

Nederlandse Aardolie Maatschappij B.V.

Schepersmaat 2, Postbus 28000, 9400 HH Assen

Experimental campaign on a clay URM full-scale specimen representative of the Groningen building stock



EUCENTRE®

European Centre for Training and Research in Earthquake Engineering

Via Ferrata 1, 27100 Pavia, Italy
Tel. +39.0382.516911 Fax. +39.0382.529131
<http://www.eucentre.it>
email: info@eucentre.it

LAB AND TESTS RESPONSIBLE	DOCUMENT AUTHOR	REVIEWER
Filippo Dacarro	Dr. Francesco Graziotti	Prof. Guido Magenes
Signature	Signature	Signature
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1 Introduction

1.1 Description of the test programme

As part of the NAM data acquisition and analysis programme aimed at developing a seismic hazard and risk model for induced seismicity in the Groningen area, laboratory tests were performed on full-scale structural members and a single-storey unreinforced masonry building, representative of a structural typology very common in the region. The reference single-storey full-scale test-building is a prototype structure representative of a clay URM detached house, common in the region of Groningen. It was built directly on the shaking table. The structure has been adapted to the need imposed by the laboratory constraints and it is not expected to be exhaustive of all possible variations in geometry present in the actual building stock in Groningen area, it was designed to combine several common features of detached houses pre 1940's (Arup 2016a).

This document describes the testing campaign performed at the laboratory of the European Centre for Training and Research in Earthquake Engineering (EUCENTRE) in 2016. Some of the tests were also conducted by EUCENTRE staff at the DICAr Laboratory of University of Pavia (UNIPV). The experimental programme included:

- Characterization tests on units, mortar and small masonry assemblies;
- In-plane quasi static tests on full-scale masonry piers;
- Shaking table tests on a full-scale single-storey masonry building specimen clay solid walls ideally representative of a pre-1940 detached house.

The complete list of the tests carried out at the EUCENTRE laboratory is reported in Table 1

Table 1. Summary table of tests and dates

Laboratory	Activity Name	Test type	Day of Test
UNIPV	MOR-SPEC	Bending- Compression Mortar	14/03/2016- 20/04/2016
UNIPV	BRICK-SPEC	Bending- Compression Brick	19/05/2016- 24/05/2016
UNIPV	EC-MAT -1-SPEC	Compression	29/03/2016- 06/04/2016
UNIPV	EC-MAT-2-SPEC	Compression	18-19/04/2016
UNIPV	T-SPEC	Shear	12/05/2016- 24/05/2016
UNIPV	EC-MAT-IP2-SPEC	In-Plane Bending	03/05/2016- 01/06/2016
UNIPV	EC-MAT-OOP2-SPEC	Out-Of-Plane Bending	04/06/2016- 06/06/2016
UNIPV	BW-SPEC	Bond Wrench	28/04-02/05/2016
UNIPV	ANCHOR-SPEC	Tensile strength	06/04/2016
EUCENTRE	EC-COMP2-1	In Plane cyclic	18/05/2016
EUCENTRE	EC-COMP2-2	In Plane cyclic	25/05/2016
EUCENTRE	EC-COMP2-3	In Plane cyclic	01/06/2016
EUCENTRE	EC-COMP2-4	In Plane cyclic	15/06/2016
EUCENTRE	EC-COMP2-5	In Plane cyclic	23/06/2016
EUCENTRE	EUC-BUILD	SHAKE- TABLE	12-18/04/2016

Tests on materials and components were performed in order to fully characterize the masonry adopted for all the tests on components (structural members) and on the building specimen. Tests on materials and tests on components allowed obtaining this basic information relevant for the interpretation of all the tests carried out on larger specimen and for the verification of the representativeness of the experimental results with respect to the masonry properties in real buildings in the area.

The masonry type subjected to characterization is the one used to build the specimen (same pattern, same materials, same curing conditions). Tests on small masonry assemblies, including compression tests, bond wrench tests, in plane flexural tests, OOP flexural tests, and shear tests on triplets, allowed determining the mechanical parameters of this clay brick masonry.

Five quasi-static tests were performed to assess the in-plane shear behaviour of clay brick masonry piers. To this aim, in-plane cyclic shear-compression tests were carried out on full-scale 20 cm thick masonry walls with different aspect ratios and subjected to different boundary and loading conditions. These tests provided the validation of strength criteria related to different failure modes, the direct assessment of hysteretic energy dissipation and the identification of displacement limit states.



The shaking table tests on the single-storey full-scale unreinforced masonry building was performed at the EUCENTRE laboratory in April 2016. The shaking table tests were performed applying uniaxial acceleration time-histories oriented parallel to the flexible direction of the roof. The applied strong motions were carefully selected to be consistent with the expected induced seismicity hazard in the region. An incremental dynamic test series was carried out up to the near collapse limit state of the specimen. This report presents the results obtained during the shaking table tests, illustrating the response of the structure, the observed damage mechanisms and their evolution in the different testing phases.



2 Nomenclature

Symbol	Material property
ρ	Density
f_c	Compressive strength of mortar
f_t	Flexural strength of mortar
f_b	Compressive strength of masonry unit
$f_{b,t}$	Flexural strength of masonry unit
f_m	Compressive strength of masonry in the direction perpendicular to bed joints
E_{m-1}	Elastic modulus of masonry in the direction perpendicular to bed joints (E1) - (30% f_m)
E_{m-2}	Elastic modulus of masonry in the direction perpendicular to bed joints (E2) - (10% f_m)
E_{m-3}	Elastic modulus of masonry in the direction perpendicular to bed joints given by the secant line passing through the points (30% f_m , $\epsilon_{30\% f_m}$) and (10% f_m , $\epsilon_{10\% f_m}$) (E3)
ν_m	Poisson ratio of masonry in the direction perpendicular to bed joints
$f_w; f_{x1}$	Flexural bond strength
f_{x2}	Out-of-plane flexural strength perpendicular to the bed joint
f_{x3}	In-plane flexural strength
f_{v0}	Masonry (bed joint) initial shear strength
μ	Masonry (bed joint) shear friction coefficient
F_t	Tensile load capacity of flat anchors



3 Test for materials characterization

3.1 Flexural and compressive strength of masonry unit

Mechanical properties of the bricks and the mortar have been evaluated by means of compressive and flexural tests.

The bricks used to build the full-scale test house and the others specimens are clay bricks as in the Dutch construction technique. They were tested by compressive tests, in order to evaluate the compressive strength of masonry unit f_b according to EN 772-1, and by bending tests, in order to evaluate the tensile strength of masonry unit f_{bt} according to the Dutch Norm NEN 6790. Figure 1 shows the bricks dimensions.



Figure 1. Mean dimensions of the clay bricks

Seven specimens for each test typology were tested at the DICAr Laboratory of University of Pavia.

3.1.1 Compressive strength of clay brick

Specimens were tested by means of a monotonic load protocol until reaching failure, following the procedures recommended by EN 772-1. Tests have been performed on 19/05/2016.

Table 2 summarize the characteristics of the specimens as well as the results of the tests in terms of ultimate load and compressive strength.

In order to obtain the normalized compressive strength f_b , the compressive strength of masonry unit is multiplied by a shape factor d , which depends on the width and height of the brick.

Table 2. Compressive tests on masonry unit results.

Specimen #	Dimensions			Weight	Density	Ultimate Load		Norm. strength	
	B [mm]	H [mm]	L [mm]	W [g]	ρ [kg/m ³]	P [t]	f_b [MPa]	d [-]	f_b [MPa]
Brick 1	99	49	207	2070	2061.4	138.4	66.25	0.75	49.69
Brick 2	98	49	205	2057	2086.2	110.0	53.84	0.75	40.38
Brick 3	99	50	208	2170	2128.9	121.8	58.03	0.75	43.52
Brick 4	98	50	206	2076	2082.5	112.0	54.56	0.75	40.92
Brick 5	99	49	205	2136	2150.0	147.2	71.51	0.75	53.63
Brick 6	99	49	206	2050	2066.9	136.6	66.20	0.75	49.65
Brick 7	98	50	206	2116	2133.5	135.4	66.29	0.75	49.72
Mean					2101.3	128.8	62.38	-	46.8
St. Dev.					35.4	14.3	6.85	-	5.1
C.o.V.					1.7%	11.1%	11.0%	-	11.0%

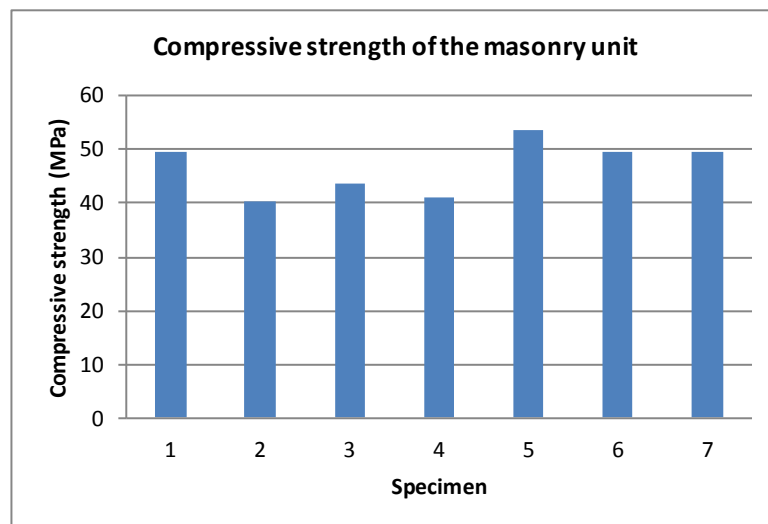


Figure 2. Results of compressive test on masonry unit.



Figure 3. Compressive failure of masonry unit.

3.1.2 Flexural strength of the clay brick

A specific test set-up is developed, using procedures recommended by NEN 6790:2005. Tests have been performed on 24/05/2016.

The bricks are subjected to a three point bending test to calculate their flexural tensile strength; therefore, the specimen is rotated 90 degrees relative to its normal position, as shown in Figure 4, and it is supported by two rollers. The distance between the two supports and an end of the specimen is 10 mm. The load is applied to the specimen by a third cylinder, located at mid-span. Figure 6 shows a view of the test set up.

The failure is reached by mean of a monotonic loading protocol, in 30 to 90 seconds according to NEN 6790:2005.

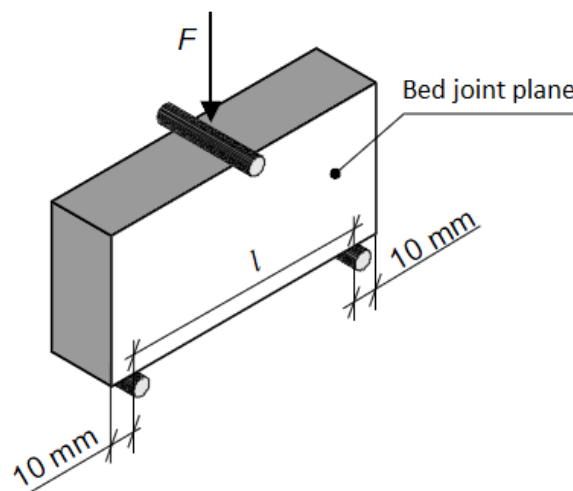


Figure 4. Brick flexural strength test set-up according to NEN 6790

The flexural strength f_{bt} is given by the formula:

$$f_{bt} = 1.5 \frac{F \cdot l}{h \cdot b^2}$$

- F : maximum load applied to the specimen
- l : distance between the two bearing rollers
- h : height of the specimen=50 mm
- b : width of the specimen=100 mm

Table 3 summarize the characteristics of the specimens as well as the results of the tests in terms of ultimate load F and tensile strength f_{bt} .

Table 3. Flexural test on masonry unit results

<i>Specimen</i>	<i>B</i> [mm]	<i>H</i> [mm]	<i>L</i> [mm]	<i>F</i> [kN]	<i>f_{bt}</i> [MPa]
Brick 1	99	50	207	14.5	8.3
Brick 2	100	50	207	15.1	8.5
Brick 3	100	50	208	15.6	8.8
Brick 4	98	50	208	13.4	7.9
Brick 5	98	50	207	14.4	8.4
Brick 6	98	51	208	16.1	9.3
Brick 7	98	51	208	14.1	8.1
<i>Mean</i>				14.8	8.5
<i>St. dev.</i>				0.90	0.45
<i>C.o.V.</i>				6.1%	5.3%

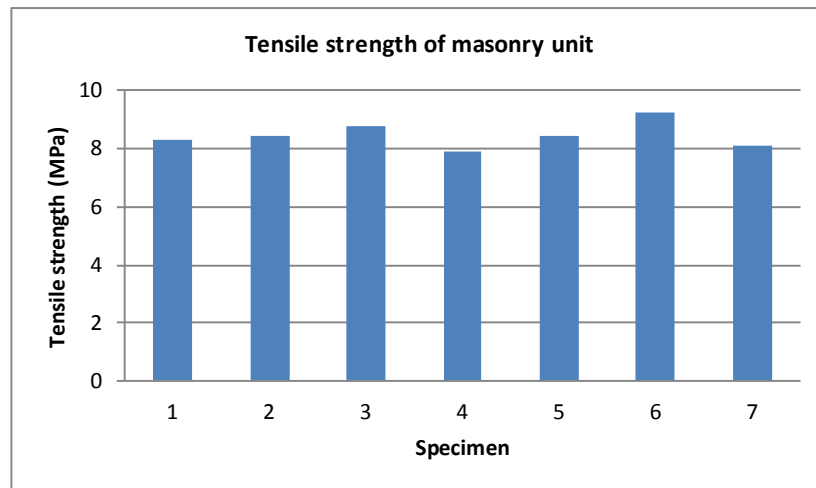


Figure 5. Results of flexural tests on masonry unit.

