The measurement of the dynamic response of the building was performed under ambient vibration and low excitation vibration (such as drop of weights) as per the request of the customer, to avoid disturbance to the neighbouring properties.
Ten triaxial seismometers were placed across the portion of the building to be monitored. The instruments were Lennartz Electronic LE-3D/5s seismometers having the characteristics reported in the following table.

Table 15 – Technical characteristics of the Lennartz Electronic LE-3D/5s

Power Consumption	10 mA @ 12 V DC
Output Voltage	400 V/m/s, precisely adjusted on all components
Damping	0.707 critical (internal damping; independent of datalogger input resistance)
Dimensions	195 mm diameter165 mm height
Weight	6.5 kg
Temperature Range	-15 … +60 °C
Eigenfrequency	0.2 Hz
Upper Corner Frequency	50 Hz
RMS Noise @ 1 Hz	< 1 nm/s
Dynamic Range (typical)	140 dB

Each of the installed instruments is continuously sensing the velocity along three orthogonal directions. Three different configurations were used with the attempt to identify both global and local modes of vibration.
The following Figure 41 shows the acquisition system to which the geophones are connected

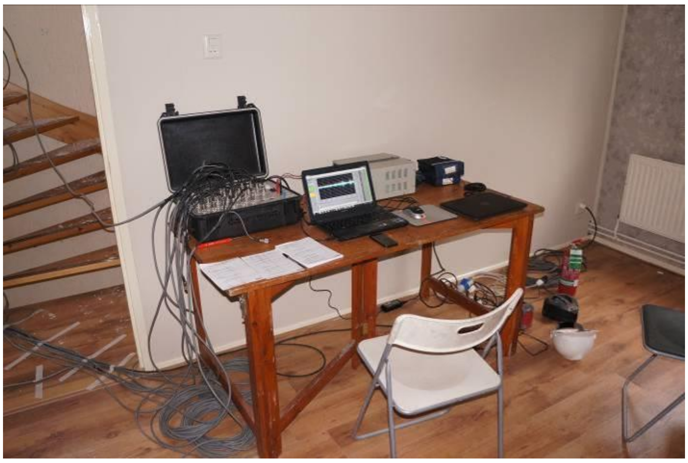


Figure 41. Picture of the acquisition system

The test consists of measuring the velocity time-history (512 Hz sampling frequency) of each geophone for several time windows (approx. 2-3 minutes each). The data were recorded on 23/9/2014 according to the following table.

Table 16. Scheme of dynamic monitoring

Configuration#	Time (start/end)	File#	Type
1	12:08/12:40	from 1_1 to 1_10	Ambient vibration
	12:45/12:50	from 1_11 to 1_12	Impulse X dir. 1st floor
	12:50/12:55	from 1_13 to 1_14	Impulse Y dir. 1st floor
	12:57/13:00	from 1_14 to 1_15	Mass (7 kg) fall (1.5 m) on external ground
2	16:30/16:55	from 2_1 to 2_10	Ambient vibration
	17:00/17:05	from 2_11 to 2_12	Impulse X dir. 1st floor
	17:10/17:15	from 2_13 to 2_14	Impulse Y dir. 1st floor
	17:20/17:25	from 2_14 to 2_15	Mass (7 kg) fall (1.5 m) on external ground
3	18:12/18:36	from 3_1 to 3_10	Ambient vibration
	18:40/18:45	from 3_11 to 3_12	Impulse Y dir. ground floor
	18:45/19:00	from 3_13 to 3_14	Mass (7 kg) fall (1.5 m) on external ground

Ambient vibration tests consisted of recording vibrations due to ambient noise. Low excitation tests were performed applying either one low impulsive load on the walls (shove with a technician's shoulder on a structural wall, separately along the X and Y direction) or a ground vibration induced by a small mass (7 kg) dropped on the ground outside the building from a selected height (1.5 m).

The following figures and tables illustrate the position of all the geophones for each configuration and the corresponding channels. All the geophones were oriented with their N axis (X direction) parallel to the loadbearing walls. Geophone #10 was placed under the ground floor on compacted soil at a depth of –70 cm corresponding approximately to the intrados of the foundation strip of the walls (Figure 42).
Due to the flexibility of the second floor wood diaphragm, in order not to affect the signal recorded with floor vibrations, the geophones placed on this level were positioned on stiff metallic shelves attached to the walls (approx. 40 cm above the floor level). In configuration #3, the 2nd level geophones (#7, #5, #9) were located on the same type of metallic shelves placed on the outer façade at the same height of the inner one (#8).

Table 17.Characteristics of geophones for Configuration #1: location of sensors, number of channels and direction of recordings

Floor level	Instrument #	Location	Channel #	Direction of recording
Underground	10	On compacted soil	27	X
			28	Y
			29	Z
Ground Level	1	On floor	0	X
			1	Y
			2	Z
	2	On floor	3	X
			4	Y
			5	Z
	3	On floor	6	X
			7	Y
			8	Z
Level 1	4	On floor	9	X
			10	Y
			11	Z
	5	On floor	12	X
			13	Y
			14	Z
	6	On floor	15	X
			16	Y
			17	Z
Level 2	7	Supported by metallic shelf on wall	18	X
			19	Y
			20	Z
	8	Supported by metallic shelf on wall	21	X
			22	Y
			23	Z
	9	Supported by metallic shelf on wall	24	X
			25	Y
			26	Z

Table 18.Characteristics of geophones for Configuration #2: location of sensors, number of channels and direction of recordings

Floor level	Instrument #	Location	Channel #	Direction of recording
Underground	10	On compacted soil	27	X
			28	Y
			29	Z
Ground Level	1	On floor	0	X
			1	Y
			2	Z
	2	On floor	3	X
			4	Y
			5	Z
	3	On floor	6	X
			7	Y
			8	Z
Level 1	4	On floor	9	X
			10	Y
			11	Z
	5	On floor	12	X
			13	Y
			14	Z
	6	On floor	15	X
			16	Y
			17	Z
Level 2	7	Supported by metallic shelf on wall	18	X
			19	Y
			20	Z
	8	Supported by metallic shelf on wall	21	X
			22	Y
			23	Z
	9	Supported by metallic shelf on wall	24	X
			25	Y
			26	Z

Table 19.Characteristics of geophones for Configuration #3: location of sensors, number of channels and direction of recordings

Floor level	Instrument #	Location	Channel #	Direction of recording
Underground	10	On compacted soil	27	X
			28	Y
			29	Z
Ground Level	1	On floor	0	X
			1	Y
			2	Z
	2	On floor	3	X
			4	Y
			5	Z
	3	On floor	6	X
			7	Y
			8	Z
Level 1	4	On floor	9	X
			10	Y
			11	Z
	6	On floor	15	X
			16	Y
			17	Z
Level 2	7	Supported by metallic shelf on wall (Outer facade)	18	X
			19	Y
			20	Z
	8	Supported by metallic shelf on wall (internal wall)	21	X
			22	Y
			23	Z
	9	Supported by metallic shelf on wall (Outer facade)	24	X
			25	Y
			26	Z
	5	Supported by metallic shelf on wall (Outer facade)	12	X
			13	Y
			14	Z

Figure 42. Geophone #10, underground on compacted soil, configuration 1, 2, 3

Figure 43. Geophone #1, ground floor, configuration 1, 2, 3

Figure 44. Geophone #2, ground floor, configuration 1

Figure 45. Geophone #3, ground floor, configuration 1

Figure 46. Geophone #4, 1st floor, configuration 1,2,3

Figure 47. Geophone #5, 1st floor, configuration 1

Figure 48. Geophone #6, 1st floor, configuration 1

Figure 49. Geophone #7, 2nd floor, configuration 1,2,3

Figure 50. Geophone #8, 2nd floor, configuration 1

Figure 51. Geophone #9, 2nd floor, configuration 1

Figure 52. Geophone #3, ground floor, configuration 2, 3

Figure 53. Geophone #5, 1^{st} floor, configuration 2

Figure 54. Geophone #6, 1st floor, configuration 2,3

Figure 55. Geophone #7, 2nd floor, configuration 2

Figure 56. Geophone #8, 2nd floor, configuration 2, 3

Figure 57. Geophone #9, 2nd floor, configuration 2

Figure 58. Geophone #5, 2nd floor, configuration 3

Figure 59. Geophone #7, 2nd floor, configuration 3

Figure 60. Geophone #9, 2nd floor, configuration 3

Data reduction and interpretation were performed aiming to the identification of the dynamic characteristic of the structure.
As a general comment, it has to be underlined that the dynamic characterisation of a building may not be an easy task when forced vibration techniques cannot be used. In this specific study case, although the quality of the recorded signals was very good, thanks to the sensitivity of the instrumentation, difficulties arose when semi-automated identification techniques were tried (based on frequency response function, coherence functions or modal assurance criterion), since the recorded signals of every channel were characterized by nonstationarity in the frequency content within a single recording, and variable frequency content from recording to recording. Quite often frequencies that were present in a recording would not be present in another recording. The choice of using semi-automated procedures was therefore discarded and the identification of modes was carried out through direct scrutiny of each recording and non-automated time windowing and subsequent data processing through methods including, but not limited to, Fast Fourier Transform and Transfer Functions, Cross-Spectra and Time-Frequency Analyses.

Configuration # 1

The position of the geophones in the first configuration is shown in Figure 61. The aim of this configuration was to give information on the global behaviour of the structure (or, better, substructure if we consider all the adjacent properties). The geophones were located at each floor level in correspondence of the walls oriented the X direction.

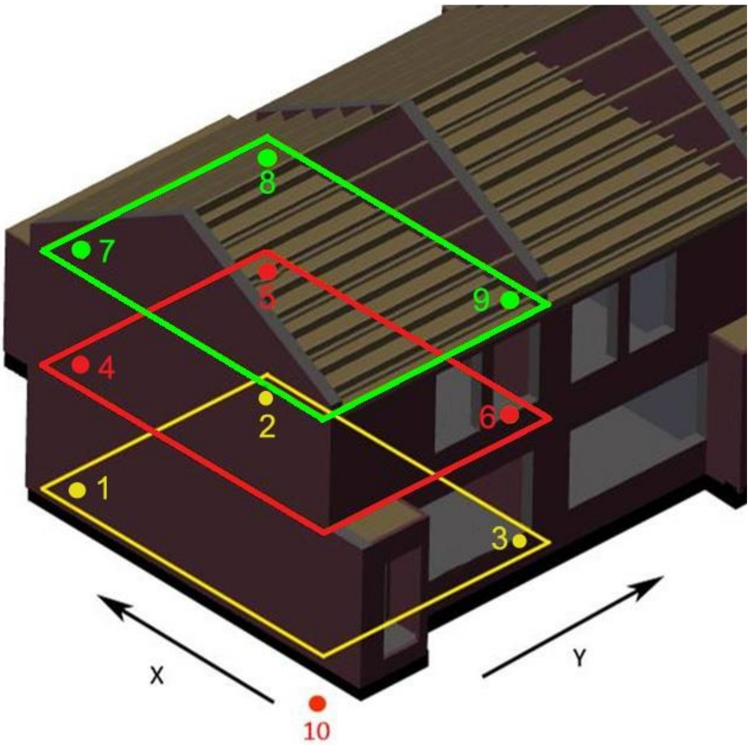


Figure 61. First configuration for the positioning of sensors for dynamic testing

Figure 62. Particular of the positioning of geophone # 9, #1, #10 (from left to right).

After the application of a low-pass filter at 50 Hz, the signals were processed to study the time and frequency response of the system. Considering the response of the geophone below the ground floor level (#10) the presence of low-frequency vibrations (in the range from 3 to 6 Hz) in the ground was detected. In the following Figure 63 a FFT of the X, Y and Z components is shown.

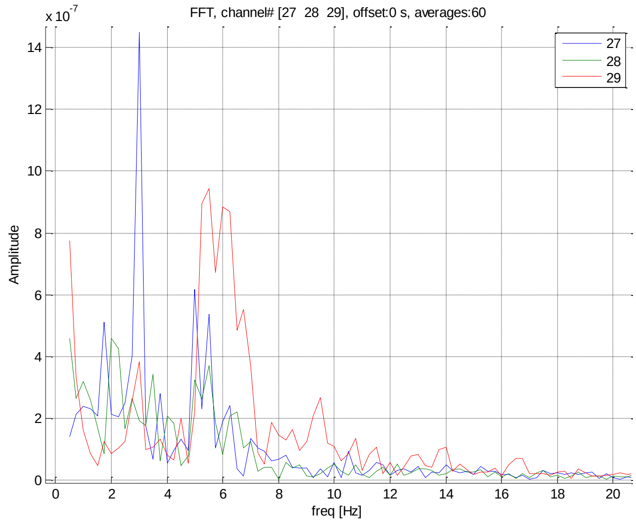


Figure 63. Location of instrument #10 (left) and FFT of acquired signals (right).

The following figures show three cross-spectra of pairs of signals from the first configuration..

In Figure 64, the cross-spectrum of channels #27 and #28 (signals recorded on the soil below the ground floor level, respectively along the X and Y direction at position # 10) is reported showing a clear frequency content at 3Hz and 5.5Hz.

In Figure 65, the cross-spectrum of channels #28 (Y direction at position #10) and #19 (Y direction at position #7, top floor) show that both the frequencies at 3Hz and 5.5Hz are present along the whole height of the building.

In Figure 66, the cross-spectrum of channels #29 (Z direction at position #10) and #20 (Z direction at position #7) signals recorded along the Z direction respectively at the ground level and at the top floor) shows a clear frequency content at 5.5Hz, without the component at 3 Hz, suggesting that the 3 Hz frequency is associated only to horizontal motion.