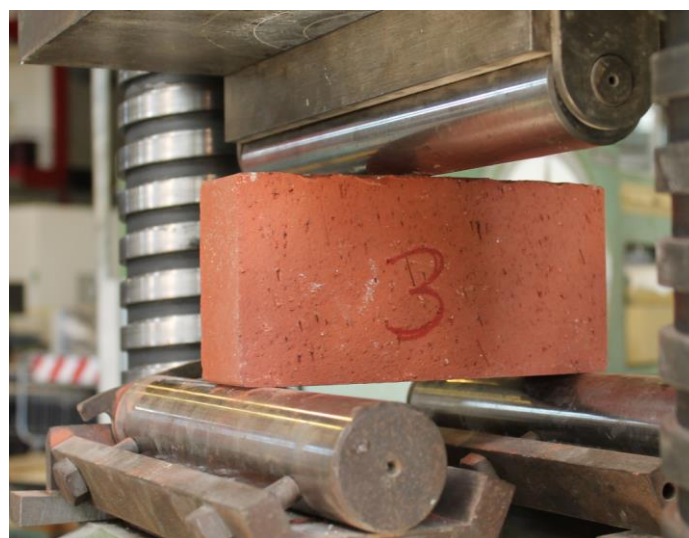


Figure 6. View of the flexural test on masonry unit set up.



Figure 7. Failure of the Brick 3

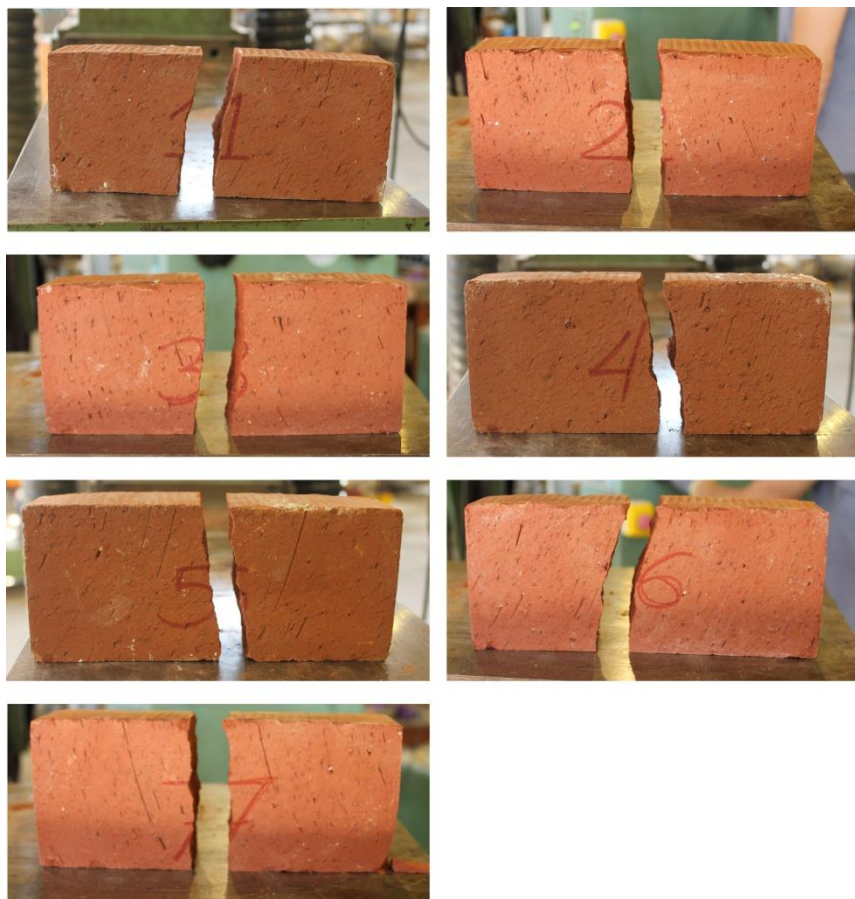


Figure 8. Flexural failures of the specimens

3.2 Flexural and compressive strength of mortar

The mechanical properties of mortar are fundamental to evaluate the overall behavior of masonry panels. Hence, the mortar used to build both the masonry specimens for complementary tests and the full-scale building, was tested in order to define its flexural and compressive strength.

There are five sets of tests according to the different uses of the mortar:

1. The first series of test is relative to the mortar cast during the construction of the full-scale test-house;
2. The second set is relative to the mortar used to build the full-scale piers specimens;
3. The third set is relative to the mortar used to build the flexural test wallettes and the 6 small wallettes for the compression tests;
4. The fourth set is relative to the mortar used to build the 3 large wallettes for the compression tests;
5. The last set is relative to the mortar used to build triplets and bond wrench specimens.

Sampling of the specimens

The specimens have dimensions 160x40x40 mm; they were stored in a mould for 48 hours and then, once demolded, for five days in a moist atmosphere, after that they were left uncovered until strength testing. The mould used to cast the mortar samples is represented in Figure 9.

The samples of mortar were cast twice per day, during the construction of the test-house and the masonry specimens used for the in-plane and out-of-plane tests.

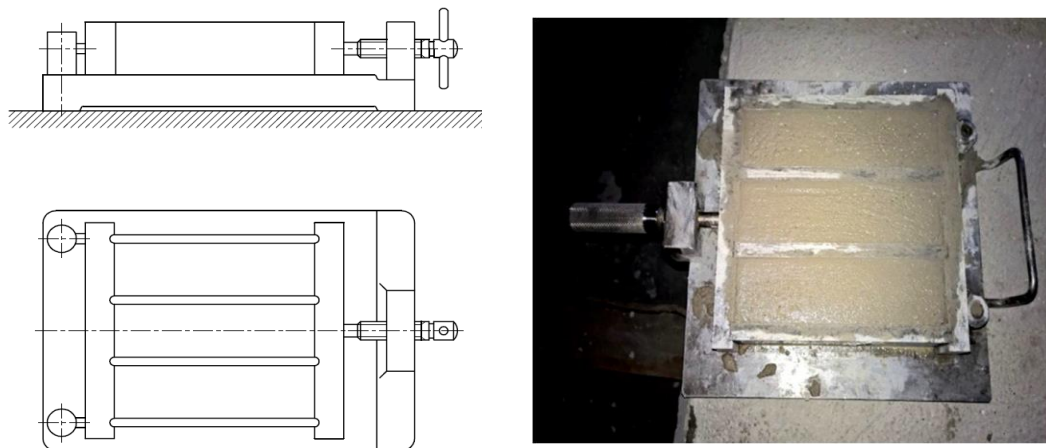


Figure 9. Mould to cast the mortar.

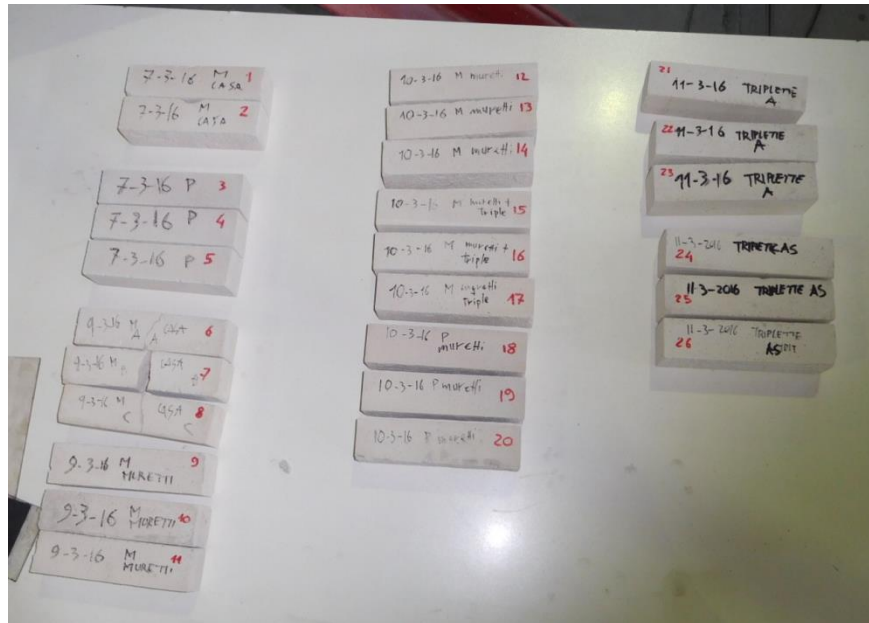


Figure 10. View of the mortar specimens

Experimental Procedure

The tests were conducted according to EN 1015-11 at DICAr Laboratory of University of Pavia.

The specimens were loaded in bending and the resulting two broken parts were loaded in compression later on.

The load "1" is applied with a constant velocity in between 100 to 50 N/s, in order to obtain the fracture of the specimen after a time in between 30 and 90 seconds. The set up of the bending test for the determination of the flexural strength of the mortar is shown in Figure 11.

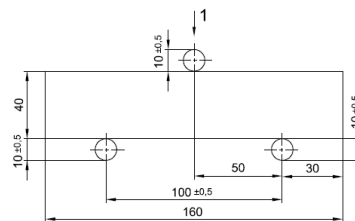


Figure 11. Test arrangement according to EN 1015-11

The flexural strength, f_t is given by the formula:

$$f_t = 1.5 \frac{F \cdot l}{b \cdot d^2}$$

F : maximum load applied on the specimen

l : distance between the rollers = 100 mm

b : width of the specimen = 40 mm

d : thickness of the specimen = 40 mm

After the flexural test the specimen results to be divided into two parts (almost identical), that were subsequently tested in compression.

The compressive load must be increased at a constant velocity in between 50 N/s and 500 N/s, in order to obtain the failure of the specimen after a time in between 30 and 90 seconds.



Figure 12. Bending test on the mortar specimen



Figure 13. Compressive test on half specimen of the mortar

Test Results

The following Tables show the results of the tests, carried out on mortar used to built the house and the specimens for complementary tests. All the results below concern mortars tested after 28 days of maturation.

Table 4. Results of tests on mortar for the full-scale test-house construction.

Masonry specimen	Casting date	Test date	Maturation [days]	Weight [g]	f_c [MPa]	f_t [MPa]
house	25/02/2016	24/03/2016	28	422.8	5.11	1.29
house	25/02/2016	24/03/2016	28	425.8	5.03	1.20
house	25/02/2016	24/03/2016	28	419.2	4.64	1.61
house	25/02/2016	24/03/2016	28	418.6	4.69	1.56
house	25/02/2016	24/03/2016	28	420.0	4.54	1.56
house	27/02/2016	29/03/2016	31	416.8	3.54	1.75
house	27/02/2016	29/03/2016	31	421.3	3.84	-
house	27/02/2016	29/03/2016	31	418.3	4.25	0.80
house	27/02/2016	29/03/2016	31	420.2	3.75	0.92
house	27/02/2016	29/03/2016	31	415.0	4.34	1.06
house	29/02/2016	29/03/2016	29	426.7	4.03	1.33
house	29/02/2016	29/03/2016	29	426.3	3.78	1.01
house	29/02/2016	29/03/2016	29	421.1	3.67	1.33
house	02/03/2016	30/03/2016	28	416.8	2.86	1.01
house	02/03/2016	30/03/2016	28	423.3	3.80	1.10
house	07/03/2016	19/04/2016	43	440.8	5.86	2.25
house	07/03/2016	19/04/2016	43	438.8	2.36	1.15
house	07/03/2016	19/04/2016	43	410.3	4.22	1.61
house	07/03/2016	19/04/2016	43	407.8	3.49	1.63
house	07/03/2016	19/04/2016	43	421.8	4.05	1.66
house	09/03/2016	19/04/2016	41	429.1	3.89	-
house	09/03/2016	19/04/2016	41	425.6	4.38	-
house	09/03/2016	19/04/2016	41	428.6	5.04	-
mean				422.4	4.13	1.36

Notice: some values of flexural strength are missing (-) because of the impossibility to perform the test due to accidental damages occurred to the specimen.

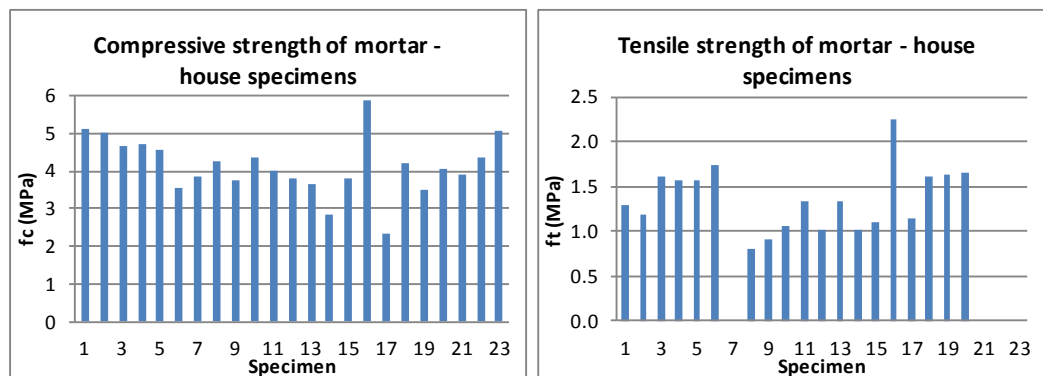


Figure 14. Results of tests on mortar for the full-scale test-house construction.

Table 5. Results of tests on mortar for the full-scale piers construction.