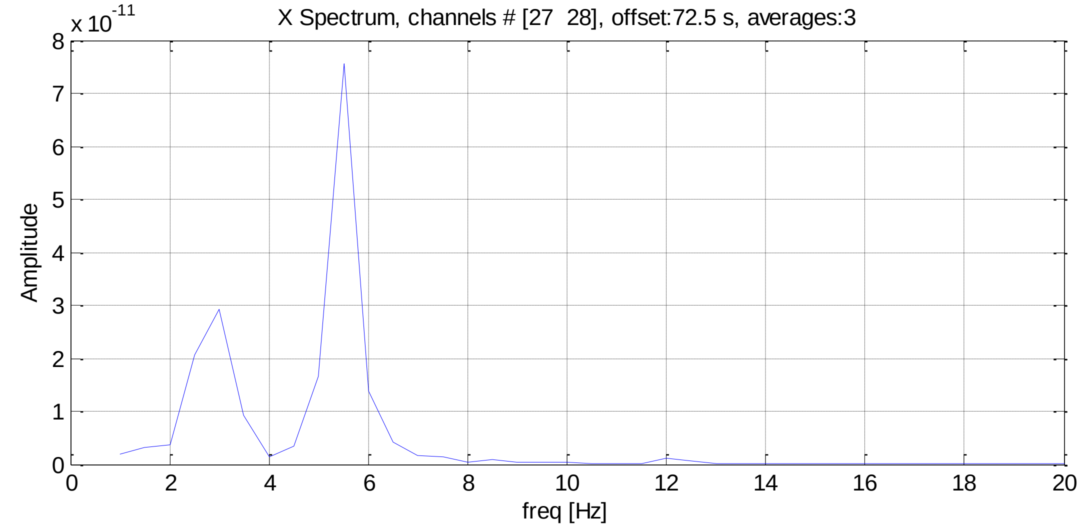


Figure 64. Cross-spectrum of signal acquired by channels #27 and #28 (X and Y directions on the Ground)

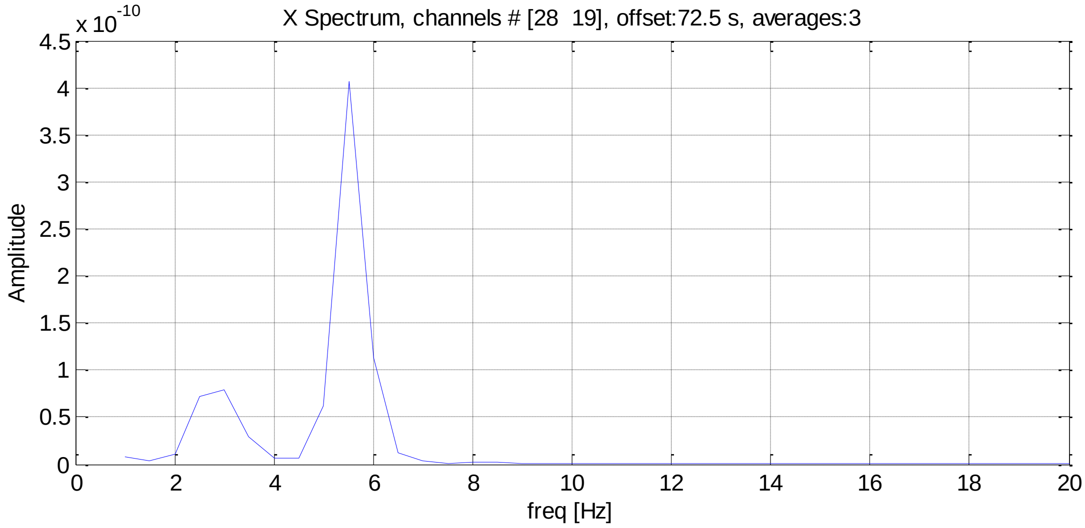


Figure 65. Cross-spectrum of signal acquired by channels #28 and #19 (Y direction on the Ground and at the top floor)

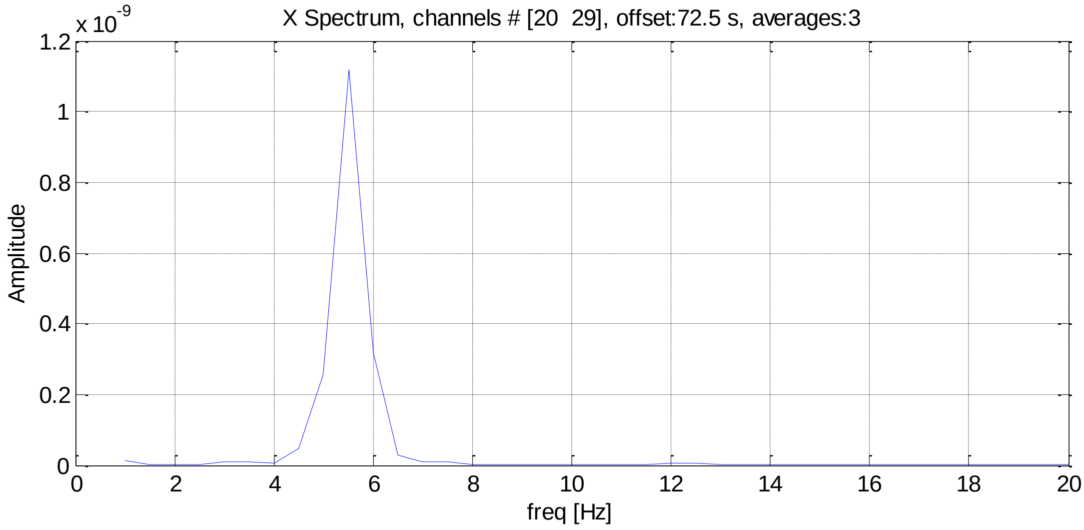


Figure 66. Cross-spectrum of signal acquired by channels #29 and #20 (Z direction on the Ground and at the top floor)

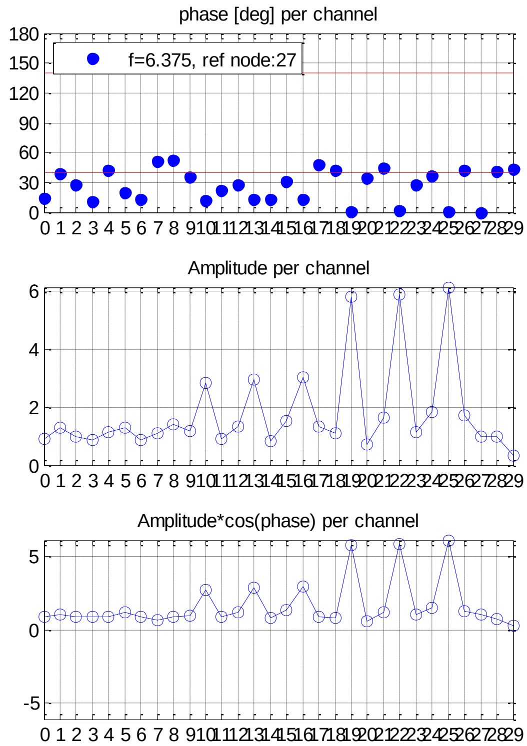
When the amplitude and phase of the vibrations at 3 Hz 5.5 Hz on all channels was examined, for the same time window, apparently such frequencies seemed to be associated to quasi-rigid body motions of the superstructure. It could possibly be inferred that such frequencies are associated to soil-structure interaction response (i.e. the building undergoing rigid body motion due to soil flexibility). However, if different time windows are considered, such frequencies shift, remaining within the 3-6 Hz range, and sometimes are not clearly detectable. With the available information, limited to the acquired dynamic response, it is not possible to conclude whether these low frequencies are associated to soil-structure interaction or simply to vibrations/waves induced by other sources (machinery or other) in the surrounding of the building unit. Possibly, free-field measurements and geological characterization of the site could help in giving a better understanding of the frequencies detected in the 3-6 Hz range.

Vibrations associated to the deformation of the superstructure appeared to be related to higher frequencies. Consistent frequencies and related deformed shapes could be detected for the first mode in the Y direction, at about 6.5 Hz, and the first mode along the X direction at about 12.5 Hz (Table 20).

Table 20 – Summary of the results for the first configuration of sensors

	type	**direction**	**Frequency [Hz]**	**Period [s]**
1	1st transl.	Y	6.37	0.1569
2	1st transl.	X	12.5	0.080

In the following figures, the deformed shapes are reported together with the amplitudes and phases evaluated with respect to the channel #27 recording the signal in X-direction.



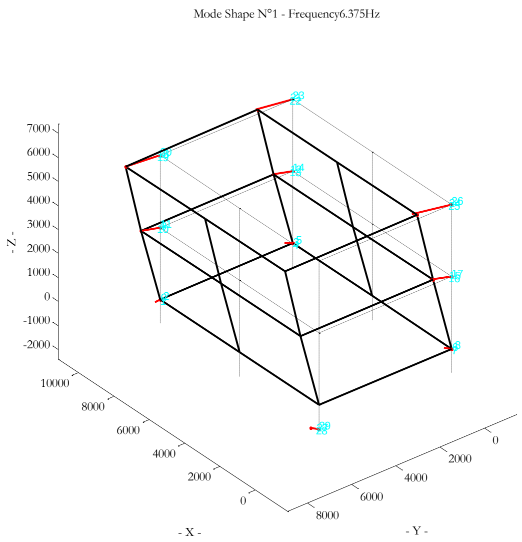
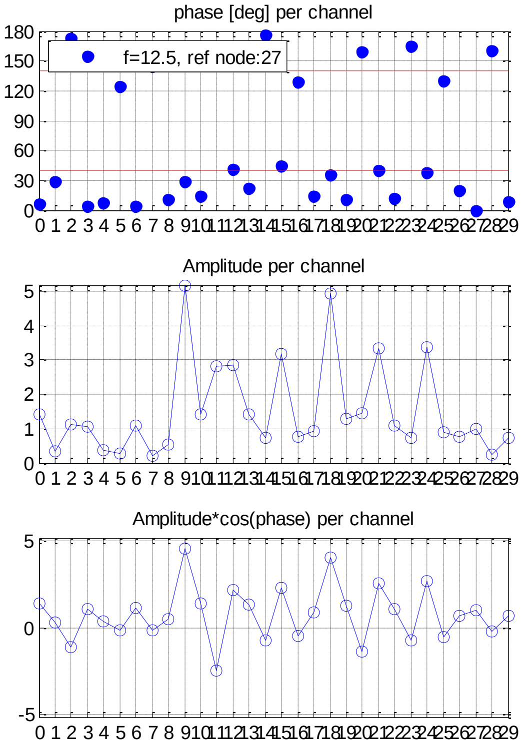


Figure 67.Phase and amplitude of signals at 6.375 Hz (left) and corresponding modal shape (right)



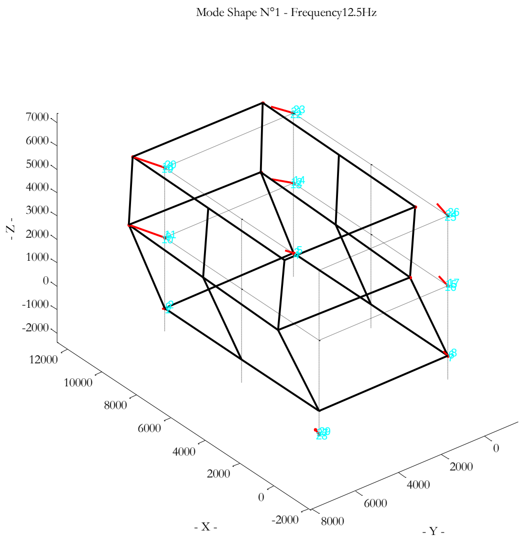


Figure 68. Phase and amplitude of signals at 12.5 Hz (left) and corresponding modal shape (right)

Very similar frequencies, but not identical, could be measured in the recordings made with the 2nd and 3rd configuration of instruments (as it will be reported in the following). The variability in such frequencies could possibly be associated to non-linearities that are present even at very low vibration amplitudes. Sources of such nonlinearities could be the connections between structural elements (floor-to-wall, roof-to-wall, partial connection of internal and external leaves). Elements connected through simple support and friction, and the likely presence of small gaps, can in principle introduce non-linearities, the consequence of which could be a small variation in the natural frequencies.
Most uncertainties in the dynamic identification would be reduced if some controlled forced vibration could have been used, however the approximation in the detection of the first fundamental modes along X and Y seems to be acceptable.

Configuration # 2

The aim of this configuration was to give information on the local out-of-plane behaviour of the inner (load-bearing) wall. Hence, eight geophones were located as shown in Figure 69 in correspondence of the wall to be analyzed.

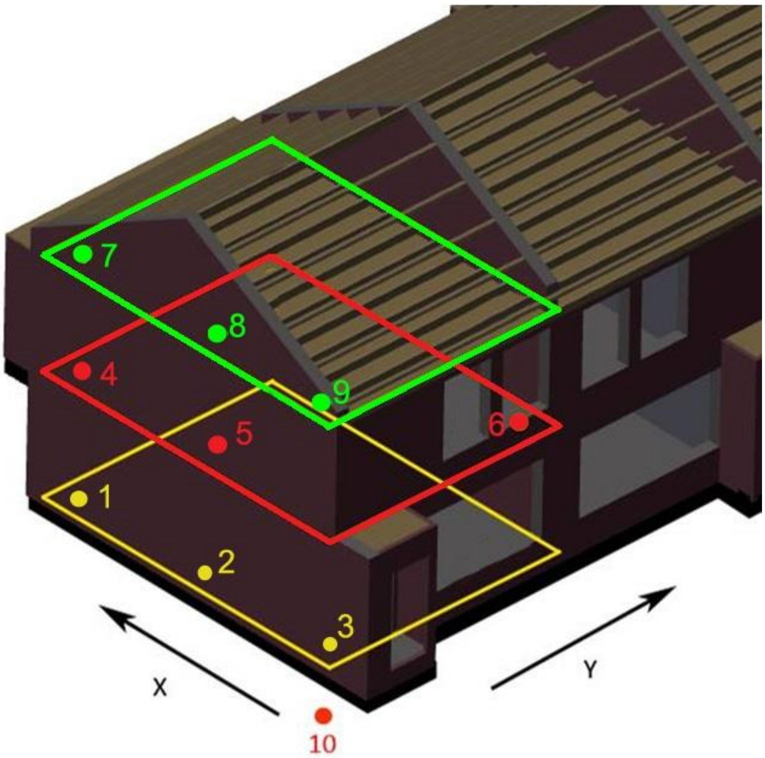


Figure 69. Second configuration for the positioning of sensors for dynamic testing

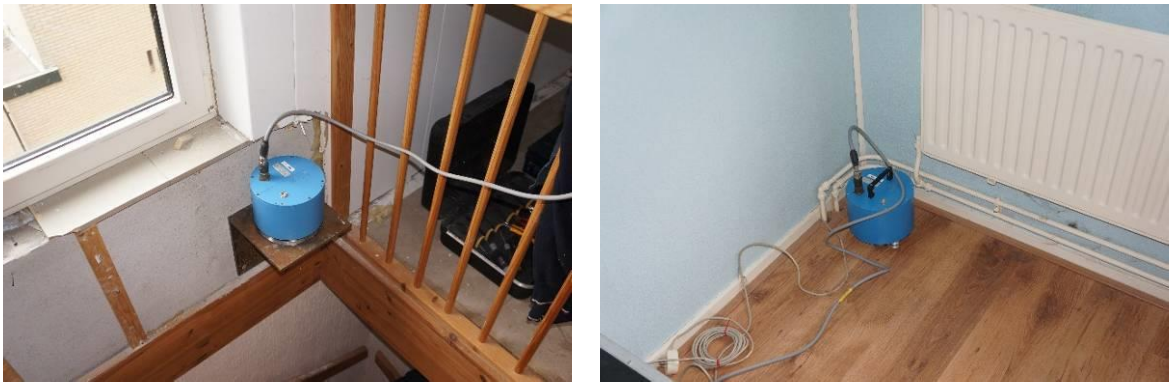


Figure 70. Particular of the positioning of geophone # 8, # 4 (from left to right).

The following figures represent the deformed shape associated to the fundamental period of the wall. Accordingly to what found using data from configuration #1, the fundamental vibration periods are at about 6.6 Hz along the Y direction and at about 12.5 Hz along the X direction.

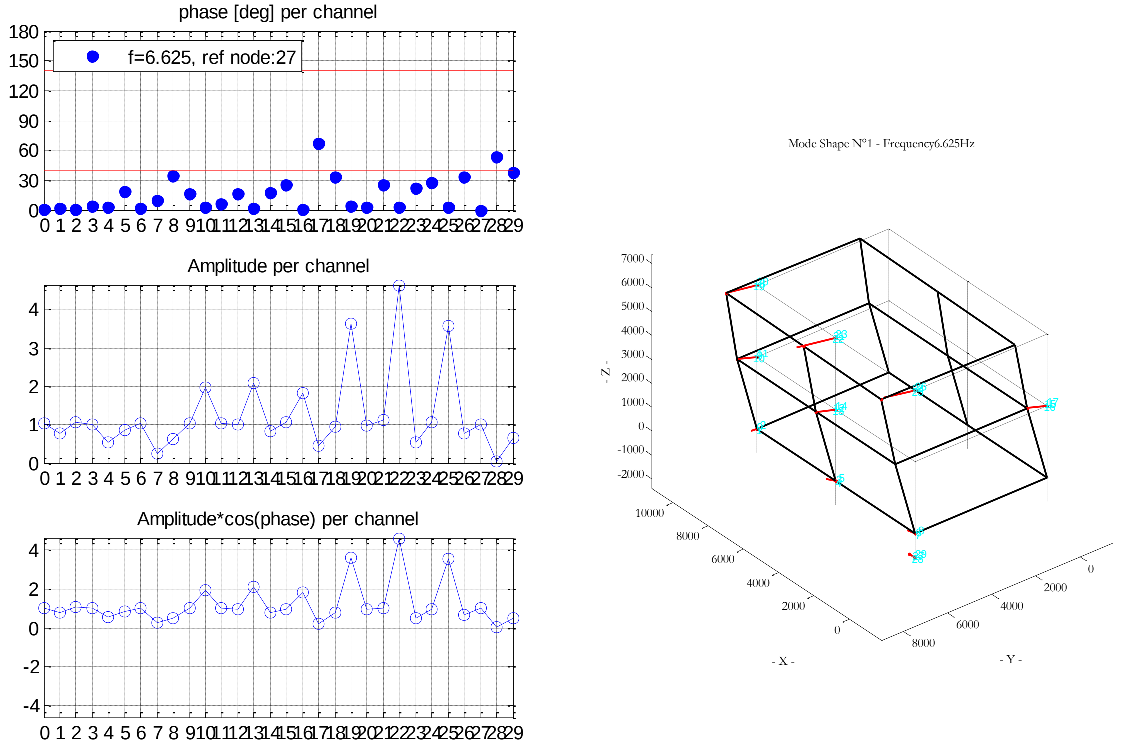


Figure 71.Phase and amplitude of signals at 6.625 Hz (left) and corresponding modal shape (right)

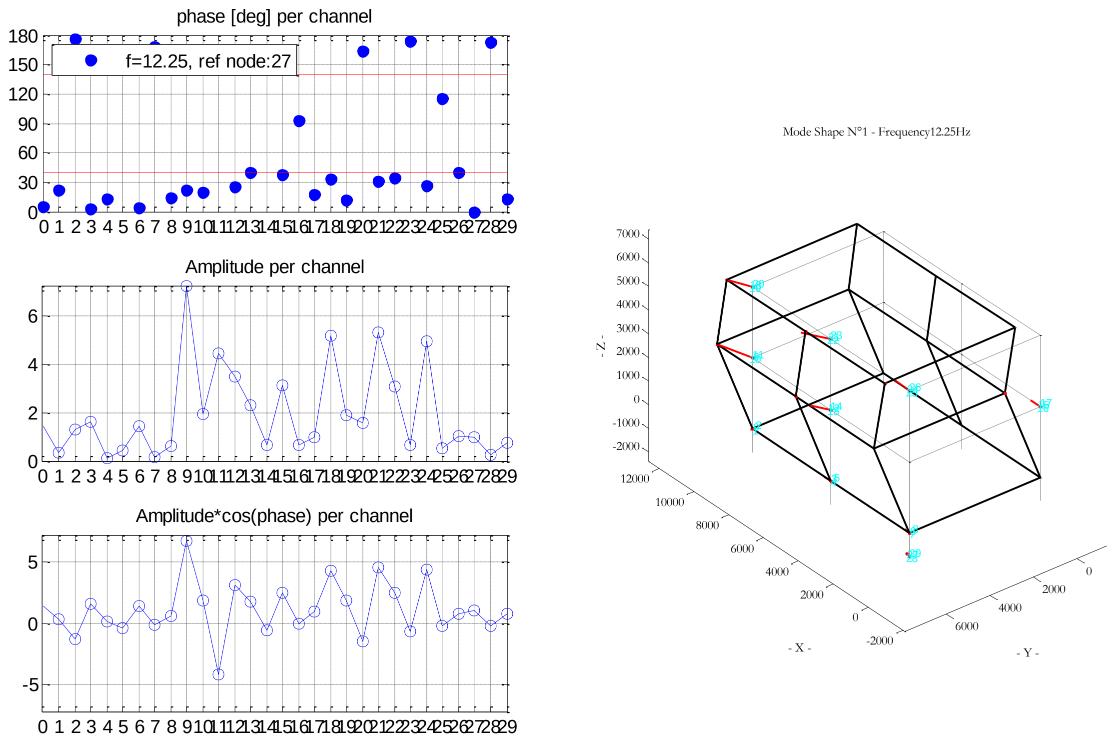


Figure 72.Phase and amplitude of signals at 6.625 Hz (left) and corresponding modal shape (right)

Table 21 gives a summary of the main results.

Table 21 – Summary of the results for the second configuration of sensors

	type	direction	Frequency [Hz]	Period [s]
1	1st transl.	Y	6.62	0.1509
2	1st transl.	X	12.25	0.0816

The second configuration seems to show the presence of an additional local vibration mode: the external walls vibrates out of plane at about 13.7 Hz. The following figures shows phases, amplitudes and the corresponding modal shape, as previously done for the other modes.

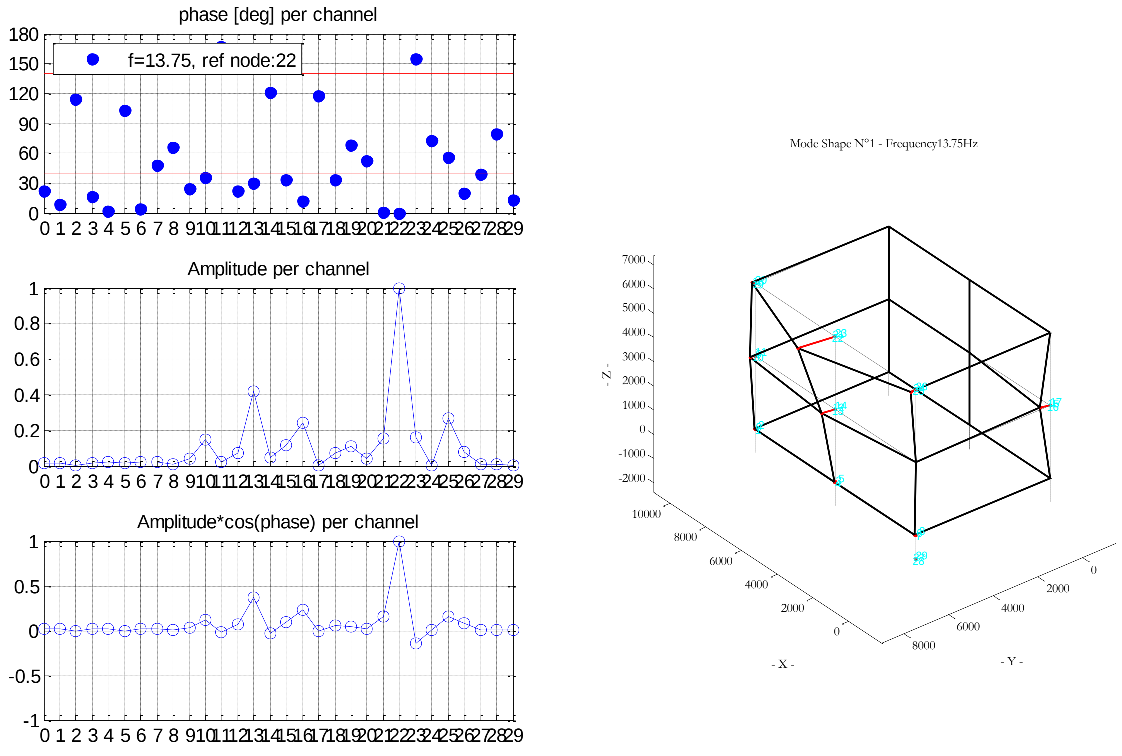


Figure 73.Phase and amplitude of signals at 13.75 Hz (left) and corresponding modal shape (right)

Configuration # 3

As concerns the third configuration, shown in Figure 74 and Figure 75, the location of sensors was designed to give information on the local out-of-plane behaviour of the outer façade and its interaction with the inner (load-bearing) wall.

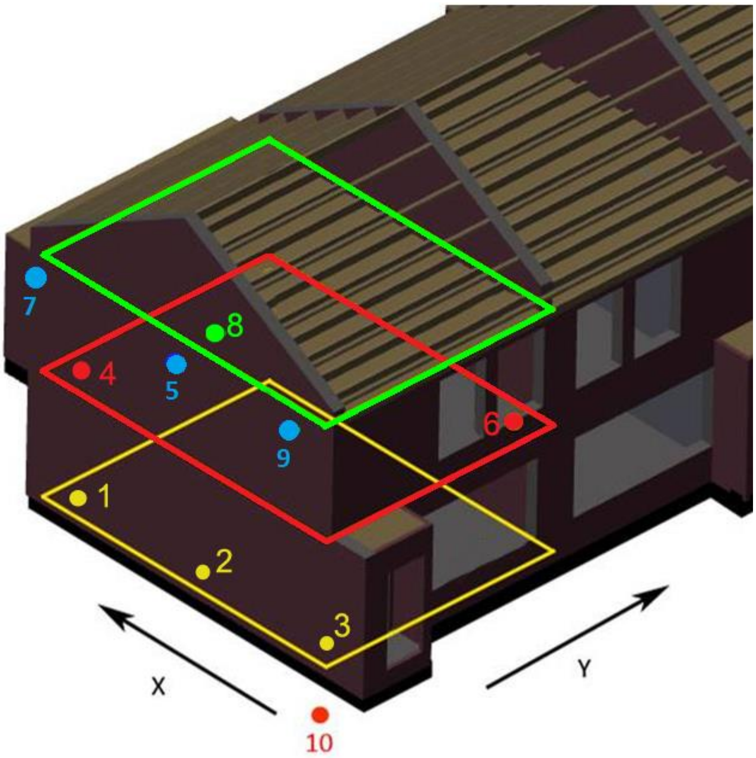
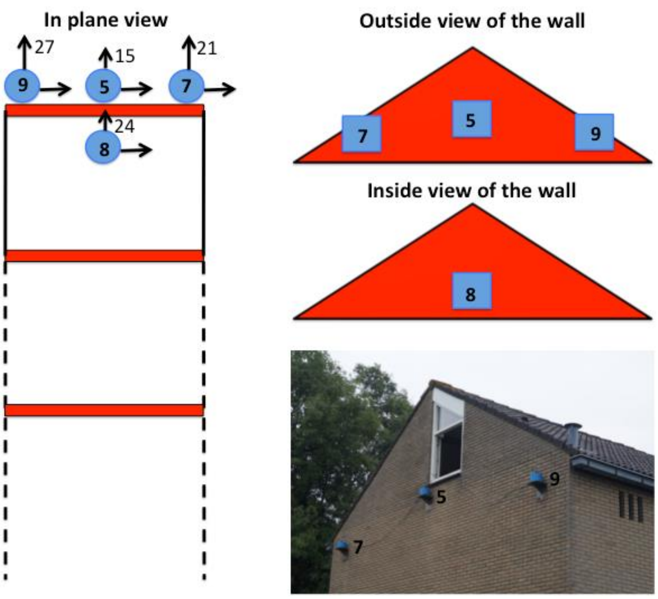


Figure 74. Third configuration for the positioning of sensors for dynamic testing



PLAN FRONT

Figure 75. Position of geophones on the outer facade

Figure 76. Detail of the positioning of geophone # 9.

The data recorded by the instruments in the third configuration seem to show that the inner and outer leaves of the cavity wall vibrate together.
Evidence of this synchronous motion is found when looking at the signals recorded by the instruments #5 and #8, as presented in Figure 77 where one second of recording in the out-of-plane direction is depicted (channel #13 and #22 correspond respectively to the Y direction of instruments #5 and #8).

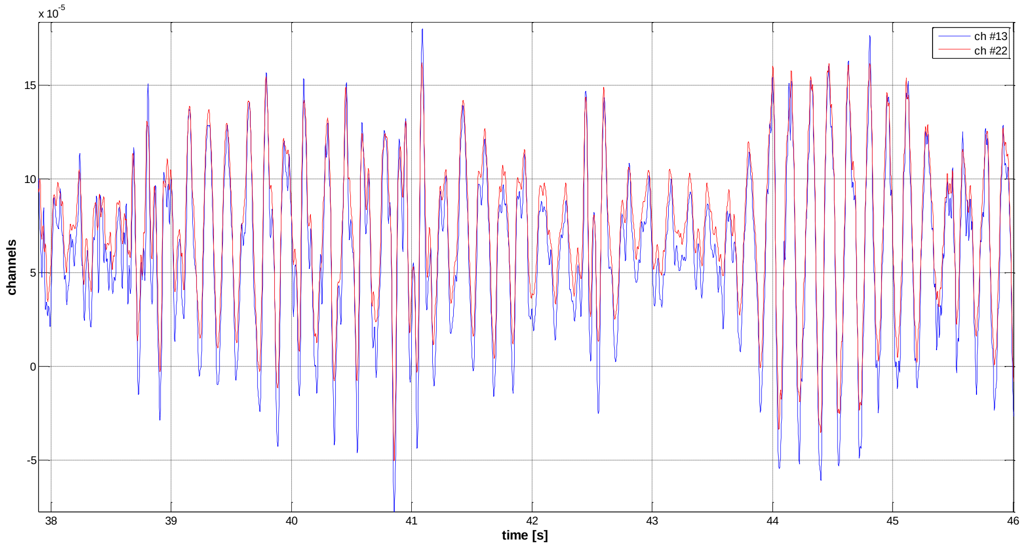


Figure 77. Out-of-plane signals by instruments #5 (ch.#18) and #8 (ch.#22).