Masonry specimen	Casting date	Test date	Maturation [days]	Weight [g]	f_c [MPa]	f_t [MPa]
full-scale piers	01/03/2016	29/03/2016	28	391.4	3.10	1.01
full-scale piers	01/03/2016	29/03/2016	28	387.2	1.61	0.37
full-scale piers	01/03/2016	29/03/2016	28	399.2	2.54	1.10
full-scale piers	01/03/2016	29/03/2016	28	412.0	3.04	1.20
full-scale piers	01/03/2016	29/03/2016	28	408.7	3.34	0.87
full-scale piers	02/03/2016	30/03/2016	28	442.6	3.66	1.47
full-scale piers	02/03/2016	30/03/2016	28	439.1	3.47	0.97
full-scale piers	02/03/2016	30/03/2016	28	436.8	4.18	1.20
full-scale piers	02/03/2016	30/03/2016	28	424.9	3.90	1.15
full-scale piers	02/03/2016	30/03/2016	28	425.1	3.87	0.87
full-scale piers	02/03/2016	30/03/2016	28	426.7	3.27	0.92
mean				417.61	3.27	1.01

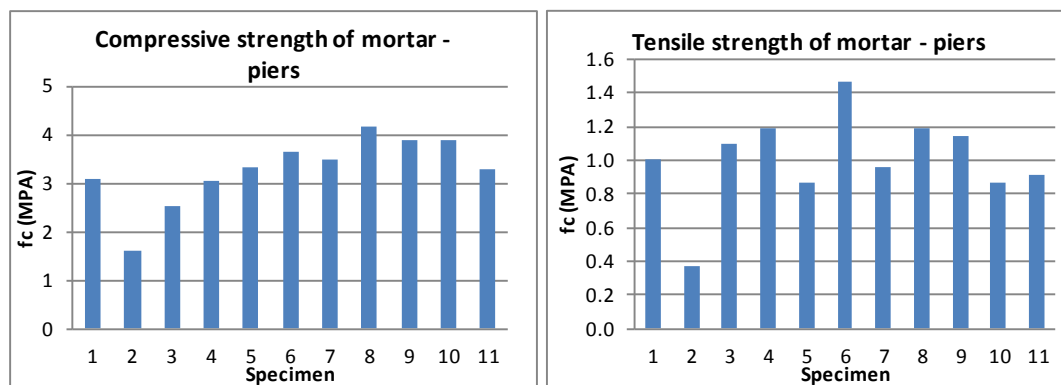


Figure 15. Results of tests on mortar for the full-scale piers construction.

Table 6. Results of tests on mortar for the wallettes construction.

<i>Masonry specimen</i>	<i>Casting date</i>	<i>Test date</i>	<i>Maturation [days]</i>	<i>Weight [g]</i>	<i>f_c [MPa]</i>	<i>f_t [MPa]</i>
wallettes	09/03/2016	19/04/2016	41	426.1	4.66	1.26
wallettes	09/03/2016	19/04/2016	41	423.2	4.06	0.00
wallettes	09/03/2016	19/04/2016	41	427.5	4.57	1.52
wallettes	10/03/2016	20/04/2016	41	434.1	6.84	1.79
wallettes	10/03/2016	20/04/2016	41	431.6	6.84	1.79
wallettes	10/03/2016	20/04/2016	41	433.0	6.82	1.98
wallettes	10/03/2016	20/04/2016	41	421.0	4.86	1.17
wallettes	10/03/2016	20/04/2016	41	422.2	4.72	1.10
wallettes	10/03/2016	20/04/2016	41	430.3	5.20	1.47
mean				427.7	5.40	1.34

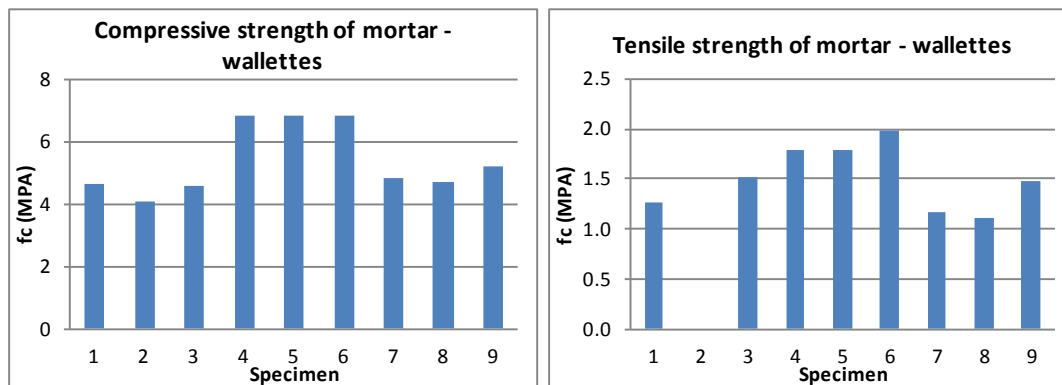


Figure 16. Results of tests on mortar for the wallettes construction.

Table 7. results of tests on mortar for the large wallettes construction.

<i>Masonry specimen</i>	<i>Casting date</i>	<i>Test date</i>	<i>Maturation [days]</i>	<i>Weight [g]</i>	<i>f_c [MPa]</i>	<i>f_t [MPa]</i>
wallettes (large)	10/03/2016	20/04/2016	41	411.7	3.54	0.83
wallettes (large)	10/03/2016	20/04/2016	41	404.4	3.48	0.92
wallettes (large)	10/03/2016	20/04/2016	41	405.2	3.56	1.15
mean				407.1	3.53	0.97

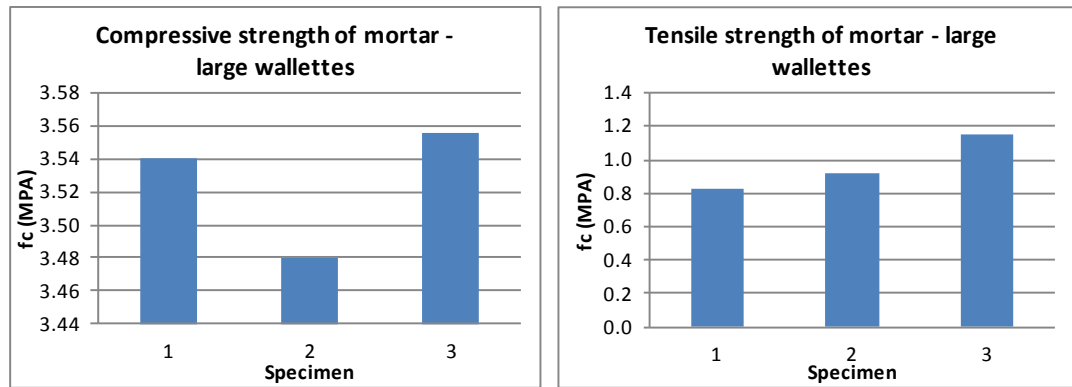


Figure 17. Results of tests on mortar for the large wallettes construction.

Table 8. results of tests on mortar for the triplets and bond wrench construction.

Masonry specimen	Casting date	Test date	Maturation [days]	Weight [g]	f_c [MPa]	f_t [MPa]
triplets, bond wrench	11/03/2016	20/04/2016	40	451.9	3.94	1.01
triplets, bond wrench	11/03/2016	20/04/2016	40	448.3	4.26	1.06
triplets, bond wrench	11/03/2016	20/04/2016	40	449.0	4.03	1.01
triplets, bond wrench	11/03/2016	20/04/2016	40	449.7	3.85	1.06
triplets, bond wrench	11/03/2016	20/04/2016	40	447.4	3.85	1.10
triplets, bond wrench	11/03/2016	20/04/2016	40	449.4	3.94	0.60
mean				449.3	3.98	0.97

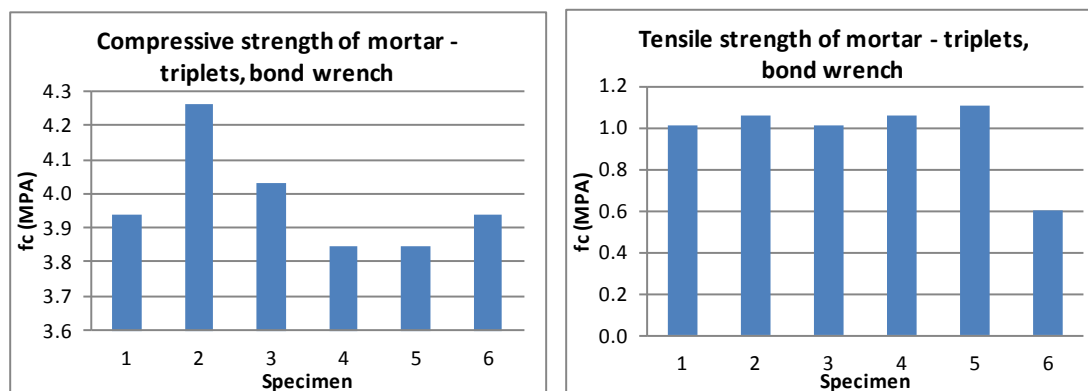


Figure 18. Results of tests on mortar for the triplets and bond wrench construction.

The following Table aims to reorganize the results, reporting the average values of compressive and tensile strength, with relative C.o.V. divided by specimen typology. Notice that the mechanical characteristics of the mortar used to build the house are slightly higher (and associated to a higher C.o.V.) than the one used for the companion specimens.

Table 9. Mean values of compressive and tensile strength of the mortar for each specimen typology

Masonry specimen	Compressive strength - f_c			Tensile strength - f_t		
	Mean [MPa]	St. Dev. [MPa]	C.o.V.	Mean [MPa]	St. Dev. [MPa]	C.o.V.
house	4.14	0.76	18%	1.36	0.36	26%
full-scale piers	3.27	0.72	22%	1.01	0.28	27%
wallettes (large)	3.53	0.04	1%	0.97	0.17	17%
triplets, bond wrench	3.98	0.16	4%	0.97	0.19	19%
All	3.85	0.75	20%	1.17	0.35	30%

Additional compressive tests were performed in order to assess the development of the full mechanical properties of mortar during the maturation period. The following graph shows the dependence of the mechanical properties of mortar with respect to the maturation period. Each point represents the average value of strength of the specimens reached at the corresponding time of maturation.

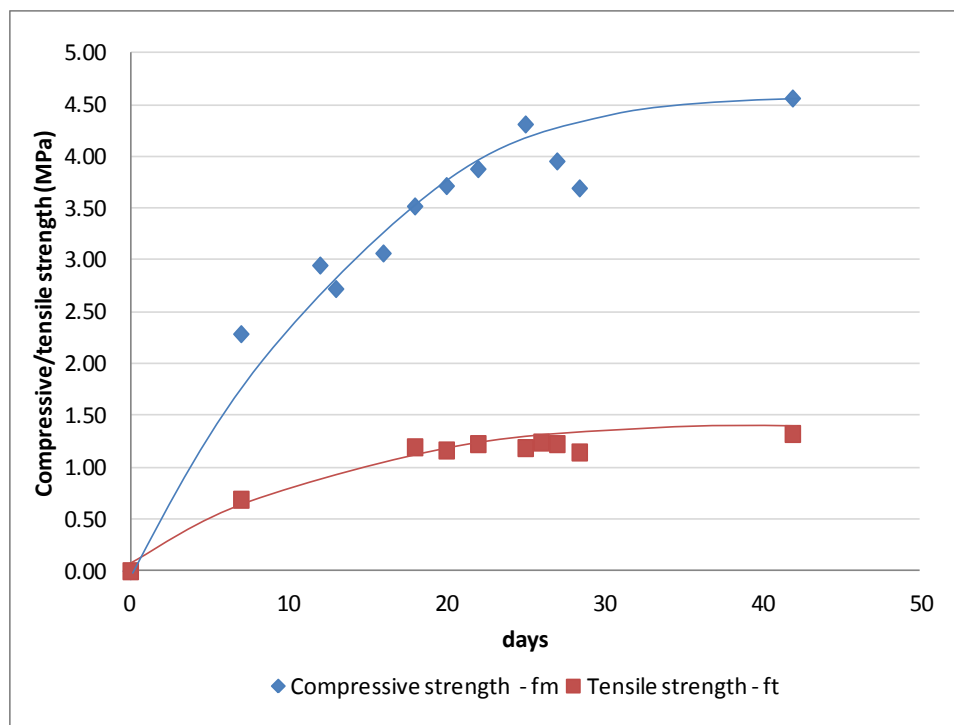


Figure 19. Compressive and tensile strength depending on the maturation period.

3.3 Compressive strength of masonry

Compression tests on masonry were performed at the DICAr Laboratory of the University of Pavia. Tested masonry specimens were made up of the same materials used to build the full-scale test-house. The compressive strength, evaluated according to the European Norm EN 1052-1:1998 procedure, is referred to the direction perpendicular to the bedjoints of the masonry specimens.

Two series of compression tests on masonry wallettes were performed according to the two wallette typologies made:

1. Compression tests performed on the first group of wallettes (6 tests) erroneously built using a very similar bond (English bond). These specimens are named EC_MAT_1 plus an ID letter; their geometrical characteristics are listed in Table 10.
2. Compression tests performed on the second group of wallettes (3 tests) larger than the first sample and built using the Dutch bond. These specimens are named EC_MAT_2 plus an ID letter, their geometrical characteristics are listed in Table 12.

The average mechanical characteristics computed from the tests on EC_MAT_2 were considered more representative. Since only 3 specimens were available, the C.o.V. of the results were computed considering the results on EC_MAT_1 tests.

3.3.1 Compressive strength of masonry – EC_MAT_1

The test samples are composed by 6 masonry wallettes listed in the following Table. The specimens were built adopting the same materials used to build the test-house and the full-scale piers, but with a different bond (English). These specimens are named EC_MAT_1 plus an ID letter.