Evaluating the cross-spectrum between the two aforementioned signals, it is possible to find the frequency at which they are vibrating and the corresponding relative phase. Figure 78 clearly shows that the dominant frequency is 6.25 Hz and the relative phase is 1.2°, hence they are almost perfectly in phase. Regarding the in-plane vibrations (along the X direction) of the inner and outer leaf, the displacement associated to the first global X mode (at 12.5 Hz) are in phase. The amplitudes of these displacements are approximately equal. This leads to the conclusion that, at least for ambient vibrations, and for the specific points which were monitored, the ties offer a good coupling between the two walls.

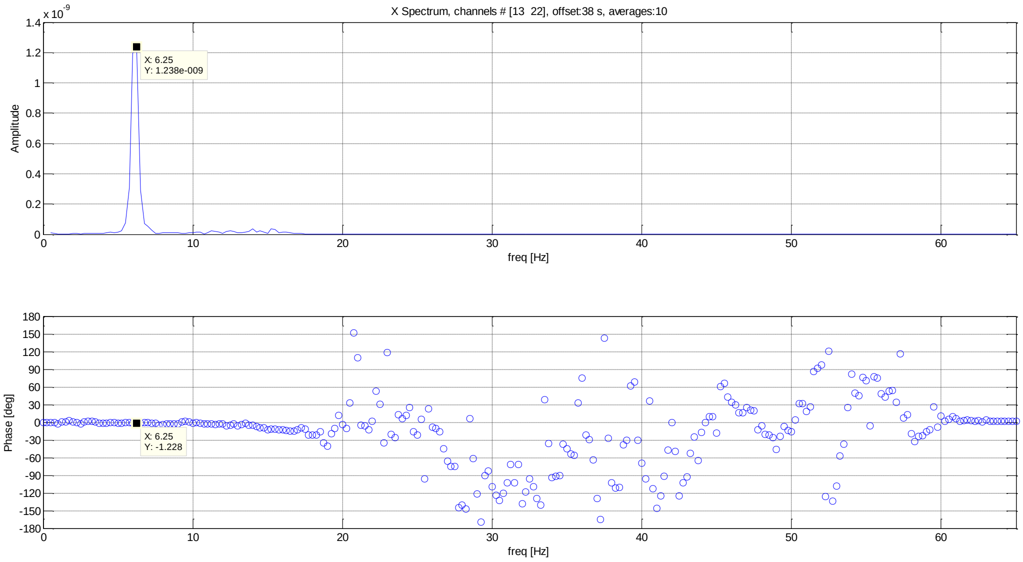


Figure 78.Cross-spectrum amplitude and phase between channels #18 and #22.

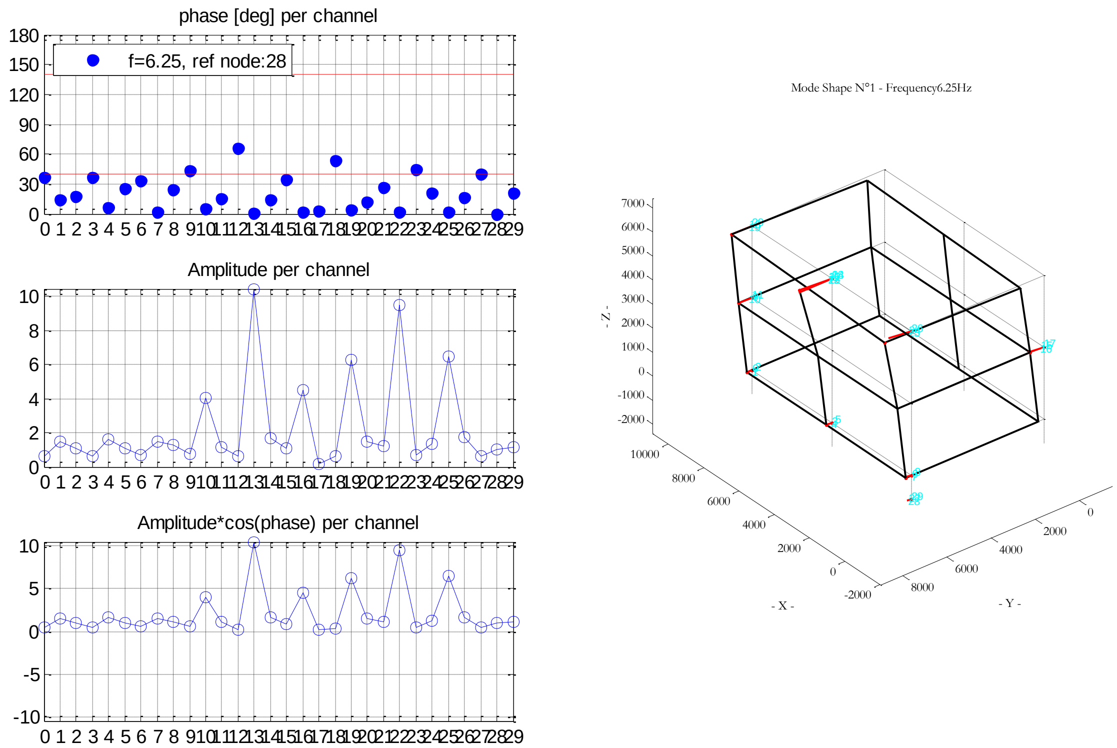


Figure 79.Phase and amplitude of signals at 6.25Hz (left) and corresponding modal shape (right)

Extending the analysis to all the recorded signal, it is possible to evaluate the phase of all the signals relative to channel #28 (which belongs to instrument #10 and records ambient vibrations in the Y direction) that is taken as reference channel and hence construct the modal shapes corresponding to the fundamental modes of the structure. Figure 79 shows the phase angle and amplitude analysis for all the acquired channels (left) and the first modal shape (right).

8. Further results

Thermography, video endoscopy, visual inspection of cavities and pacometer test were used in order to better understand the geometry, the discontinuities and the position of steel ties.
The main results of these activities are:

1) The map should be corrected as shown in next figure. There is no continuity in the inner wall between two adjacent houses.

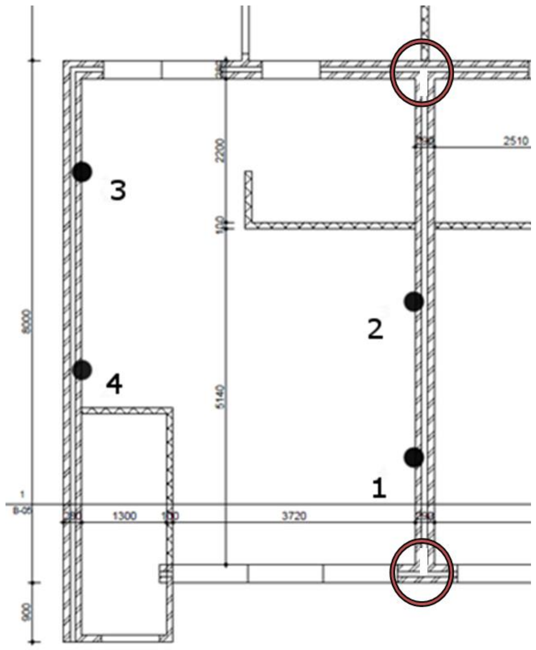


Figure 80- Updated map with discontinuity in internal piers.

2) Some discontinuities were identified with the thermography technique. In particular next Figure show an electric wire that reduces the effective area of masonry (second floor, on the right of point 5)

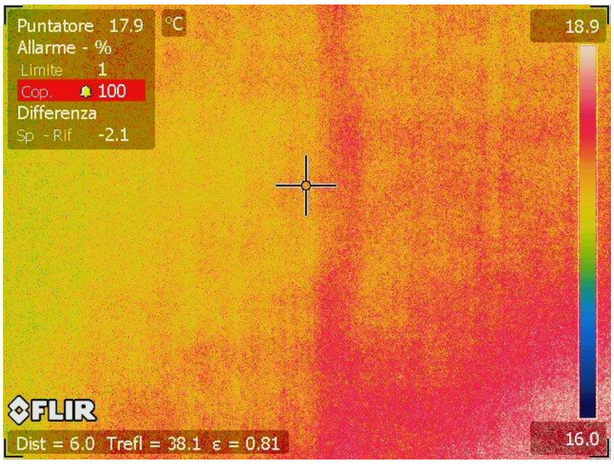


Figure 81 - Discontinuity detected by thermography

3) The steel ties were identified by mean of two pacometers. In particular each tie is a **4 mm diameter smooth steel bar with two hooks**. In the outer walls, the **distance of the ties** is almost constant, **90 cm** (**horizontal and vertical, square mesh**). Between two internal houses the mesh is less regular, with less ties. The next figures show two different ties.

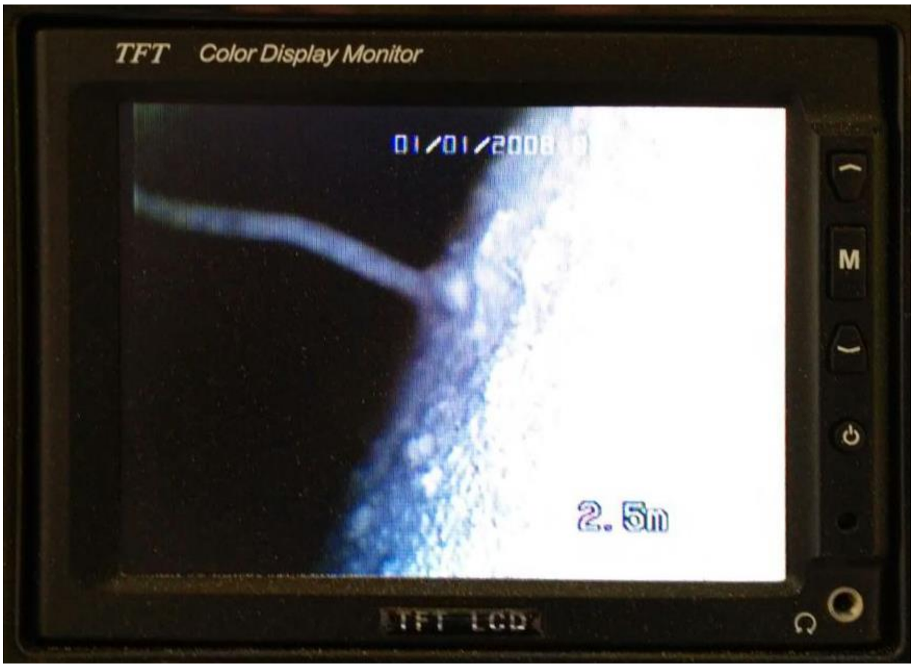


Figure 82 - Steel tie detected by video endoscopy

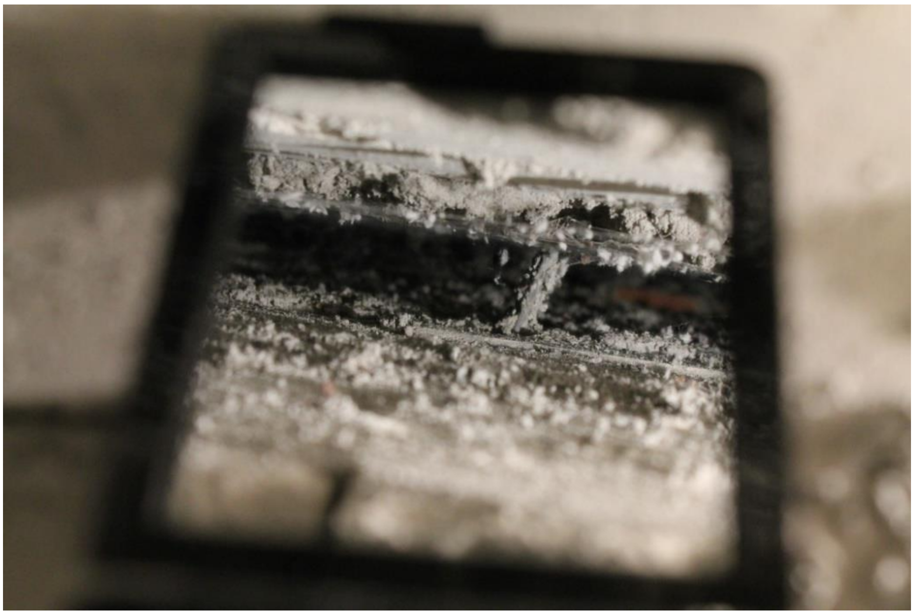


Figure 83 - Steel tie detected by visual inspection (mirror)

9. Summary of the main results

Single Flatjack test Results - Summary

In the following Table and Figure the values of compressive stress in the masonry walls for each location are summarized.

Table 22 - Compressive stress in the masonry walls for each location

	Manual Measurements			LVDT		
	Restoring Pressure			Restoring Pressure		
Location	**bar**	**kPa**	**MPa**	**bar**	**kPa**	**MPa**
1	4.32	431.89	0.43	3.66	365.70	**0.37**
2	2.14	213.96	0.21	2.08	208.45	**0.21**
3	1.17	116.67	0.12	1.15	115.11	**0.12**
4	3.54	354.06	0.35	3.27	326.87	**0.33**

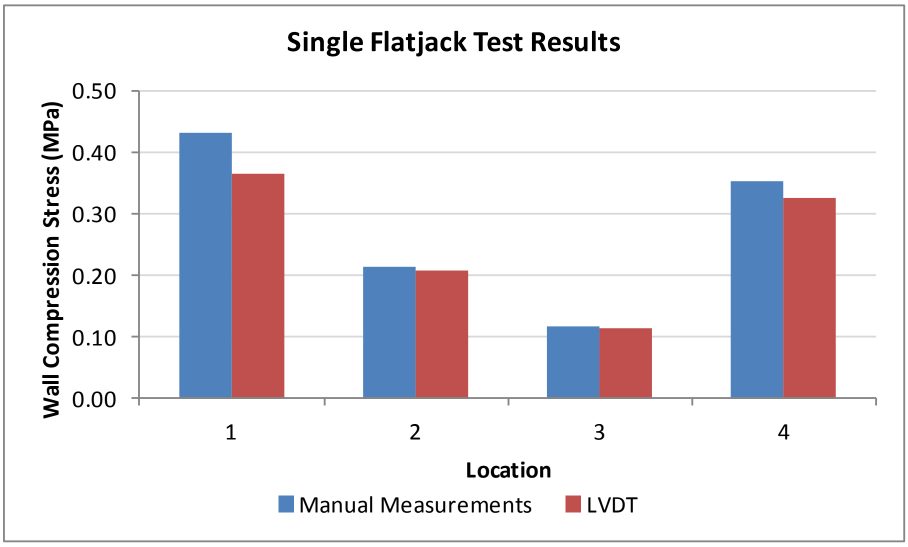


Figure 84 - Compressive stress in the masonry walls for each location

Double Flatjack test Results - Summary

In the following Table and Figure the values of Young Elastic Modulus calculated for each location are resumed.

Table 23 - Young Elastic Modulus for each location

Location	E (MPa)	
1	**3065**	
2	**4992**	
3	**6517**	
4	**7391**	
3 bis	**5951**	
mean	**5583**	MPa
st.dev.	1656	MPa
C.o.V.	0.30	-

Figure 85 - Young Elastic Modulus for each location

Shove test Results - Summary

In the following Table and Figures, the characteristic values obtained from the Shove Test are summarized for each location:

- The joint shear stress index, τ_0, at σ_b =0
- The coefficient μ.

These values are necessary to define the relation: of the joint shear strength, τ, as a function of the normal compressive stress in the test unit brick, σ_b:

$$\tau = \tau_0 + \mu \cdot \sigma_b$$

Table 24 - Friction coefficient, μ, and joint shear stress index, τ_0, at σ_b =0, for each location

Location	**μ** [-]	**τ_0** [MPa]
1	0.538	1.022
2	0.598	1.055
3 bis	0.742	0.865
4	0.751	1.209
mean	**0.657**	**1.038**
st. dev.	**0.106**	**0.141**

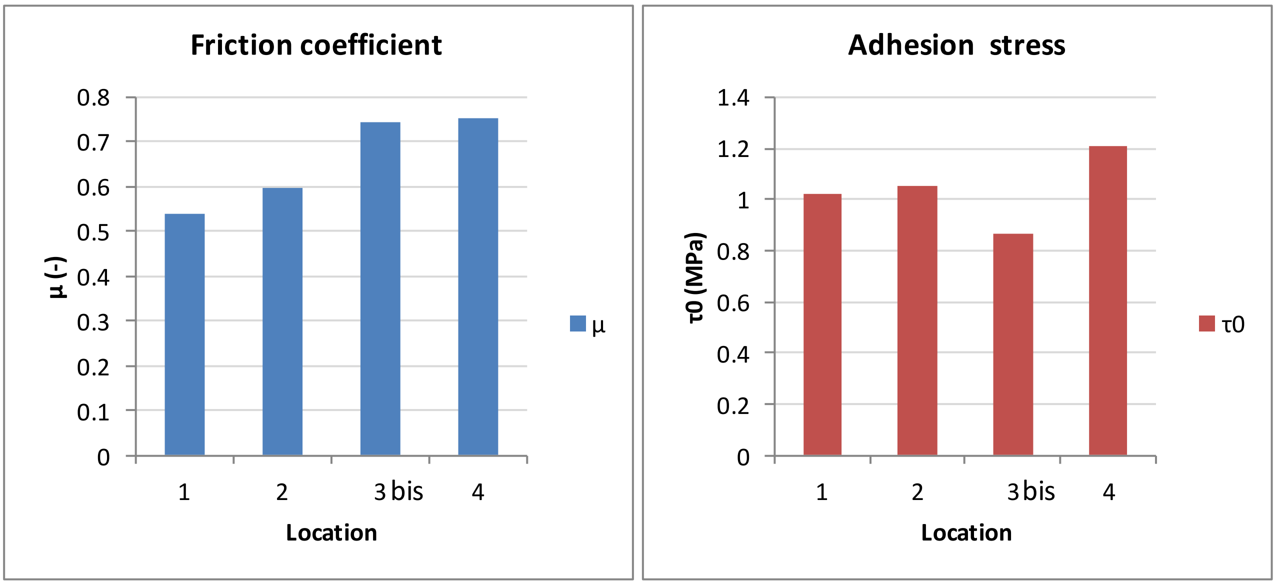


Figure 86 - Friction coefficient, μ, and joint shear stress index, τ_0, at σ_b =0, for each location

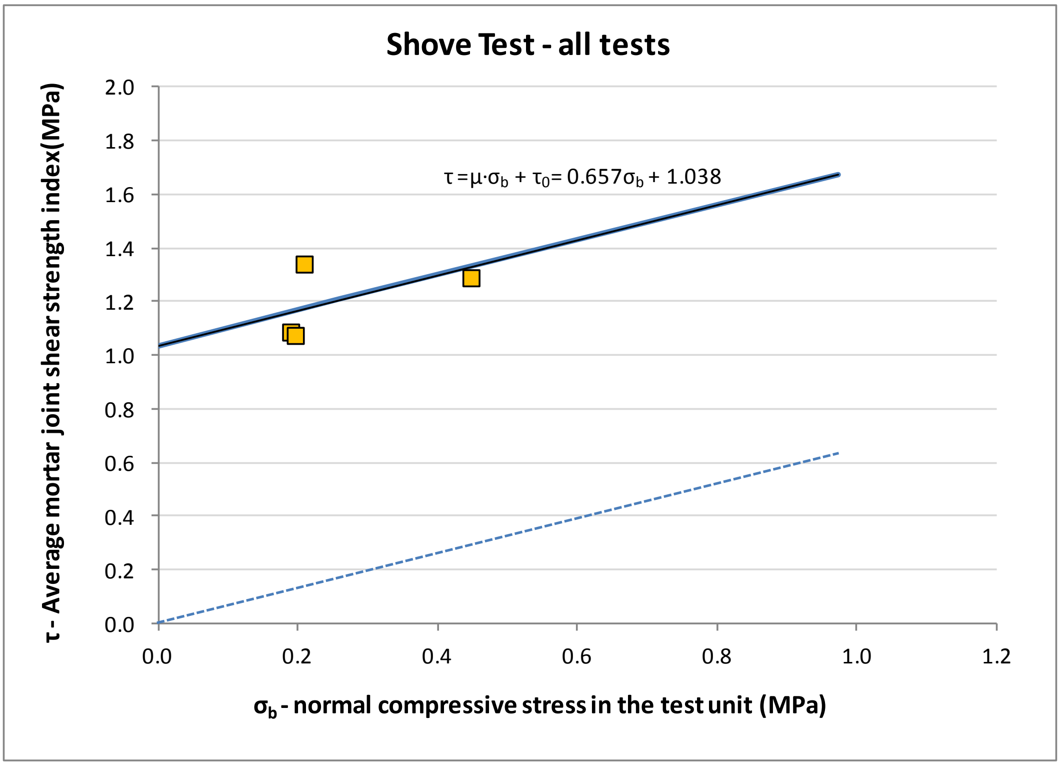


Figure 87 – Shove Test Interpretation of Results (average), σ_b, τ graph

Ultrasonic Test Results - Summary

In the following Table, the values of wave velocity for each location are reported.

Table 25 - Wave velocity for each location

SIGNLE UNIT	V_{direct}	**2753**	**m/s**
	$V_{indirect}$	**2768**	**m/s**

	HORIZONTAL			VERTICAL		
MASONRY Location	V_H m/s	st. dev. m/s	C.o.V. -	V_V m/s	st. dev. m/s	C.o.V. -
1	**2273**	329	0.14	**2396**	112	0.05
2	**2137**	423	0.20	**2452**	111	0.05
3	**1713**	216	0.13	**2059**	215	0.10
4	**1999**	234	0.12	**2328**	32	0.01
5	**2353**	406	0.17	**1968**	396	0.20
6	**2104**	620	0.29	**2108**	247	0.12
7	**2247**	590	0.26	**2733**	127	0.05
8 - out	**2891**	350	0.12	**2758**	140	0.05

Rebound Hammer Results - Summary

In the following Table, the Rebound Hammer Numbers for each location are reported.

Table 26 - Rebound Hammer Numbers

	Location	# rebound	mean	st. dev.	C.o.V.
Inner walls	**1**	35.51	**35.29**	2.44	0.07
	2	35.48			
	3	33.83			
	4	35.48			
	5	34.00			
	6	35.90			
	7	37.60			
	8 - out	32.00	**32.00**	2.14	0.07

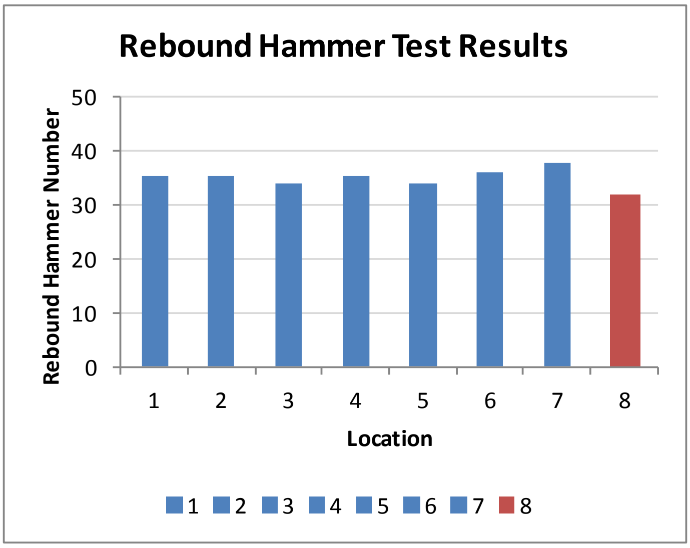


Figure 88 - Rebound Hammer Numbers

Penetrometric test Results - Summary

In the following Table, the results of the Penetrometric Test on the mortar are reported.

Table 27 - Penetrometric Test on Mortar Results

PNT_G	f_m [MPa]	C.o.V.
Horizontal joint	**8.06**	0.31
Vertical joint	2.56	0.51
Plaster	1.52	0.22
External wall	**3.06**	0.34

Dynamic identification – Summary

In the following Table, the main results of the dynamic identification are reported.

Table 28 – Summary of the first and second vibration mode

	type	direction	Frequency [Hz]	Period [s]
1	1st transl.	Y	6.37	0.16
2	1st transl.	X	12.5	0.08

Procedures of in-situ test

This document provides information on the procedures of in-situ tests performed during the experimental campaign for the investigation of the mechanical properties of existing buildings in the Groningen area.

The test, which have been performed by the Italian company P&P, can be divided in two categories:

- Non-destructive tests:
  - Rebound Hammer Test
  - Penetrometric Test on Mortar
  - Ultrasonic Test
- Slightly destructive tests:
  - Single Flat Jack Test
  - Double Flat Jack Test
  - Shove Test

Rebound Hammer Test

The purpose of the test is to provide indications on the quality of the brick measuring the hardness of the material as well as the uniformity of the quality in different locations of the structure.

The instrument used to perform the test is the Schmidt Rebound Hammer N type (impact energy = 2.207 Nm) (Figure 1).

Procedure:

1) Identify the area of interest where the test will be performed and remove the cover material (plaster, paint, etc.) in order to expose the masonry units.
2) Subdivide the area in 4 zones (A, B, C, D) (Figure 1), in each zone select the masonry units to be tested: at least two measure points must be taken per unit and at least 40 measure points must be taken in total. If also a shove test is performed on the same wall, it is possible to conduct a rebound hammer test on the shove testing unit, before the removal of the adjacent bricks. Also it is suggested to choose the rebound hammer testing point all around the shove testing unit.
3) Perform the Rebound Hammer Test (Figure 1) and record the results.

An interpretation of such data on clay bricks is reported in "Brencich, A., and E. Sterpi. *Compressive strength of solid clay brick masonry: calibration of experimental tests and theoretical issues.* Proc., Int. Conf. on Structural Analysis of Historical Constructions, New Delhi, India. New Delhi, India: Macmillan India Ltd., 2006."

Figure 1. Rebound Hammer Test: identification of the 4 zones (left) and performance of the test (right).

Penetrometric Test on Mortar

The purpose of the test is the definition of the quality of the mortar joints as well as the investigation of the homogeneity of the mortar quality in different locations of the structure. The test is based on the idea of measuring the number of strokes necessary to penetrate, for a certain distance, a probe into the mortar.

The test instrument is the Schmidt Rebound Hammer, presented in the previous paragraph, on the top of which a steel probe has been connected (Figure 2).

Procedure:

1) Identify the area of interest where the test will be performed and remove the cover material (plaster, paint, etc.) in order to expose the masonry units;
2) For each test the number of strokes necessary to the probe to penetrate of 1 cm must be recorded, the test must be continued until the probe penetrated of 5 cm into the mortar, if the number of strokes is greater than 25 the test must be stopped.

Figure 2. Penetrometric Test on Mortar.

Ultrasonic Test

The test is aimed to obtain indications on the quality of the masonry material, identification of possible flaws, cracks and voids into the masonry and investigate the homogeneity of the masonry material in different parts of the structure.

The test measures the time of travel of ultrasonic way between a transmitter and a receiver placed in different locations. Knowing the distance of the source and arrival of the ultrasonic signal it is possible to evaluate the speed of propagation which can be also related to mechanical properties like, for example, the elastic moduli.

The measurement can be (Figure 3):

- Direct if the transmitter and the receiver are aligned one opposite to the other, with the sample in between;
- Indirect if the transmitter and receiver are positioned on the same plane (*e.g.* same wall side).

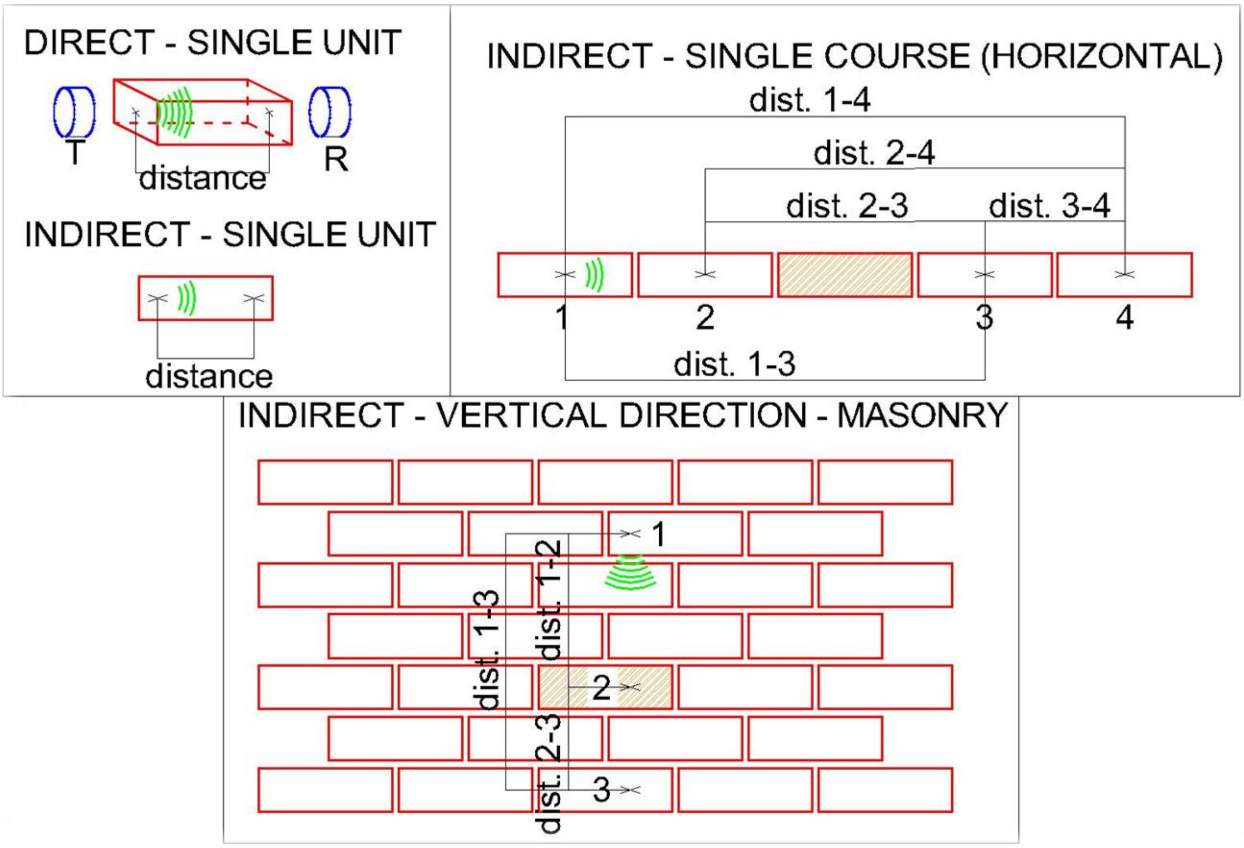


Figure 3. Ultrasonic Test: direct and indirect single unit configuration (top left), indirect horizontal configuration (top right) and indirect vertical configuration (bottom).