Table 10 Specimen ID number and geometric properties

Specimen	Height [mm]	Width [mm]	Thickness [mm]	Area [mm²]	Weight [kg]	Density [kg/m³]
EC_MAT_1_A	662	432.5	208.5	90176	118	1978
EC_MAT_1_B	666	436.25	209	91176	121	1996
EC_MAT_1_C	664	430.5	207.5	89329	117	1976
EC_MAT_1_D	670	440.25	207	91132	120	1968
EC_MAT_1_E	667	432.5	207.75	89852	120	1999
EC_MAT_1_F	659	433.25	207.75	90008	118	1989

The masonry wallettes was built on 09/03/2016. The compression tests were performed from 29th March 2016 to 6th April 2016. The wallettes were composed by 11 layers of

bricks; the specimens dimensions were chosen according to the UNI EN-1052-1 prescriptions (section 7.1). As shown in Figure 20, the specimens are 660 mm high, 430 mm wide and 210 mm thick. Figure 20 also shows the specimens brick texture, the instrumentation adopted and the ID number associated to each potentiometer.

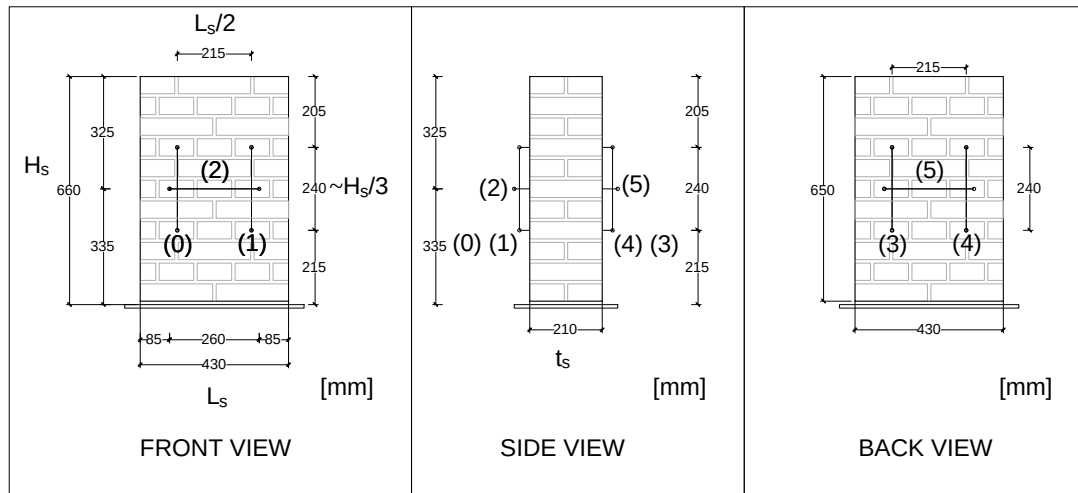


Figure 20. Geometry and instrumentation of tested wallets

The instrumentation on the front side of the specimens included two 240 mm long vertical potentiometers, spaced at 215 mm, and a 260 mm long horizontal potentiometer, positioned at mid-height of the sixth course of bricks. On the back side two 240 mm long vertical potentiometers, spaced at 215 mm, were placed and a 260 mm long horizontal potentiometer were positioned at mid-height of the sixth layer of bricks.

Figure 21 shows the test layout and the positioning of the test specimen on the testing machine.



Figure 21. Set up of the compressive test on the wallets

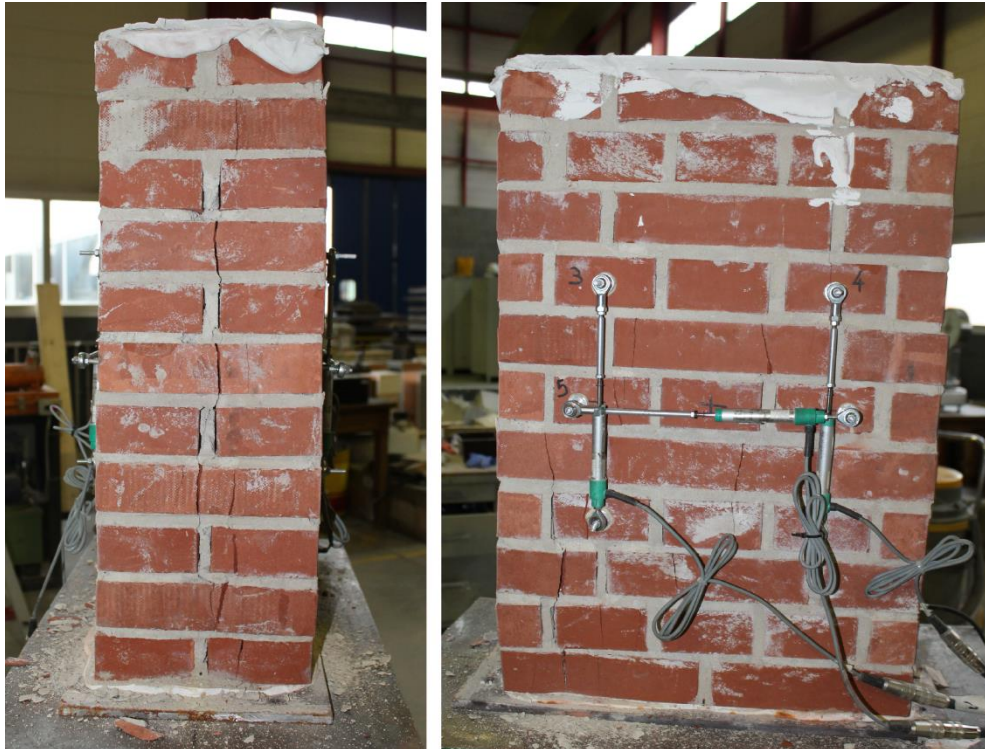


Figure 22. View of the cracks on specimen EC_MAT_1_D after the test

Experimental Procedure

The test envisaged the failure of the specimen by means of a vertical compression load applied on top. The loading protocol consisted on series of 3 cycles of loading and unloading at constant force amplitude; the latter was then increased progressively for the further cycles and the force increment was equal to either 200 or 250 kN. The loading velocity is consistent with UNI EN-1052-1 prescriptions. Figure 23 shows the applied loading history of specimen EC_MAT_1_A till failure.

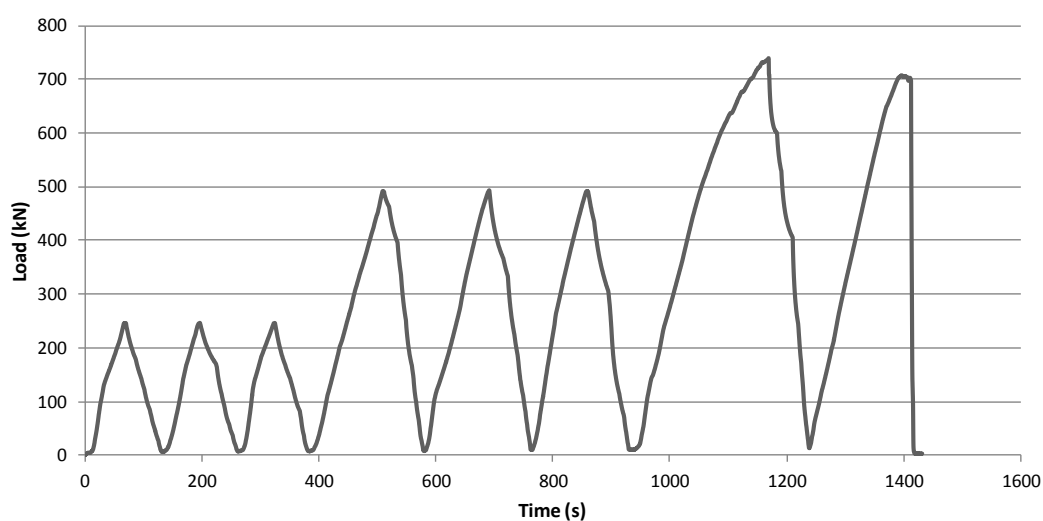


Figure 23- Loading history used for the compressive test EC_MAT_1_A

Test results

The following table summarizes the maximum stress reached by each specimen (f_m); the Young moduli computed as the slope of the lines passing through the origin and respectively through the points at 33% (E_1) or 10% (E_2) of the maximum stress; and the Young modulus E_3 computed as the slope of secant line passing through the points at 33% and 10% of f_m .

Table 11. Compressive strength and Elastic Moduli of the wallettes.

	Compressive strength	Elastic Moduli		
	f_m [MPa]	E_1 (33% f_m) [MPa]	E_2 (10% f_m) [MPa]	E_3 (10%-33% f_m) [MPa]
EC_MAT_1_A	8.20	6643	17031	5266
EC_MAT_1_B	9.91	5781	5989	5696
EC_MAT_1_C	9.84	7791	12911	6659
EC_MAT_1_D	9.26	7511	9919	6803
EC_MAT_1_E	8.84	9704	12499	8855
EC_MAT_1_F	9.36	11310	20412	9495
<i>Mean</i>	9.23	8123	13127	7129
St. Dev.	0.64	2040	5099	1699
C.o.V.	7%	25%	39%	24%

3.3.2 Compressive strength of masonry – EC_MAT_2

The test sample is composed by 3 masonry wallettes listed in the following Table. The specimens were built adopting the same materials, the Dutch construction technique and the Dutch bond used to build the test-house and the full-scale piers. These specimens are named EC_MAT_2 plus an ID letter.

Table 12. Specimen ID number and geometric properties of the second series.

Specimen	Height [mm]	Width [mm]	Thickness [mm]	Area [mm ²]	Weight [kg]	Density [kg/m ³]
EC_MAT_2_A	664.5	529.3	206.5	109290	143	1972
EC_MAT_2_B	662	533	204.6	109065	143	1975
EC_MAT_2_C	667	531.5	204.5	108692	142	1961

The date of construction of the masonry wallettes was 10/03/2016. The compression tests were performed on 18th April 2016 and 19th April 2016. The wallettes were composed by 11 layers of clay bricks; the specimens dimensions were chosen according to the UNI EN-1052-1 prescriptions (section 7.1). As shown in Figure 24, the specimens are 660 mm high, 530 mm wide and 210 mm thick. Figure 24 also shows the specimens brick texture, the instrumentation adopted and the ID number associated to each potentiometer.

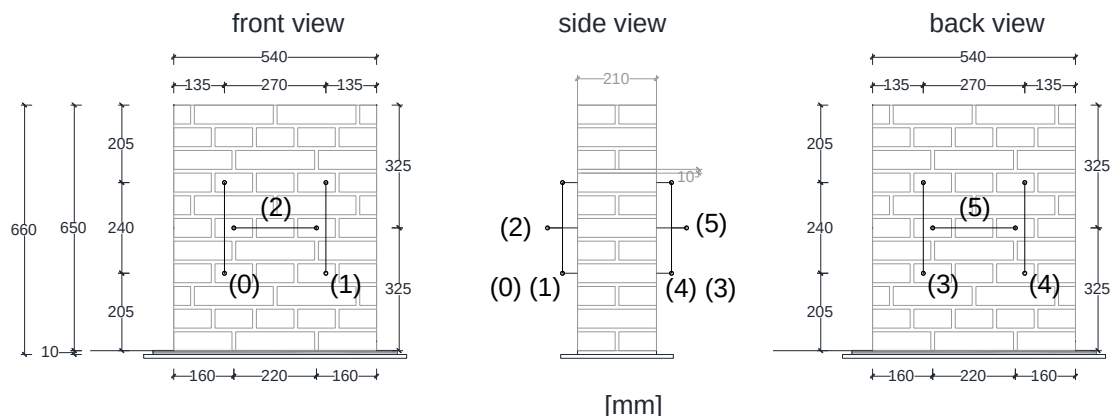


Figure 24. Geometry and instrumentation of tested wallettes.

The instrumentation on the front side of the specimens included two 240 mm long vertical potentiometers, spaced at 270 mm, and a 220 mm long horizontal potentiometer, positioned at mid-height of the sixth course of bricks. On the back side two 240 mm long vertical potentiometers, spaced at 270 mm, were placed and a 220 mm long horizontal potentiometer were positioned at mid-height of the sixth layer of bricks.

Figure 25 shows the test layout and the positioning of the instrumentation while Figure 26 shows the specimen at the end of the test.



Figure 25. Setup of the compressive test of the masonry wallette EC_MAT_2_C.



Figure 26. View of the cracks on wallette EC_MAT_2_C after the test.

Experimental Procedure

The test envisaged the failure of the specimen by means of a vertical compression load applied on top. The loading protocol consisted on series of 3 cycles of loading and unloading at constant force amplitude; the latter was then increased progressively for the further cycles and the force increment was equal to 300 kN. The loading velocity is consistent with UNI EN-1052-1 prescriptions. Figure 27 shows the applied loading history of specimen EC_MAT_2_C till failure.

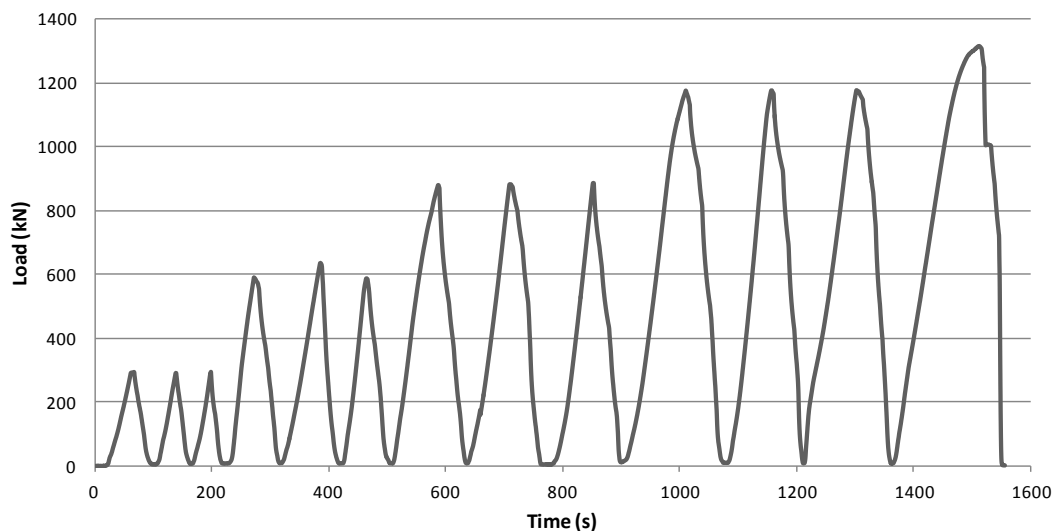


Figure 27. Loading history used for the compressive test EC_MAT_2_C.