Procedure:

1) Identify the area of interest where the test will be performed and remove the cover material (plaster, paint, etc.) in order to expose the masonry units;
2) In each area the following test must be performed:

- Direct Test on Single Brick (Figure 4);
- Indirect Test on Single Brick (Figure 4);
- Indirect Horizontal Test configurations (Figure 5): 1-4, 2-4, 2-3, 3-4, 1-3;
- Indirect Vertical Test configurations (Figure 5): 1-2, 1-3, 2-3;

3) For each test configuration the following quantities must be measured: distance between the transmitter and the receiver and transit time necessary to the wave to travel from the transmitter to the receiver;
4) For each test configuration evaluate the wave velocity as the ratio between distance and travel time.

Figure 4. Ultrasonic Test: direct single unit configuration (left) and indirect single unit configuration (right).

Figure 5. Ultrasonic Test: indirect horizontal configuration (left) and indirect vertical configuration (right).

Single Flat Jack Test

Test Method: ASTM - C1196 (2009)

The purpose of the test is to evaluate, in situ, the average compressive stress in unreinforced solid masonry walls.
As first step a slot is cut in the masonry wall in the area of interest, the compressive stress acting on the wall causes a reduction of the thickness of the slot. A flat jack is inserted in the slot and the pressure in it is increased until the original thickness of the slot is restored. The stress state in the masonry is almost equal to the flat jack pressure, multiplied by coefficients that take into account the characteristics of the jack, the surface of the jack in contact with the masonry, and the area of the opening.

Procedure:

1) Define the location and length of the slot to be formed and mark it with a visible line on the masonry wall.
2) Place at least four pairs of gage points equally spaced and vertically aligned above and below the slot (Figure 7). Each row of gage points (above and below the opening to be formed) must have the same distance from the slot.
   Minimum gage length: 0.3·A (A: length of flat jack, Figure 6),
   Maximum gage length: 0.6·A.
   Position of the first and last gage point: at least at 1/8th of the length A, measured from the centre of the opening to be formed, from the edge of the slot (Figure 7).
   P&P/Eucentre Procedure: Install 4 LVDTs (precision 0.001 mm) connecting each vertical couple of gage points to monitor the vertical relative displacement of the couple of points. In this case a proper self centring screw should be used. This system should be easily tested removing and attaching several times the LVDTs on two undisplaced points. The error should be less than 0.005 mm.
3) Measure the initial distance between each couple of gage points, at the undamaged masonry condition.
4) Realize the slot (Figure 8) and measure again the gage points distance.
5) Clean the opening, now formed, form mortar and brick particles.
6) Repeat again the measure of gage points, after that the opening is prepared for the insertion of the flat jack, in order to obtain the initial deviation from the original gage distance.
7) Insert the flat jack into the slot (Figure 9).
8) Connect hydraulic hoses and fill the calibrated flat jack, until the pressure begins to increase, with the hydraulic oil.
9) Increase the pressure till about 50% of the estimated maximum flat jack pressure (which corresponds to the estimated compressive stress in the wall, with standard calculations). Reduce the pressure in the flat jack to zero. This is done in order to "seat" better the flat jack in the opening (Figure 10).

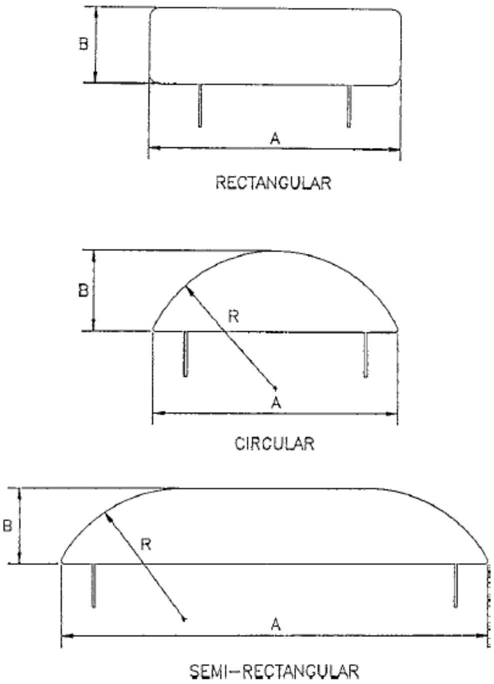


Figure 6. Possible configurations of flat jack, plan-view.

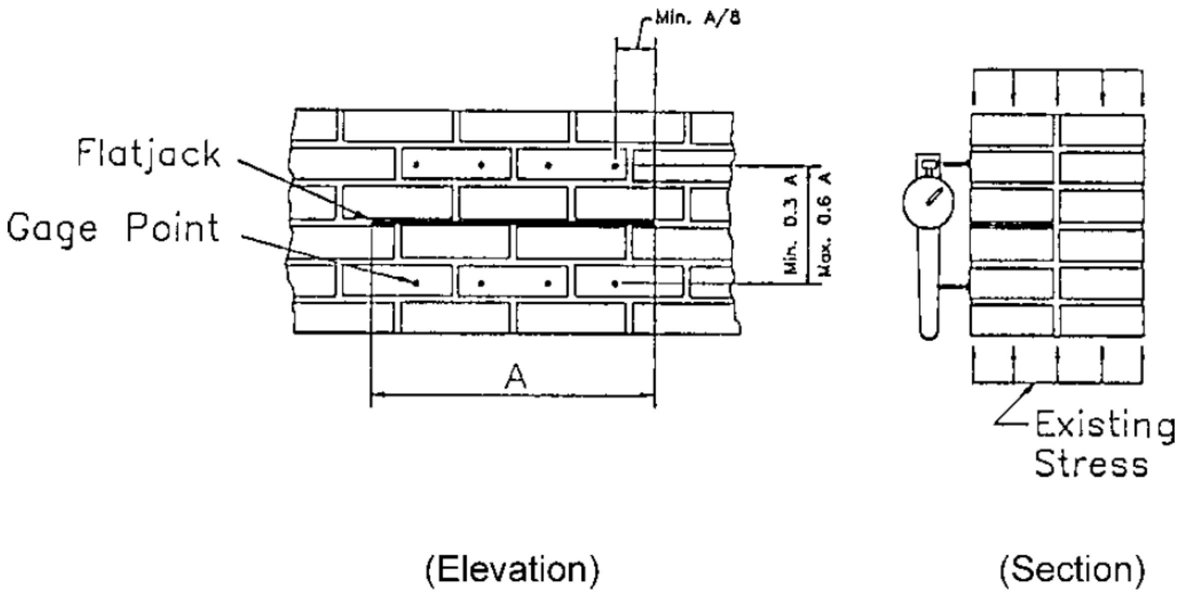


Figure 7. Single Flat jack Test: setup for in situ stress measurement.

10) Increase the flat jack pressure to 25%, 50%, 75% of the estimated maximum pressure, and hold the pressure constant at each level. Measure and record the distance between each pair of gage points, at each load increment. It is necessary to repeat each measure three times. It is recommended to conduct the test as soon as possible, after the realization of the opening.
P&P/Eucentre procedure: since LVDTs are used, the pressure/displacement curve could be controlled real time with no pauses.
11) Continue to increase the flat jack pressure until the original gages distances are restored.

Allowable average deviation from original gage length: greater than 0.013 mm, or 1/20th of the maximum initial deviation.
Allowable single deviation from original gage length: the maximum between: 0.025 mm and 1/10th of the maximum deviation.
Record the final pressure in the flat jack.

12) Reduce the pressure in the flat jack to zero.
13) Is recommended to repeat points 10) e 11) in order to verify the final pressure in the flat jack.
14) Disconnect the hoses and remove the flat jack. The slot must be filled with mortar, or other material, with similar characteristics of the original one.

Calculations:

The average compressive stress in the masonry, f_m, is given by:

$$f_m = K_m K_a p$$

where:

K_m dimensionless constant which reflects the geometrical and stiffness properties of the flat jack.
K_a ratio of measured area of flat jack to the average measured area of the slot.
p flat jack pressure required to restore the gage points to the distance initially measured between them within the tolerance allowed (MPa).

By calibration of the flat jack not only the value of K_m and the nominal area of the device can be obtained but also a more refined relationship between f_m and p. The relationship, where the contribution of K_a has not been taken into account yet, has the following form:

$$y = ax + b$$

where:

y force applied to flat jack [Force].
x theoretical load (=p) [Force].
a,b coefficient of the linear interpolation, provided by flat jack manufacturer.

Therefore, the compressive stress in the masonry f_m can be expressed in the following form:

$$f_m = K_a \left(a \cdot p + \frac{b}{A_{flj}} \right)$$

with:

A_{flj} area of the flat jack.

Figure 8. Single Flat Jack Test: cutting of the slot in the masonry wall.

Figure 9. Single Flat Jack Test: the flat jack is inserted in the slot.

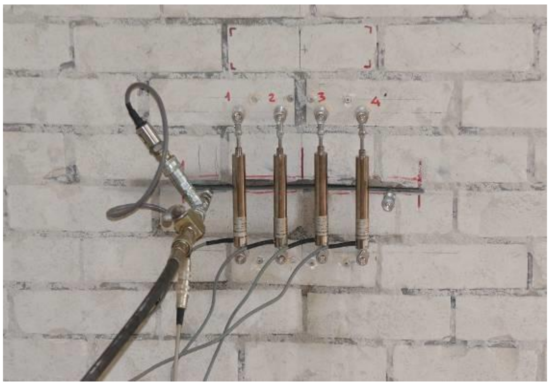


Figure 10. Single Flat Jack Test: increasing of the pressure in the flat jack to restore the slot thickness.

Double Flat Jack Test

<u>Test Method: ASTM C1197 (2009)</u>

The purpose of the test is to evaluate, in situ, the deformation properties of an unreinforced masonry wall.
In order to measure the deformation characteristics of the wall two flat jacks are placed into parallel slots, one above the other, which are cut in two different bedjoints (Figure 11). The pressure in the flat jacks is gradually increased inducing a compressive stress state in the portion of the wall between the two devices. By increasing gradually the pressure in the flat jacks and recording the deformation of the masonry between the slots it is possible to obtain the load-deformation (i.e. stress-strain) curve. Besides the wall deformability the test can also allow, in some cases, to measure also the maximum compressive strength.

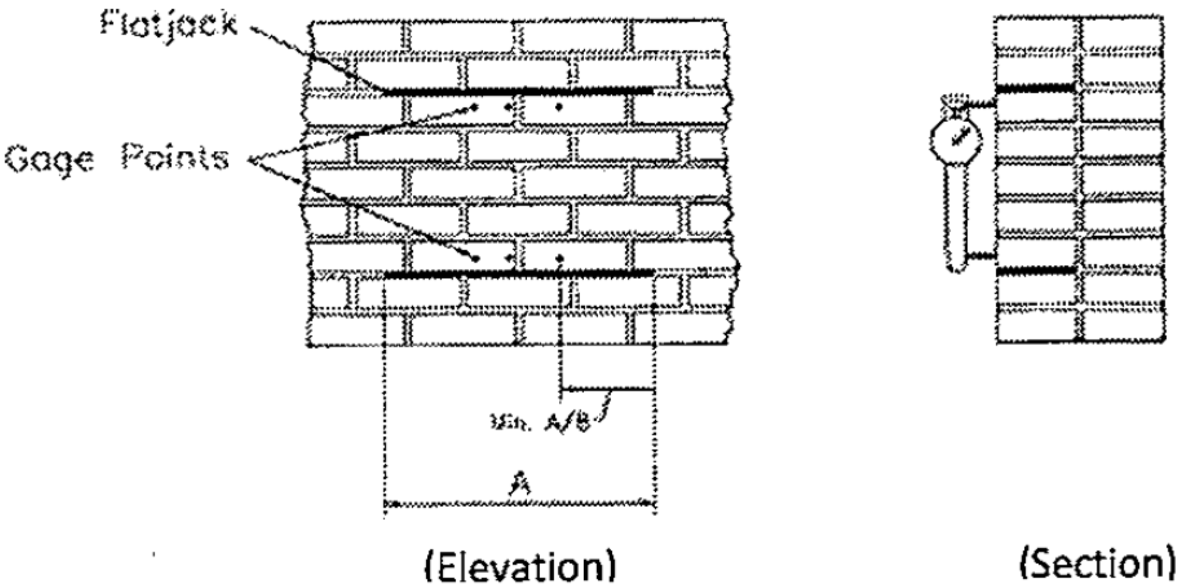


Figure 11. Double Flat Jack Test: test setup.

<u>Procedure:</u>

1) Define the location and length of the slots to cut and mark them with a visible line on the masonry wall.
   <u>IMPORTANT:</u> the test location must be carefully chosen; due to the high pressure in the flat jacks damages to the upper portion and lifting of the wall can be experienced. It is suggested, when possible, to perform the test on walls subjected to high axial load and select locations far from openings (door, windows, etc.)
2) Realize the slots (Figure 12) and measure their dimensions and positions. The two openings for the flat jacks shall be separated by at least five courses of masonry, but not more than 1,5 times the flat jack length.
3) Clean the openings of mortar and bricks particles.
4) Place at least three LVDT in vertical direction, equally spaced (Figure 11). The LVDT must be perpendicular respect to the slots and have at least a length of 20 cm. The LVDT must be attached to the masonry units and not to the mortar joints.

P&P/EUCENTRE procedure: 4 vertical LVDTs were installed.

Position of the first and last measurement points: at least at 1/8th of the length A from the edge of the slot (A: length of flat jack, Figure 6).

Install one LVDT parallel to the slots (horizontal), equally spaced from the two openings.

Important: record the reference length of each LVDT, every time that they are installed.