Test results

The vertical and horizontal deformations were obtained averaging the deformation recorded by each vertical and horizontal potentiometer. In particular, the vertical deformation is obtained averaging the one recorded by potentiometers 0-1-3-4, while the horizontal deformation is obtained from horizontal potentiometers 2 and 5.

Young modulus ($E1$) is the secant elastic modulus at 33% of f_m and the origin (0,0).

Young modulus ($E2$) was computed as the secant at 10% of f_m and the origin (0,0).

Young modulus ($E3$) was computed as the secant at 10% of f_m and 33% of f_m .

Other calculation on elastic modulus (*e.g.* considering the unloading and reloading phase could be calculated if needed).

Looking at the cyclic pressure-deformation curve, it can be seen as specimens exhibit a non-linear behavior even for low level of vertical pressure. Figure 28 shows the stress-deformation curve ($\sigma - \epsilon$) of the specimen EC_MAT_2_C.

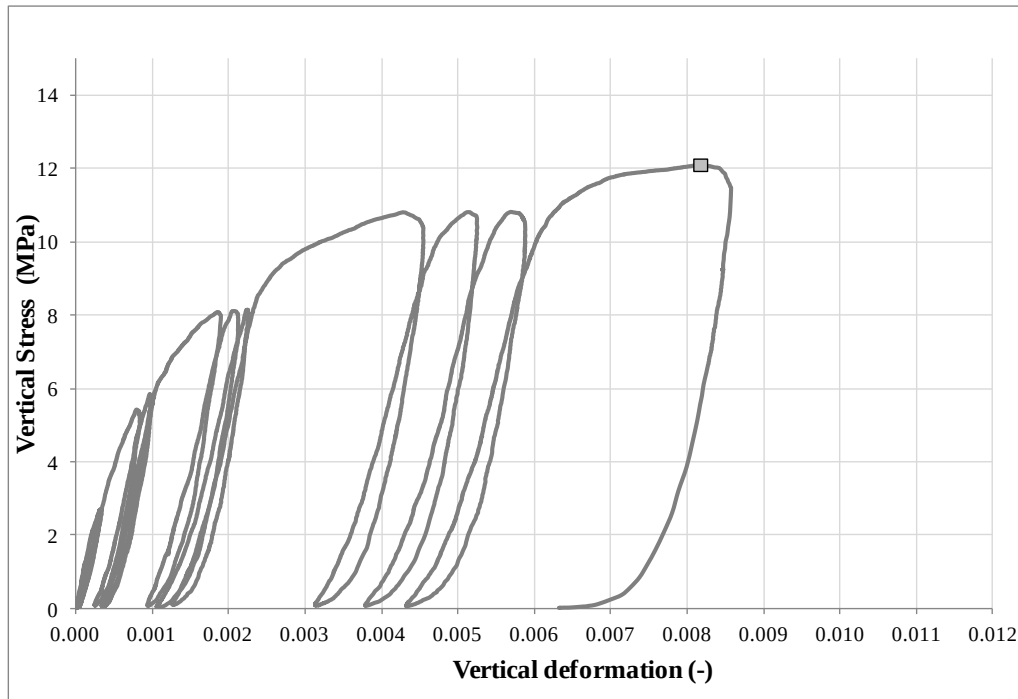


Figure 28. Vertical stress vs. vertical deformation relationship of the specimen EC_MAT_2_C.

The following table summarizes the maximum stress reached by each specimen (f_m); the Young moduli computed as the slope of the lines passing through the origin and respectively through the points at 33% (E_1) or 10% (E_2) of the maximum stress; and the Young modulus E_3 computed as the slope of secant line passing through the points at 33% and 10% of f_m .

Table 13. Compressive strength and Elastic moduli of the wallettes.

Specimen	Compressive strength	Elastic Moduli		
	f_m [MPa]	E_1 (33% f_m) [MPa]	E_2 (10% f_m) [MPa]	E_3 (10%-33% f_m) [MPa]
EC_MAT_2_A	11.21	10750	13412	9907
EC_MAT_2_B	10.35	11058	15099	9920
EC_MAT_2_C	12.10	7690	10559	6888
Mean	11.22	9833	13023	8905

The values of C.o.V. are computed from the analysis of EC_MAT_1.

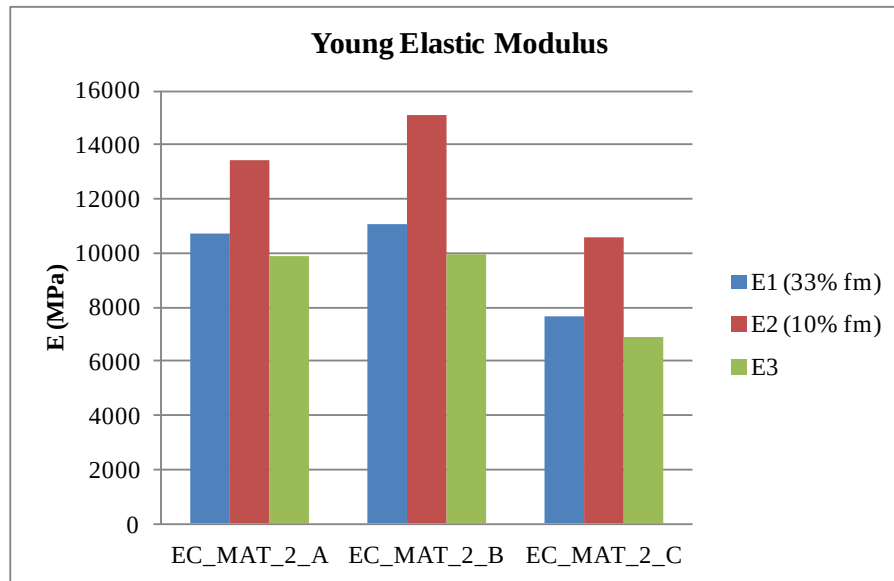


Figure 29. Elastic moduli of wallettes.

The following figures show the vertical stress vs. vertical deformation (black line) and the vertical stress vs. horizontal deformation (red line) related to each specimen tested.

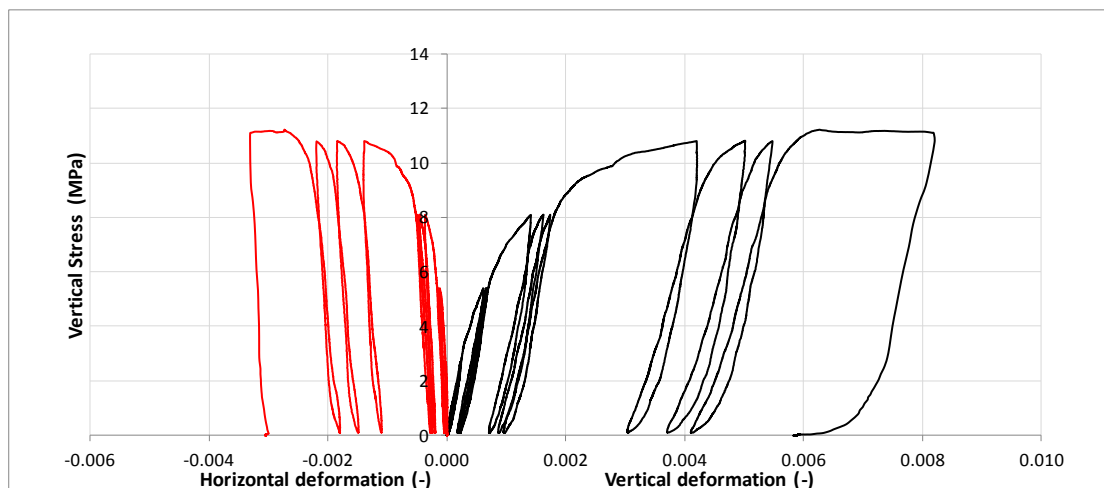


Figure 30. EC_MAT_2_A: Vertical stress vs. vertical and horizontal deformations.

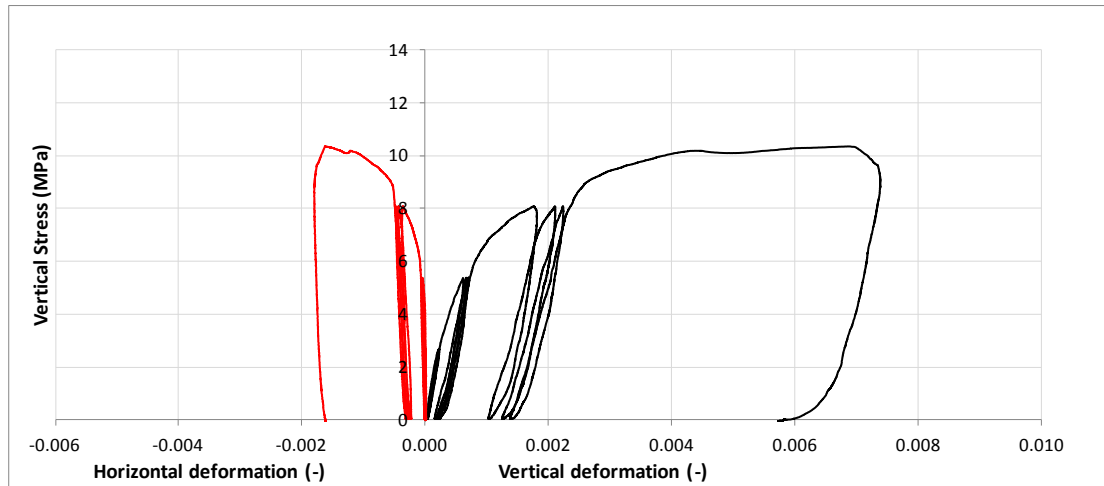


Figure 31. EC_MAT_2_B: Vertical stress vs. vertical and horizontal deformations.

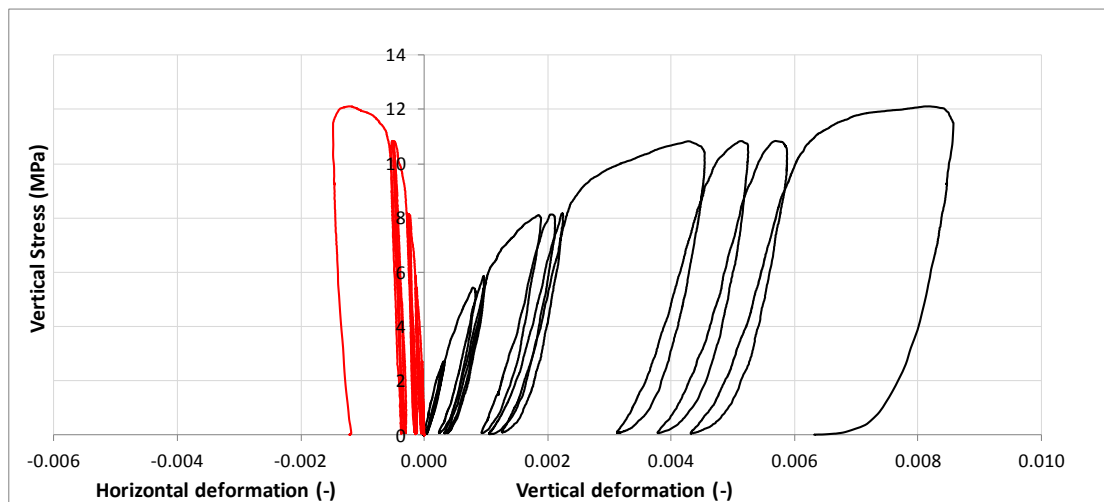


Figure 32. EC_MAT_2_C: Vertical stress vs. vertical and horizontal deformations.

The comparison between EC_MAT_1 and EC_MAT_2 results shows two different behaviors of the horizontal deformations, that can be related to the different bonds adopted. Wallettes built following the Dutch bond show smaller horizontal deformation and Poisson modulus than wallettes built using English bond.

3.4 In-plane flexural strength of masonry

In-plane flexural tests on masonry wallettes were performed at the DICAr Laboratory of the University of Pavia in order to evaluate the in-plane flexural strength of masonry elements, according to the European Norm EN 1052-2.

The specimens were built adopting the same materials and the Dutch construction technique used to build the test-house and the full-scale piers.

A series of tests on six masonry wallettes were performed. The specimens are named EC_MAT_IP2 plus an ID number, their geometry and static scheme of the test are shown in Figure 33.

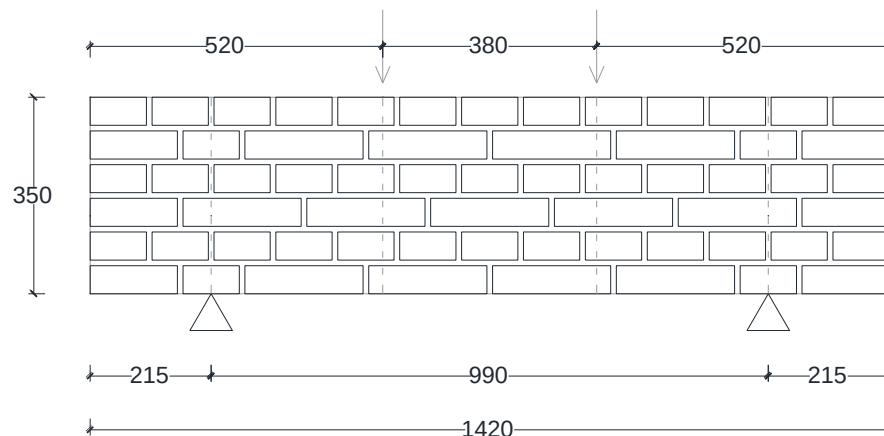


Figure 33. Geometry of the specimen and scheme of the in-plane flexural test.