5) Insert the flat jacks into the slots.
6) Connect the hydraulic hoses and fill the calibrated flat jacks until the pressure starts increasing.
7) Increase the pressure up to almost the 50% of the estimated maximum flat jack pressure (which corresponds to the estimated compressive stress in the wall obtained with standard calculations). Reduce the pressure in the flat jacks to zero. This is done in order to "seat" better the flat jacks in the openings.
8) Record the initial measurements.
9) Increase the pressure in the flat jacks slowly. Record the LVDT measurements at each increment of pressure. Monitor the flat jack *pressure - masonry deformation* ratio, p, at each step. In case the failure of the masonry between the flat jacks wants to be avoided the test should stop when the ratio p begins decreasing significantly.
10) If the masonry is old or has elements with low resistance, or the cement content in the mortar is zero, the flat jacks can load the masonry till its failure, reaching the maximum strength. However, this may cause some damage in the masonry zones adjacent to the flat jacks.
11) Once the last measurement have been collected reduce the pressure in the flat jacks to zero.
12) Disconnect the hoses and remove the flat jacks. The slots must be filled with mortar, or other material, with similar characteristics of the original one.

Calculations:

The average compressive stress in the masonry, f_m, is given by:

$$f_m = K_m K_a p$$

where:

K_m dimensionless constant which reflects the geometrical and stiffness properties of the flat jack.
K_a ratio of measured area of flat jack to the average measured area of the slot.
p flat jack pressure required to restore the gage points to the distance initially measured between them within the tolerance allowed (MPa).

By calibration of the flat jack not only the value of K_m and the nominal area of the device can be obtained but also a more refined relationship between f_m and p. The relationship, where the contribution of K_a has not been taken into account yet, has the following form:

$$y = ax + b$$

where:

y force applied to flat jack [Force].
x theoretical load (=p) [Force].
a,b coefficient of the linear interpolation, provided by flat jack manufacturer.

Therefore, the compressive stress in the masonry f_m can be expressed in the following form:

$$f_m = K_a \left(a \cdot p + \frac{b}{A_{flj}} \right)$$

with:

A_{flj} area of the flat jack.

The tangent modulus at any stress interval is given by:

$$E_t = \frac{\delta f_m}{\delta \varepsilon_m}$$

where:

δf_m increment of stress.
$\delta \varepsilon_m$ increment of strain.

The chord modulus at any point, i, is given by:

$$E_{si} = \frac{f_{mi}}{\varepsilon_{mi}}$$

where:

f_{mi} stress at point i.
ε_{mi} strain at point i.

Figure 12. Double Flat Jack Test: cutting of the two slots (left) and test execution (right).

Shove Test

Test Method: ASTM C1531 (2009)

The purpose of the test is to evaluate, in situ, the shear resistance of the two horizontal bedjoints framing a single masonry unit. Considering all test methods proposed in literature the more appropriate appears to be the one that utilizes two flat jacks controlling the vertical load, and one small cylindrical jack that control the horizontal load.
Since the test requires the use of two flat jacks to control the vertical stress state in the masonry generally the shove test is performed after the single and double flat jack tests have already been performed.

Procedure:

1) Define the location and length of the slots to cut and mark them with a visible line on the masonry wall.
2) Execute the first horizontal cut in the masonry
3) Perform the Single Flat Jack Test.
4) Execute the second slot, at a distance of five course of bricks
5) Perform the Double Flat Jack Test.
6) Remove the two masonry units adjacent (left and right) to the masonry unit that must be tested (Figure 13 and Figure 14). The bedjoints on the side of the masonry unit to be tested must be "cleaned" to avoid that the presence of exceedances of mortar prevent the masonry unit from sliding.

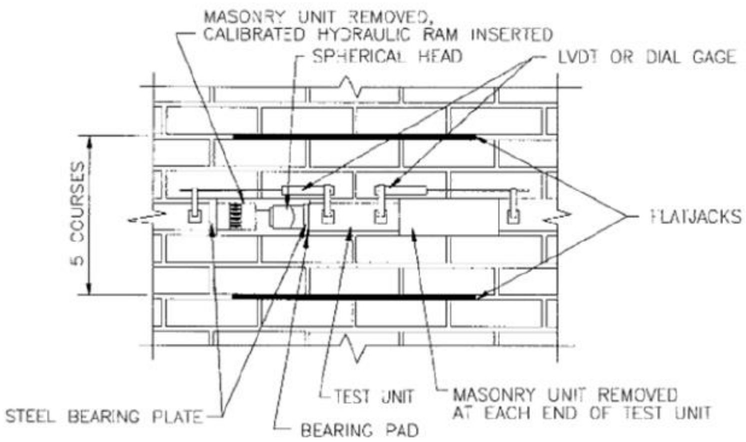


Figure 13. Shove Test: test setup.

7) Remove the vertical joint of mortar on the back side of the masonry unit to be tested with a drill with an L shape head. This is not necessary for single leaf masonry.
8) Instrument the test unit with horizontal and vertical LVDTs (Figure 14).
9) Insert the horizontal load jack (spherical seat) and the metallic plates for the diffusion of the load (minimum thickness: 3 mm), and neoprene.

<u>Evaluation of cohesion:</u>

10) Set the flat jacks pressure at a value lower that 0.07 MPa and maintain it constant.
11) Increase the pressure in the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction). This provides information on the first rupture of the interface, taking in account of cohesion as well as friction effects.

<u>Evaluation of friction coefficient:</u>

12) Increase the pressure in the flat jacks till the one recorded in the Single Flat Jack Test, and maintain it constant.
13) Increase the pressure in the horizontal loading jack and record the measurements until the post peak phase. If the brick displaced more than 0.5 cm, reverse the horizontal load.
14) Repeat the procedure at least 4 times increasing the pressure in the flat jacks up to a level considered still in the "elastic phase" judging from the results of the Double Flat Jack Test.
15) Release the pressure in the horizontal loading jack and remove it, eventually dismount also the instrumentation.
16) The test could be performed also in the opposite direction, just by transferring the loading jack in the opposite opening.

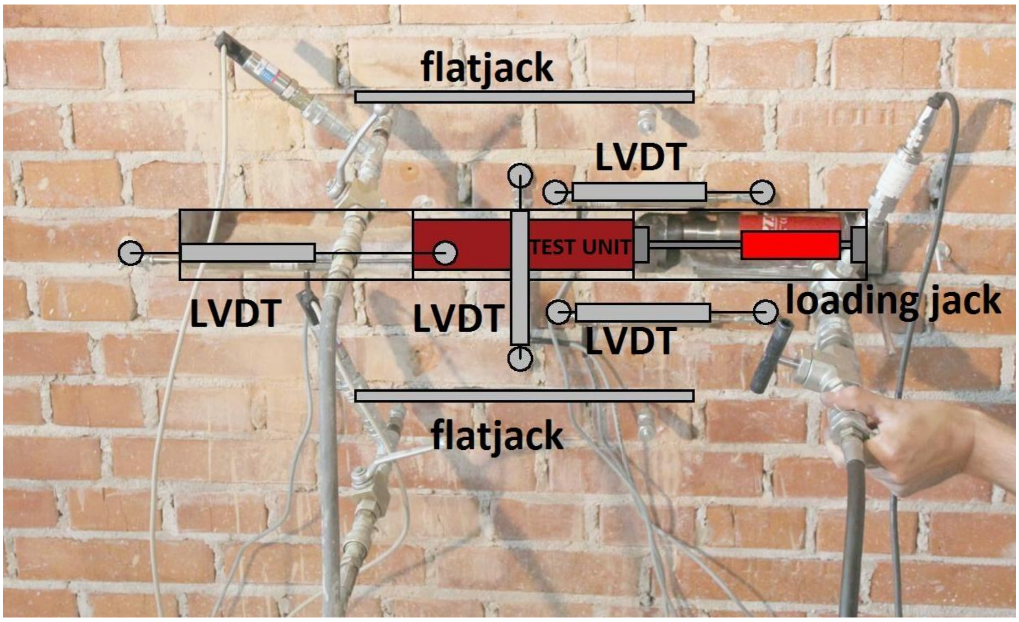


Figure 14. Shove Test: test apparatus.

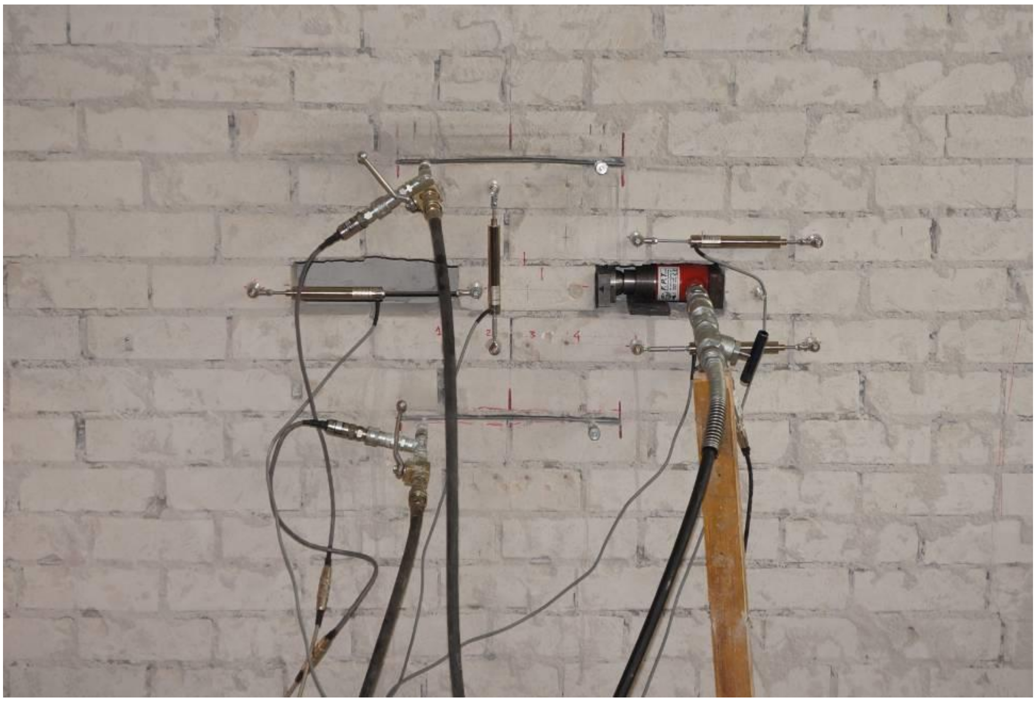


Figure 15. Shove Test: execution of the test.

Calculations:

Particular attention must be paid for normal stress distribution. Recent analyses have shown that the distribution of normal stress on the test unit is not uniform, with the average stress equivalent to "j" times the applied flat jack stress:

$$\sigma_n = j \cdot \sigma_{flj}$$

where:

σ_n normal stress on test unit.
j modification factor, the value of j depends on the test configuration.
σ_{flj} stress applied by the flat jacks to the masonry, computed in accordance with the Test Method ASTM C1197 (Double Flat Jack Test).

Data interpretation:

The physical quantities measured during the Shove Test at each level of vertical compression ensured by the two flat jacks are essentially:

- LVDTs record (displacements)
- Flat jacks and hoses records (pressures)

The horizontal displacement, D, of the test unit at each horizontal load step, i, is:

$$D_i^{hor} = d_i^{hor} - d_{i-1}^{hor}$$

where:

d_i^{hor} horizontal LVDT record at the load increment i.

For a specific vertical pressure it is possible to plot the horizontal load, of the loading jack, versus the horizontal displacement of the test unit, and to identify the point corresponding to the elastic limit.

For each level of vertical compression the following quantities are recorded:

τ_i average mortar joint shear strength index.
σ_v normal compressive stress.

The average mortar joint shear strength index τ_i, for each level of normal compressive stress σ_v, is defined as:

$$\tau_i = \frac{P_{hi}}{A_j}$$

where:

P_{hi} maximum horizontal force resisted by the test unit at the i^{th} level of normal compressive stress (elastic limit in the current case).
A_j gross area of upper and lower bed joints in the case of solid unit masonry, or net mortar bedded area for the case of hollow-core masonry units.

Plotting the compressive stress/joint shear couples of point in the same graph and performing a linear regression it is possible to define the relationship expressing the joint shear strength index, τ, as a function of the compressive stress, σ_v:

$$\tau = \tau_0 + \mu(\sigma_v)$$

where:

μ Friction coefficient, which is the slope of the line, relative to the linear regression of all the points (τ_i,σ_{vi}), except the first. This because the first point is the one representative of the adhesion stress contribution to shear resistance, at low level of compressive stress.
τ_0 Cohesion, which is the joint shear stress index at zero nominal compressive stress, i.e. the Y-intercept of the linear function with coefficient of friction equal to μ passing through the point (τ_1,σ_{v1})

New procedure for the Shove Test interpretation:

The above mentioned Shove Test procedure is the “standard” procedure that should be followed according to existing guidelines (ASTM C 1531-09). Hovewer recent tests performed in Eucentre laboratory during the preliminary experimental campaign demonstrated that the above mentioned standars have certain limitations concerning the calculation of exact values of cohesion and friction coefficient, in particular in double leaf and heavily loaded walls.

The preliminary experimental campaig, where it was possible to compare the results from shove test with the results from laboratory shear triple test, allowed the definition and validation of a new procedure for the interpetation of the data colleted during the shove test. The procedure is here presented in a step by step format.

<u>Procedure:</u>

1) Identify the masonry unit to be tested with the Shove Test and the slots, above and below it, where to insert the flat jacks for both Double Flatjack Test and Shove Test (**Error! Reference source not found.**).

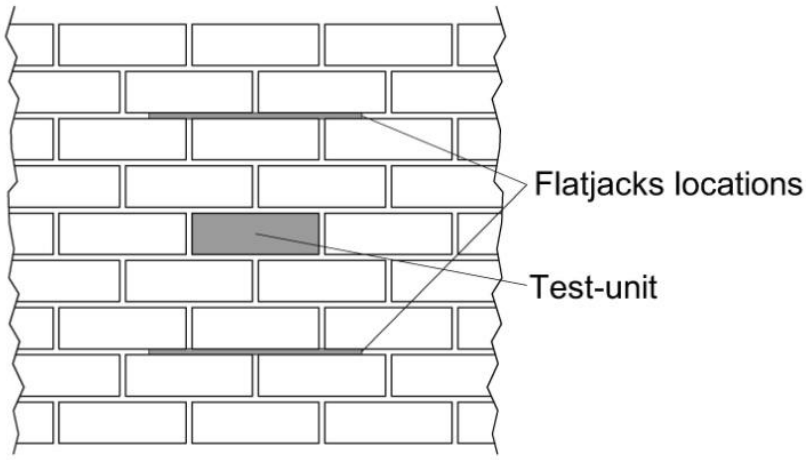


Figure 16. Identification of flat jacks slots and masonry unit location.