The date of construction of the masonry wallettes was 10/03/2016. The in-plane flexural tests were performed from 03th May 2016 to 01th July 2016. The wallettes were composed by 6 layers of bricks.

The instrumentation on each side of the specimens included five vertical potentiometers to monitor the deflection in the mid-span and the curvature, other two potentiometers were placed in correspondence of the supports in order to check possible displacements. Two horizontal potentiometers were also located in the lower course of brick of the specimen.

The instrumentation scheme is shown in Figure 34. The LVDTs are numerated from 0 to 8 for one side and from 9 to 17 for the other side following the same numeration order.

The geometry of the specimen and distances between loading points and between loading and support points were chosen according to the EN 1052-2.

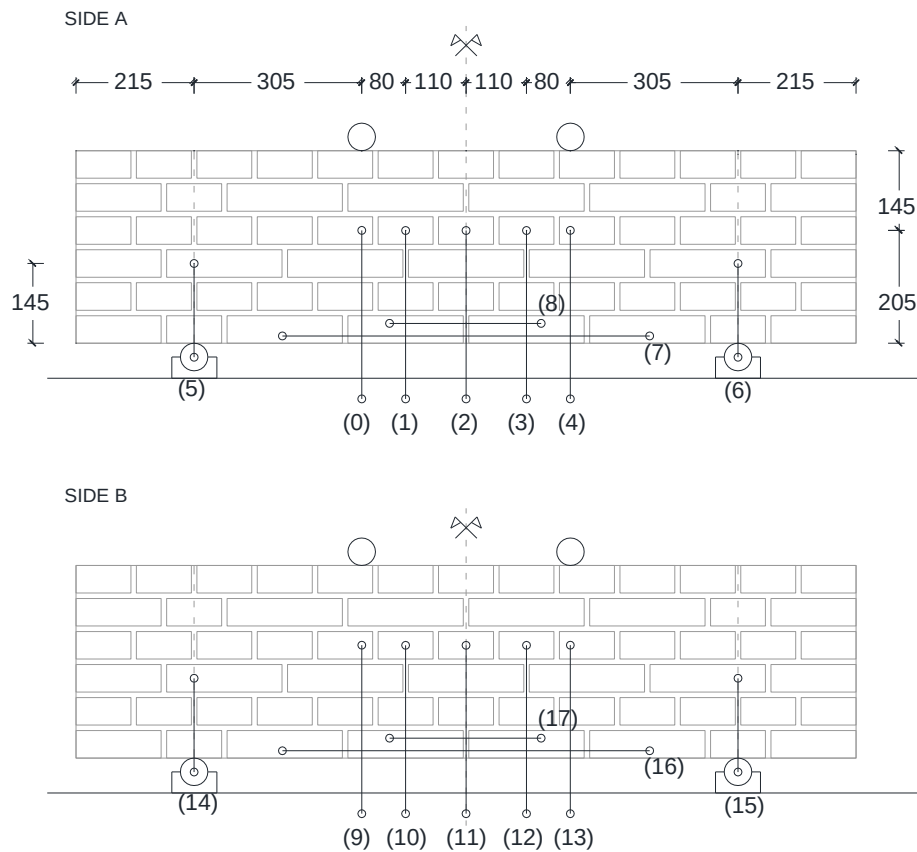


Figure 34. Instrumentation of the in-plane flexural test.

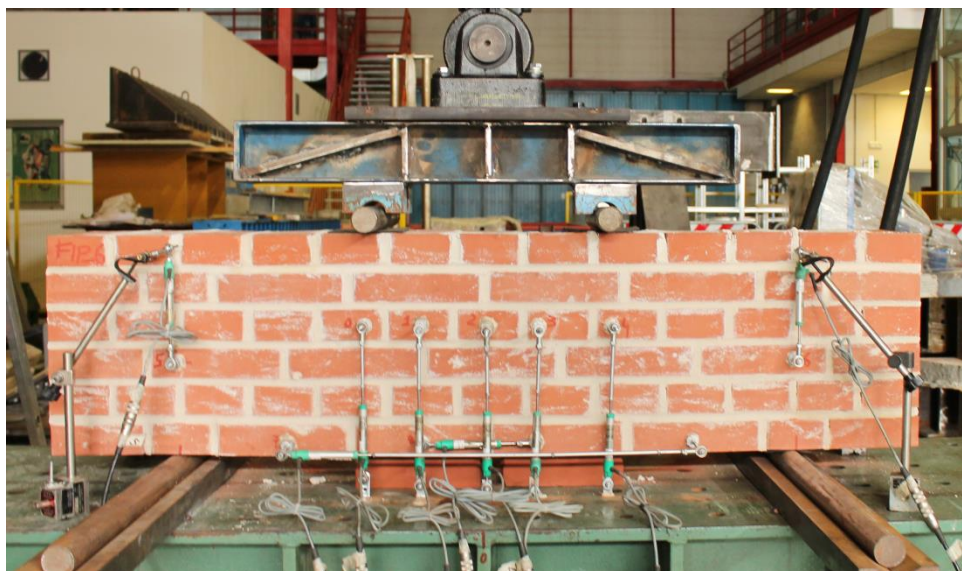


Figure 35. Set up of the in-plane flexural test.

Experimental Procedure

The masonry specimens were subjected to a four-points bending test. The failure was reached due to a vertical force applied on the top through two rollers welded to a steel profile (see Figure 35). In order to reduce imperfections due to the slight irregularity of the top side of the specimen, rubber strips were placed between the masonry specimen and the rollers.

The loading protocol consisted on a monotonic load in force control for four specimen and in displacement control for the other two. The loading velocity is consistent with UNI EN-1052-2 prescriptions of 0.03-0.3 MPa/min (in terms of f_{x3}) for the force-control, or 0.3 mm/min (in terms of midspan displacement) for the displacement control ones.

Test Results

The tests allow to measure the failure load and evaluate the flexural strength of the masonry as:

$$f_{x3} = \frac{M_{max}}{W} = \frac{(\frac{F_{fail}}{2} a + pl^2/8)}{W}$$

where M_{max} is the maximum bending moment, F_{fail} is the maximum total load at failure, a (=305 mm) is the distance between the support and the loading roller, l (=990 mm) is the distance between supports, W is the section modulus, p (=1.4 N/mm) is the masonry self-weight uniform load (assuming density=1980 kg/m³).

Values of f_{x3} obtained from the tests are listed in Table 14.

Table 14. In-plane flexural strength results.

Specimen	f_{x3} [MPa]
EC_MAT_IP2_1	0.50
EC_MAT_IP2_2	0.52
EC_MAT_IP2_3	0.39
EC_MAT_IP2_4 ^u	0.53
EC_MAT_IP2_5 ^{*u}	0.39
EC_MAT_IP2_6 [*]	0.33
Mean	0.44
St. Dev.	0.08
C.o.V.	19.1%

^u: Specimen tested upside down.

^{*}: Specimen tested in displacement control.

EC_MAT_IP2_4 and 5 have been tested turned upside down in order to investigate the influence of the pattern in the determination of the flexural strength, no differences were found between the two cases.

Figure 36 shows the force-displacement diagram for the mid-span point for each force control performed test. Figure 37 shows the force-displacement diagram for the mid-span point for each displacement control performed test. The mid-span vertical displacement has been linearly interpolated from the reading of LVDT 2 and 11.

The diagram concerning the test on the EC_MAT_IP2_4 specimen is not available because of a problem with the displacement acquisition; however, as there were no problem with the force acquisition, the flexural strength value was normally evaluated (Table 14).

Figure 38 shows the failures of the specimens.

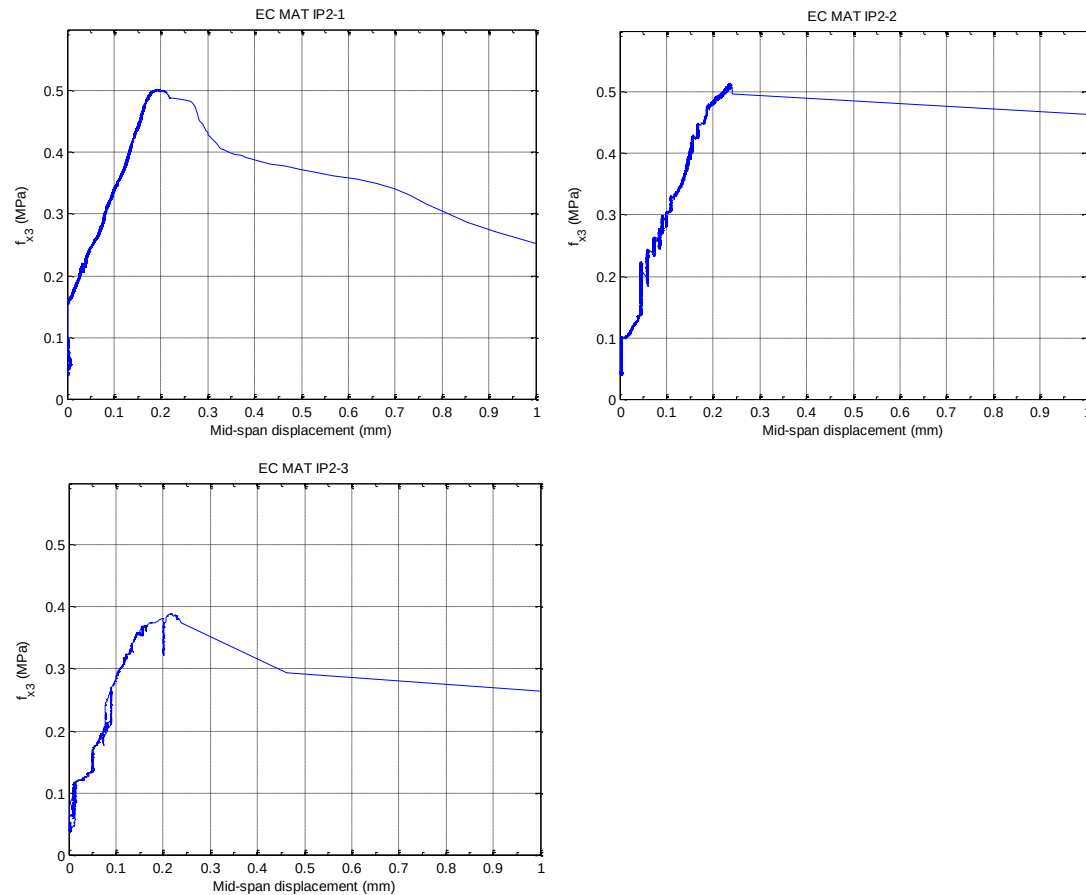


Figure 36. Flexural strength-displacement curve for specimens EC_MAT_IP2_1 to 3 (force control)

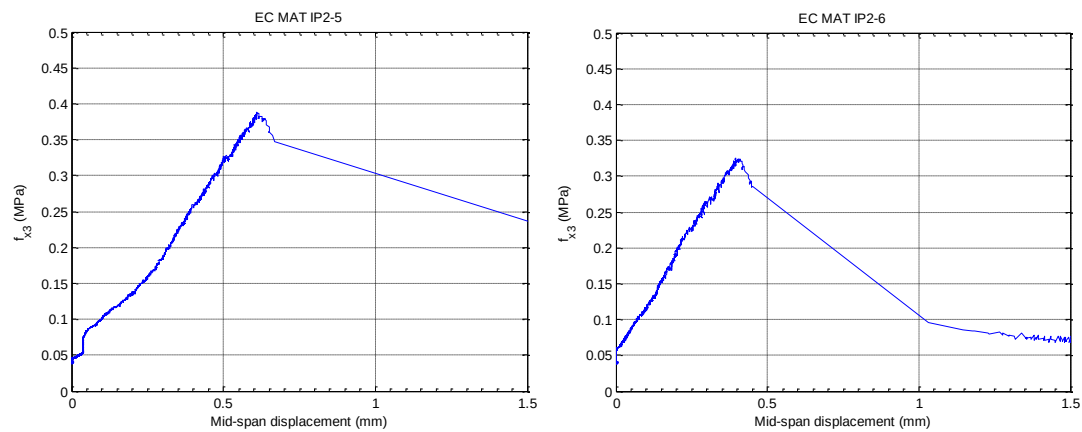


Figure 37. Flexural strength-displacement curve for specimens EC_MAT_IP2_5 & 6 (displacement control)

Notice the influence of the test control on the results: displacement control tests in tend to have a lower value of flexural strength and a peak moved to the right compared to the force control tests.

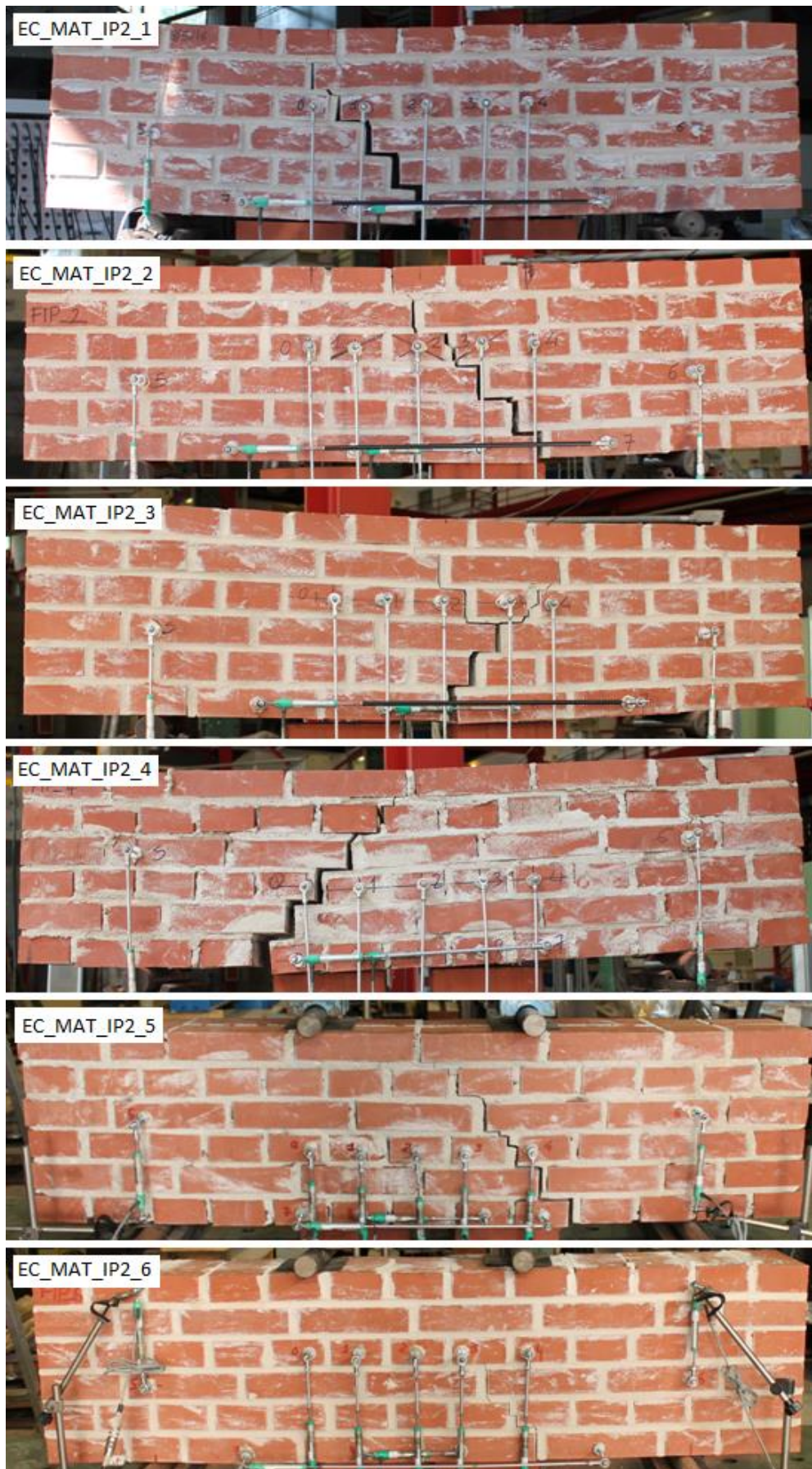


Figure 38. Crack pattern after failure for each specimen.

3.5 Out-of-plane flexural strength of masonry for a plane of failure perpendicular to the bed joints

Out-of-plane flexural tests on masonry wallettes were performed at the DICAr Laboratory of the University of Pavia in order to evaluate the out-of-plane flexural strength of masonry elements for a plane of failure perpendicular to the bed joints, according the European Norm EN 1052-2.

The specimens were built adopting the same materials and the Dutch construction technique used to build the test-house and the full-scale piers.

A series of tests on five masonry wallettes were performed in order to evaluate the flexural strength for a plane of failure perpendicular to the bed-joints. The specimens are named EC_MAT_OOP2 plus a an ID number.

The instrumentation on each side of the specimens included five vertical potentiometers to control the deflection in the mid-span and the curvature, plus two more potentiometers in correspondence of the supports in order to check possible displacements. Horizontal potentiometers were adopted on the two sides of the specimen.

Geometry, static scheme and instrumentation scheme of the test are shown in Figure 39. The LVDTs are numerated form 0 to 7 for one side and from 8 to 15 for the other side following the same numeration order.

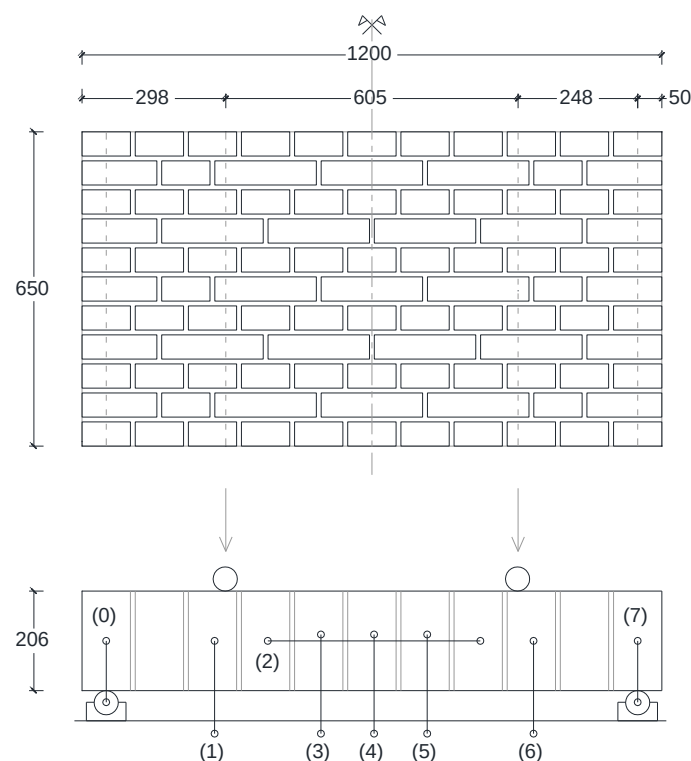


Figure 39. Instrumentation of the out-of-plane flexural test.

The date of construction of the masonry wallettes was 10/03/2016. The in-plane flexural tests were performed from 04th July 2016 to 06th July 2016.

The wallettes were composed by 11 layers of bricks. The geometry of the specimen and distances between loading points and between loading and support points were chosen according to the EN 1052-2.

Experimental Procedure

The masonry specimen were subjected to a four-points out-of-plane bending test. The failure were reached due to a vertical force applied on the upper side through two rollers welded to a steel profile (see Figure 40). In order to reduce imperfections due to the slight irregularity of the top side of the specimen, gypsum capped strips have been made between the masonry specimen and the rollers.

The loading protocol consisted on a monotonic load in displacement control. The loading velocity of 0.005 mm/s (in terms of midspan displacement) is consistent with UNI EN-1052-2 prescriptions of 0.03-0.3 MPa/min.

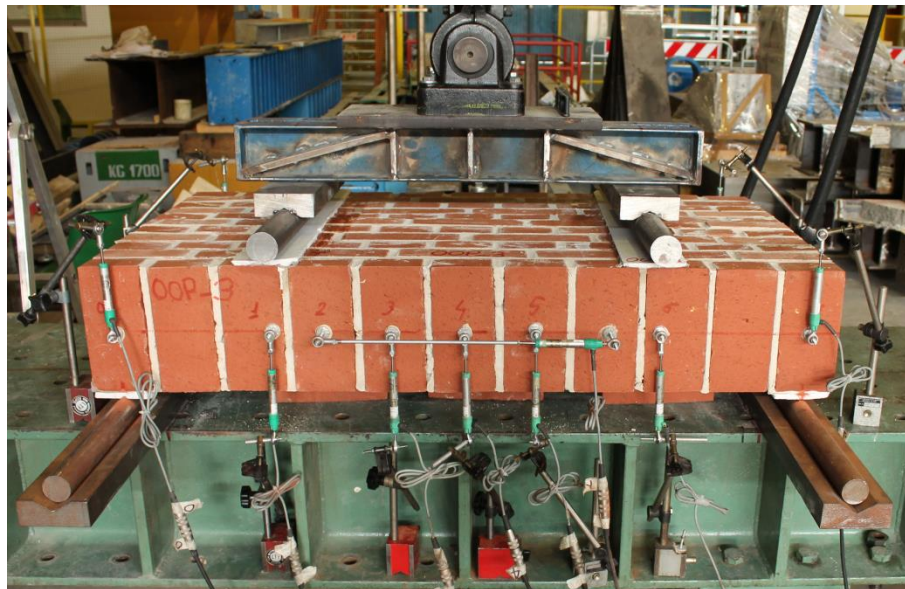


Figure 40. Set up of the in-plane flexural test.

Test Results

The tests allow to measure the failure load and evaluate the out-of-plane flexural strength of the masonry as:

$$f_{x2} = \frac{M_{max}}{W} = \frac{(\frac{F_{fail}}{2} a + pl^2/8)}{W}$$

where M_{max} is the maximum bending moment, F_{fail} is the maximum total load at failure, a (=298 mm) is the support-loading distance, l (=1100 mm) is the distance between supports, W is the section modulus, p (=1.4 N/mm) is the masonry self-weight uniform load (assuming density=1980 kg/m³).

Values of f_{x2} obtained from the tests are listed in Table 15.

Table 15. Out-of-plane flexural strength results.

<i>Specimen</i>	<i>f_{x2} [MPa]</i>
EC_MAT_OOP2_1*	0.63
EC_MAT_OOP2_2	0.71
EC_MAT_OOP2_3	0.52
EC_MAT_OOP2_4	0.61
EC_MAT_OOP2_5	0.73
<i>Mean</i>	0.64
<i>St. Dev.</i>	0.10
<i>C.o.V.</i>	15.1%

*Results from the test on the specimen EC_MAT_OOP2_1 were not counted in the mean and st.dev. calculations since its failure is not consistent with UNI EN-1052-2. See Figure 43.

Figure 41 shows the force-displacement diagram for the mid-span point for each performed test. The mid-span vertical displacement has been linearly interpolated from the reading of LVDT 4 and 12.

Figure 42 and Figure 43 show the failures of the specimens. Note that the failure of the specimen EC_MAT_OOP2_1 has not taken place within the length between the supports.

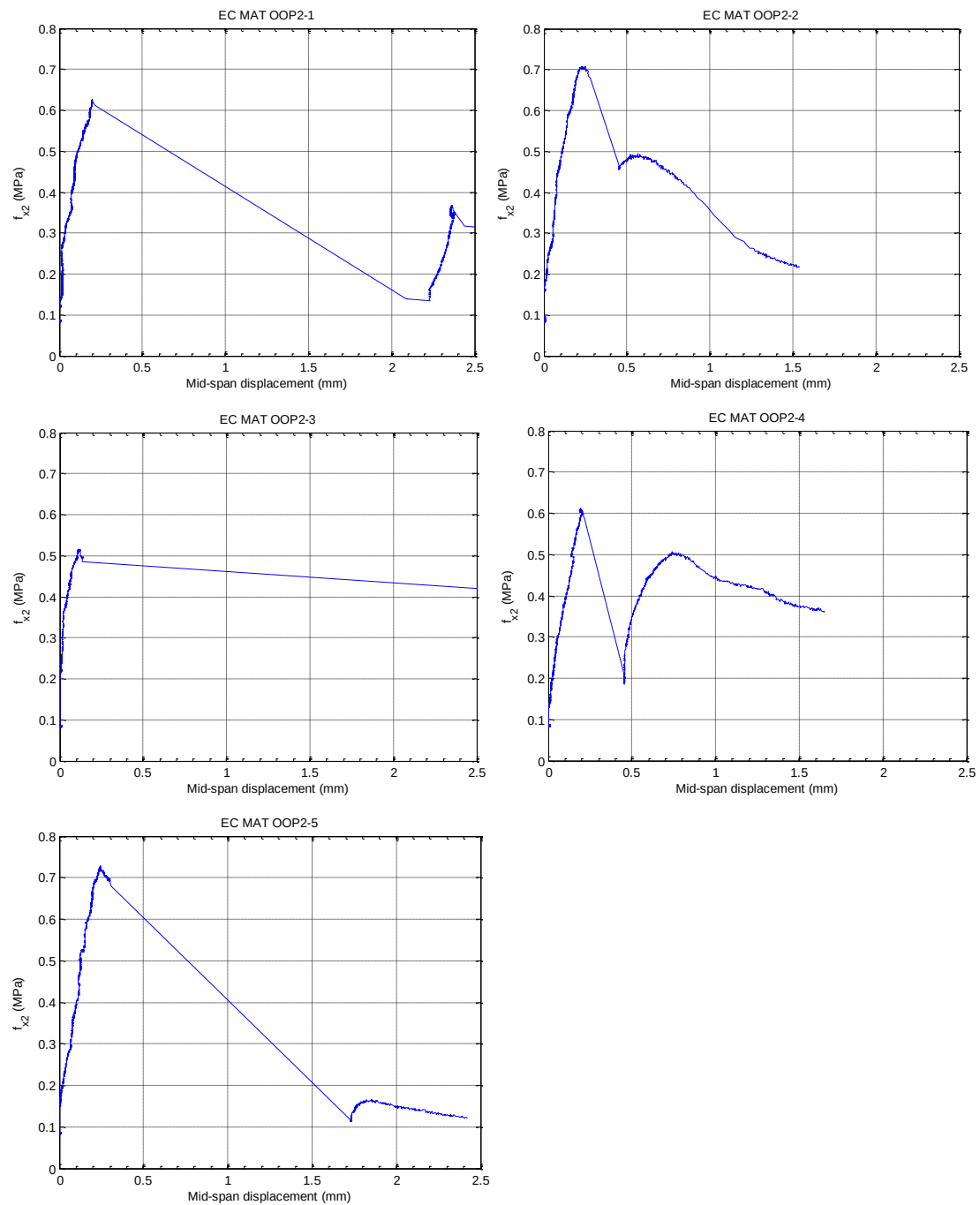


Figure 41. Flexural strength-displacement curve for specimens EC_MAT_OOP2_1to5



Figure 42. Crack pattern after failure for specimens EC_MAT_OOP2-2 to 5



Figure 43. Crack pattern after failure of the specimen EC_MAT_OOP2_1

3.6 Bond strength of masonry

A series of bond wrench test on 18 masonry specimens was performed in order to characterize the bond strength of the horizontal bed-joint of mortar. The specimens are formed by two masonry bricks connected by a bed-joint, and it is subjected to a bending moment. The geometry of the specimens is shown in Figure 44.