2) Execute the first horizontal cut in the masonry
3) Perform the Single Flat jack test.
4) Execute the second slot, at a distance of five course of bricks.
5) Perform the Double Flat Jack Test and determine the elastic modulus of the masonry *E* (Figure 17); at the end of the test keep the flat jacks in place.

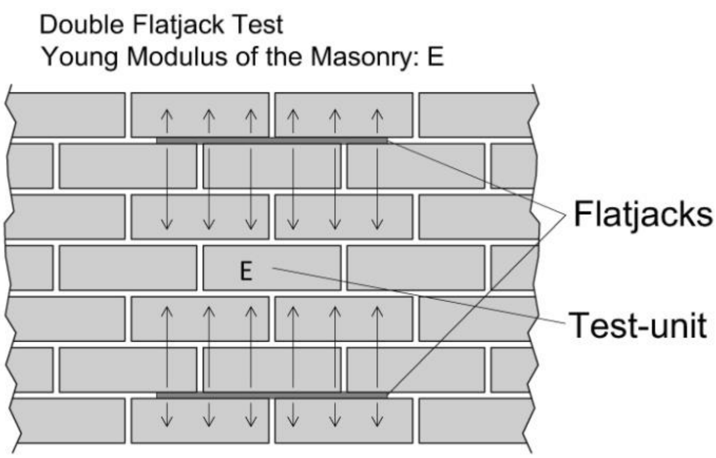


Figure 17. Double Flat Jack Test: evaluation of the Elastic Modulus of the masonry.

6) Remove the two masonry units adjacent (left and right) to the unit block that must be tested (Figure 17). The bedjoints on the side of the masonry unit to be tested must be "cleaned" to avoid that the presence of exceedances of mortar that could prevent the masonry unit from sliding.
7) Perform the Double Flat Jack Test in the "Shove Test configuration" to determine the Fictitious Elastic Modulus E^* (Figure 18). At the end of the test keep the flat jacks in place.

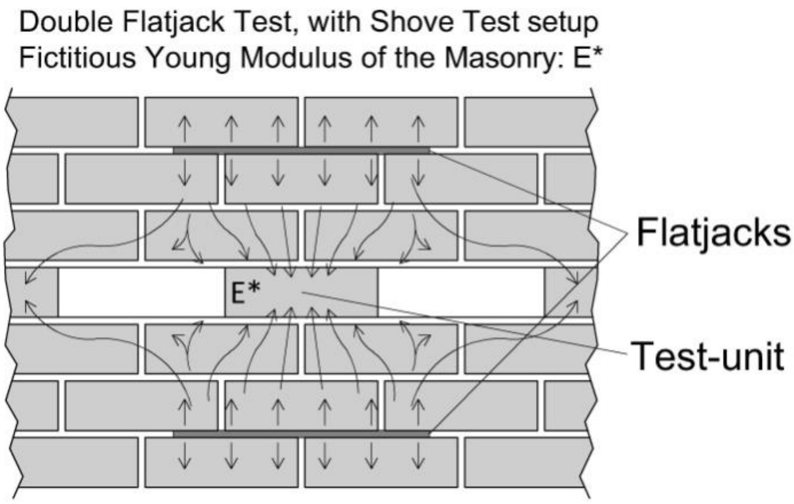


Figure 18. Modified Double Flat Jack Test: the Double Flat Jack test is perfomed in the "Shove Test configuration" (left and right birck have been removed).

8) Instrument the test unit with horizontal and vertical LVDTs (Figure 19).

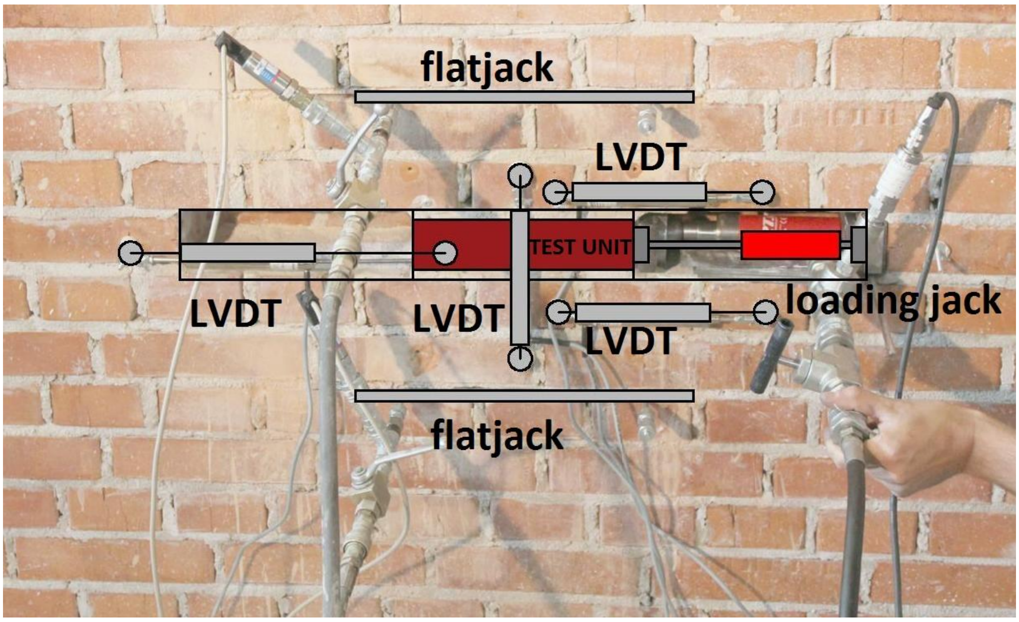


Figure 19. Shove Test: test apparatus.

9) Insert the horizontal load jack (spherical seat) and the metallic plates for the diffusion of the load (minimum thickness: 3mm), and neoprene.

Evaluation of cohesion:

10) Set the flat jacks pressure at a value lower that 0.07 MPa and maintain it constant.
11) Increase the pressure on the horizontal loading jack gradually and record all the measurements of both displacements and pressures, till the post peak phase (relative to horizontal direction). This provides information on the first rupture of the interface, taking in account of cohesion as well as friction effects.

Evaluation of friction coefficient:

12) Increase the pressure in the flat jacks till the one recorded in the Single Flat Jack Test, and maintain it constant.
13) Increase the pressure in the horizontal loading jack and record the measurements until the post peak phase. If the brick displaced more than 0.5 cm, reverse the horizontal load.
14) Repeat the procedure at least 4 times increasing the pressure in the flat jacks up to a level considered still in the "elastic phase" judging from the results of Double Flat Jack Test.
15) Release the pressure in the horizontal loading jack and remove it, eventually dismount also the instrumentation.
16) The test could be performed also in the opposite direction, just by transferring the loading jack in the opposite opening.

Calculations:

The idea of this new modified procedure is to replace the relationship that is proposed by the general procedure $\tau(\sigma_j)$, where the shear strength is expressed as a function of the stress at the flat jack interface, with the more appropriate relationship $\tau(\sigma_b)$, where the shear strength is expressed as a function of the compressive stress at the masonry test unit.
The procedure is based on two transformations:

$$\tau(\sigma_j) \quad \rightarrow \quad \tau(\sigma_{bj}) \quad \rightarrow \quad \tau(\sigma_b)$$

where:

σ_j Normal compressive stress at the flat jack interface.
σ_{bj} Normal compressive stress in the masonry test unit due to the flat jacks pressure.
σ_b Normal compressive stress in the masonry test unit, after the correction for far field effects.

As for the standard procedure the following quantities are recorded:

τ_i average mortar joint shear strength index.
σ_v normal compressive stress.

The average mortar joint shear strength index τ_i, for each level of normal compressive stress σ_v, is defined as:

$$\tau_i = \frac{P_{hi}}{A_j}$$

where:

P_{hi} maximum horizontal force resisted by the test unit at the i^{th} level of normal compressive stress (elastic limit in the current case).

A_j gross area of upper and lower bed joints in the case of solid unit masonry, or net mortar bedded area for the case of hollow-core masonry units.

The recorded couples of shear stress/compressive stress point can be plotted in the graph σ_j,τ (compressive stress at the flat jack interface against the shear stress) as shown in Figure 20. From the graph with a linear regression is it possible to obtain the values of $\tilde{\tau}_{0\text{-}res}$ and $\tilde{\mu}$.

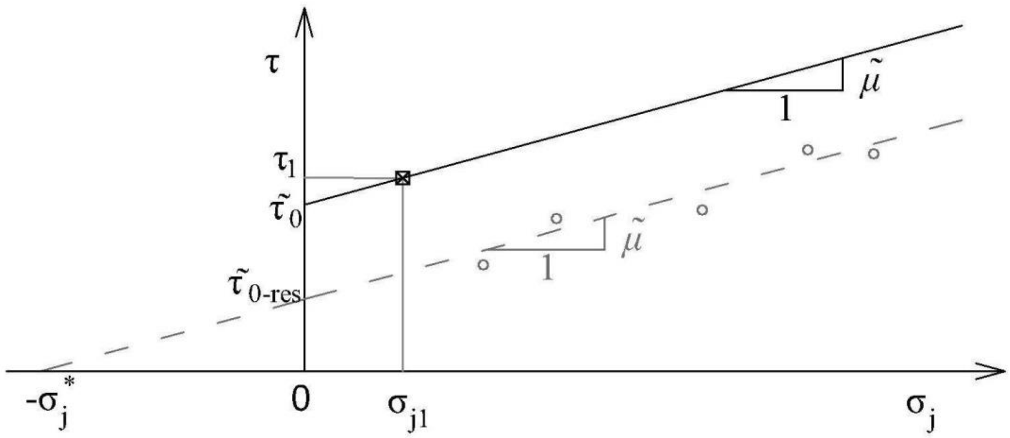


Figure 20. Shove Test Results, $\tau=\tau(\sigma_j)$.

It is then possible to evaluate the so called "jack to brick correction factor" with the following formula:

$$k_{bj} = E/E^*$$

where:

E Elastic modulus of masonry evaluated from the Double Flat Jack Test.

E^* Elastic modulus of masonry evaluated from the Flat Jack Test in the "Shove Test configuration".

The values of compressive stress at the flat jack level are then corrected using the above mentioned correction factor to obtain the equivalent values of compressive stress in the brick unit due to the flat jack pressure:

$$\sigma_{bj} = E/E^* \cdot \sigma_j = k_{bj} \cdot \sigma_j$$

At this point it is possible to plot the couple of points in the graph σ_{bj},τ (compressive stress at the masonry unit due to flat jack pressure interface against the shear stress) as shown in Figure 21. With a linear regression the value of μ can be obtained. Alternatively the latter can also be evaluated using the following formula:

$$\mu = \tilde{\mu}/k_{bj}.$$

At thiss point the following quantities can be evaluated:

$$\sigma_b{*} = \tilde{\tau}_{0\text{-}res}/\mu$$

$$\tilde{\tau}_0 = \tau_1 - \mu \cdot \sigma_{bj1}$$

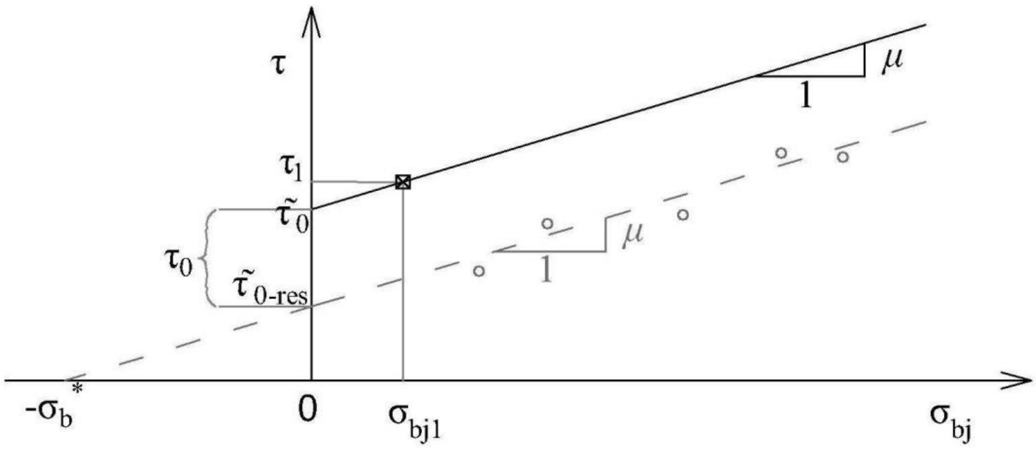


Figure 21. Shove Test Results, $\tau=\tau(\sigma_{bj})$.

Finally the choesion can be calculated as:

$$\tau_0 = \tilde{\tau}_0 - \tilde{\tau}_{0\text{-}res} = cohesion\ (c)$$

The final step is to correct the compressive stress in the masonry unit which is the sum of two factors, as shown in the followinf formula:

$$\sigma_b = \sigma_{bj} + \sigma_b{*}$$

where:

σ_{bj} Normal compressive stress in the masonry test unit due to the flat jacks pressure.

$\sigma_b{*}$ Contribution to the normal compressive stress in the test unit due to the far field effect (e.g. the weight of the wall and the load applied at the top of the wall).

Finally the results of the Shove Test can be plotted in the σ_b,τ graph (compressive stress at the masonry unit considering far field effects against the shear stress) as shown in Figure 22Figure 22 and expressed in the Coulomb style format:

$$\tau = \tau_0 + \mu \cdot \sigma_b$$

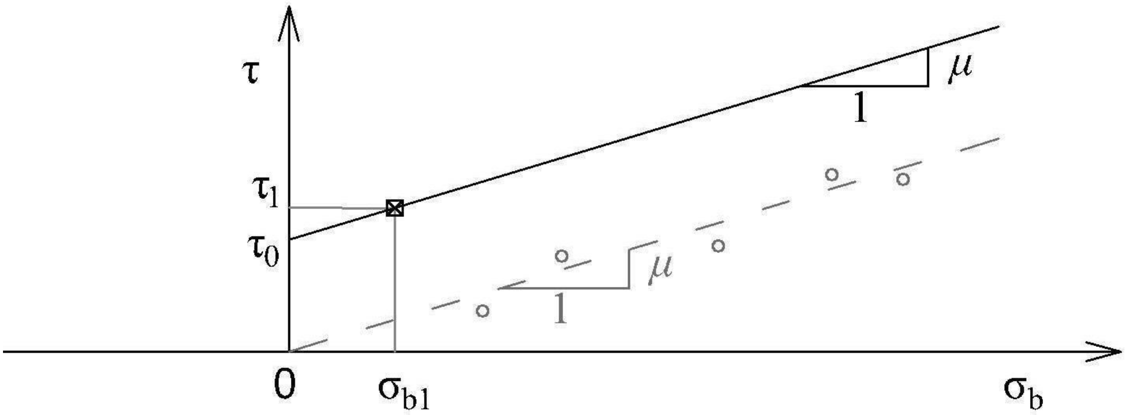


Figure 22. Shove Test Results, $\tau=\tau(\sigma_b)$.