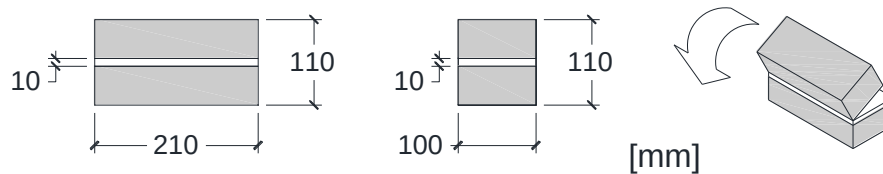


Figure 44. Geometry of the specimens.

The reference code for the present experimental test is the European Norm EN 1052-5 of October 2005: “Determination of bond strength by the bond wrench method”. The tests were performed at the DICAr Laboratory of the University of Pavia (Italy).

Figure 45 shows the test layout: the test brick is subjected to a bending moment and a compressive force, due to the two forces F_1 and F_2 , while the rest of the specimen is clamped.

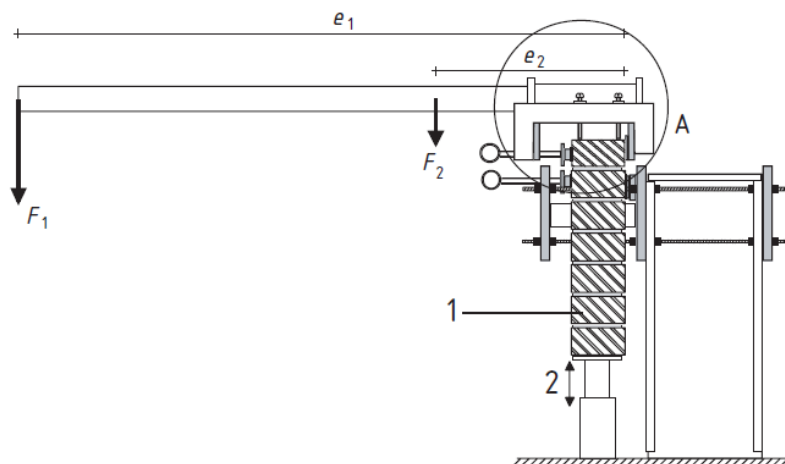


Figure 45. Layout of the Bond Wrench Test (EN 1052-5)

The list of symbols according to EN 1052-5 is reported below:

- h height of the specimen
- e_1 distance from the applied load to the tension face of the specimen in mm
- e_2 distance from the centre of gravity of the lever and upper clamp to the tension face of the specimen in mm

Note that in the present case e_2 is equal to half width of the brick because the upper clamp is perfectly balanced, i.e. its centre of mass is vertically aligned with the centre of mass of the bricks.

W weight of masonry unit pulled of the specimen and any adherent mortar

F_1 applied load

F_2 weight of the bond wrench

f_w bond strength

d mean depth of the specimen

b mean width of the bed joint tested

Z section modulus of the projected plan area of the failure surface

Figure 46 represents the acceptable types of bed-joint failure, according to EN 1052-5, observed during the tests in the DICAr Laboratory.

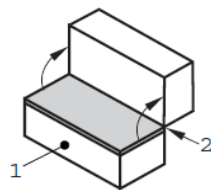


Figure A.1 — Failure at interface between mortar and upper unit

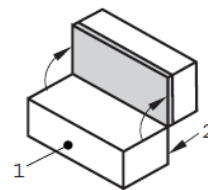


Figure A.2 — Failure at interface between mortar and lower unit

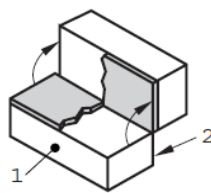


Figure A.3 — Failure at interface between mortar and both units

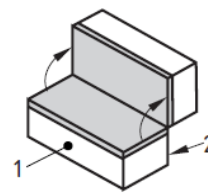


Figure A.4 — Tension failure within mortar bed

Figure 46. Type of acceptable failure observed during the tests.

Test Results

Table 16 shows the geometric properties of the bond wrench test set up and of the tested specimens, using EN 1052-5 nomenclature. The specimens are named BW plus a number from 1 to 18.

Tests were performed on the dates 28/04/2016, 29/04/2016 and on 02/05/2016.

Table 16. Geometric properties of the specimens.

		BW specimens
h	[mm]	110
b	[mm]	207
d	[mm]	100
e ₁	[mm]	345
e ₂	[mm]	50
Z	[mm ³]	345000
F ₂	[N]	30.5

Table 17 and Figure 47 show the results of the bond wrench tests and the mean value of bond strength.

Table 17. Bond wrench tests results

Specimen	W	F1	M	f_{wi}	Failure mode
	[N]	[N]	[Nm]	[MPa]	
BW_1	23.6	278.6	82.2	0.222	A2
BW_2	20.9	57.8	17.1	0.044	A1
BW_3	24.5	104.8	30.9	0.082	A2
BW_4	19.7	38.5	11.3	0.029	A1
BW_5	24.3	174.8	51.6	0.138	A2
BW_6	23.6	343.0	101.2	0.274	A2
BW_7	24.0	337.7	99.6	0.270	A2
BW_8	23.9	302.5	89.2	0.241	A2
BW_9	20.3	330.6	97.5	0.264	A1
BW_10	20.2	347.0	102.4	0.278	A1
BW_11	19.6	430.5	127.0	0.345	A1
BW_12	24.6	366.7	108.2	0.293	A2
BW_13	23.3	334.9	98.8	0.268	A2
BW_14	19.6	314.9	92.9	0.252	A1
BW_15	24.2	260.9	77.0	0.208	A2
BW_16	29.0	160.1	47.2	0.126	A2
BW_17	24.5	254.4	75.0	0.203	A2
BW_18	20.8	821.0	242.2	0.660	A3/A4
Mean			86.2	0.233	
St. Dev.			50.8	0.139	
C.o.V.			59%	60%	

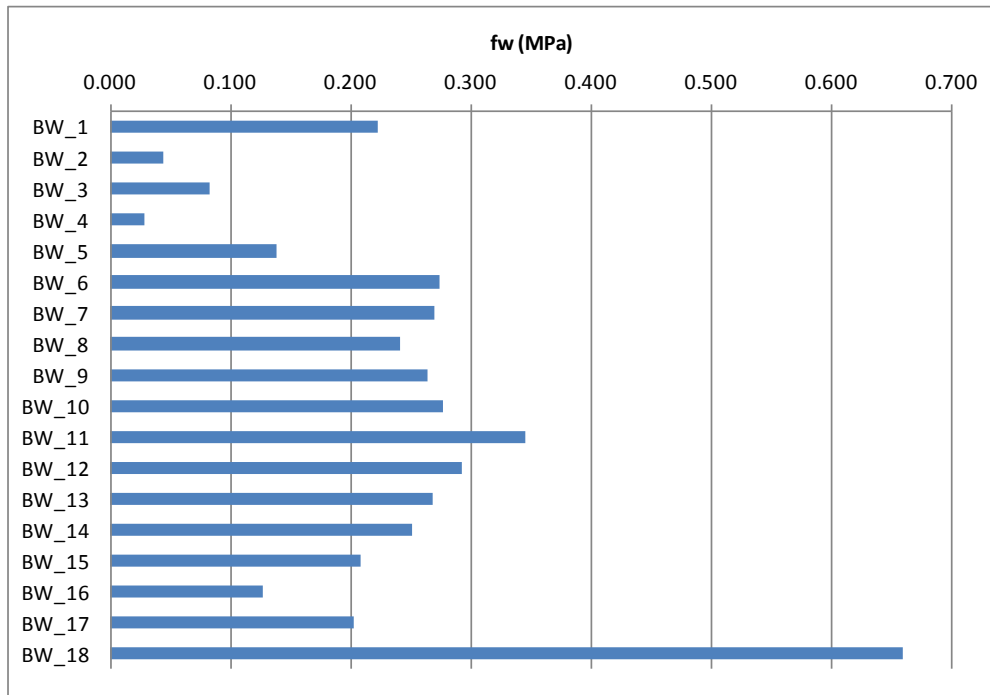


Figure 47. Bond wrench tests results.

All the specimen broke according to the A1 or A2 type of failure described in EN 1052-5 (see Figure 46), except the specimen BW_18 that produced the greater bond strength and it failed following an A3\A4 intermediate failure type, as shown in Figure 50.



Figure 48. View of the specimens of the first series.



Figure 49. Set up of the test.



Figure 50. Result of the test on the specimen BW_tri_2_B.

3.7 Shear strength of masonry

The purpose of the test is to evaluate the shear resistance of the two horizontal bed-joints relative to a single unit block of the masonry.

The tests were performed at the DICAr Laboratory of the University of Pavia in Italy from 12/05/2016 to 24/05/2016.

The reference code for the present experimental work is the European Norm EN 1052-3:2002 and its amendment EN 1052-3:2002/A1:2007.

The specimens type chosen according to the Norms is the type “T”: three bricks bonded with two layers of mortar made using the same materials of the in-plane and out of plane tested walls and the full-scale test-house.

The procedure "A" is the one adopted: at least three specimens must be tested at three different level of pre-compression.

The scheme of the specimen is shown in Figure 51. The dimensions of the bricks and specimens are represented in Figure 52.

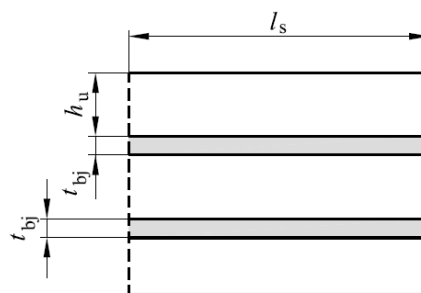


Figure 51. Specimen dimensions of Triplet Test (EN 1052-3:2002).



Figure 52. Brick and specimen dimensions.

The length of the bricks is smaller than 300 mm, hence the length of the specimen is exactly equal to the length of the brick according to EN 1052-3.

The mortar bed-joints are 10 mm thick.

Experimental Procedure

The tests were conducted in displacement-control, their procedure is summarized in the following list:

1. Two layers of gypsum are applied on the external faces of the specimens;
2. The specimen is positioned and instrumented in the testing apparatus, between two steel plates, while it is supported by roller bearings;
3. Apply a compression, F_p at the two lateral faces of the specimen, as shown in Figure 53;

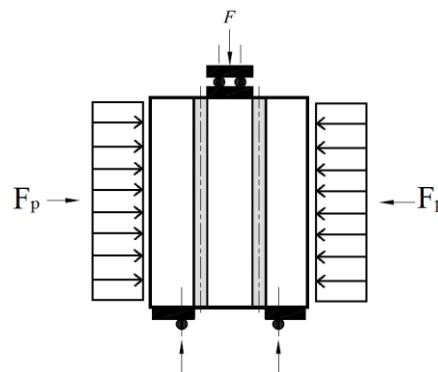


Figure 53. Triplet Test, load application.

4. For elements that have a compressive resistance greater than 10 MPa, as in this case, the compressive load F_p must impose to the specimen a stress value of about 0.2 MPa, 0.6 MPa and 1.0 MPa. The compressive load, uniformly distributed on the faces of the specimen, is maintained constant during the test, even if the triplet specimen tend to dilate during the test, due to the dilatancy phenomenon usually associated to this type of masonry (or more in general to the Coulomb mechanism). In order to accommodate displacement, a spring ($k=50$ N/mm) is positioned in series with the horizontal force.
5. A shear force F is applied vertically on the specimen with a velocity of 0.36 mm/min which respect the EN 1052-3 limits of between 0.1 MPa/min and 0.4 MPa/min;
6. Measured quantities and data captured:
 - the area of the transversal section A_i , of the specimens, parallel to the shear force
 - the evolution of the load applied $F_{i,max}$
 - the evolution of the displacement applied (controlled)
 - the evolution of the compressive load F_{pi}
 - the displacements 0,1,2,3,4,5 by using LVDTs, as represented in Figure 54

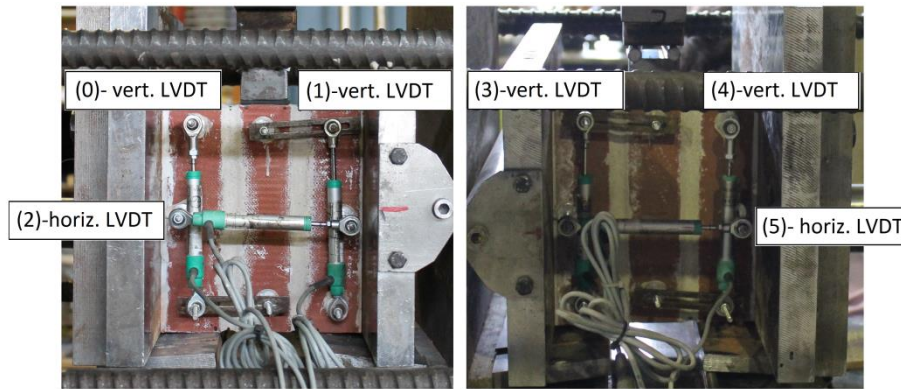


Figure 54. Triplet Test, LVDTs positions.

7. If the specimen was not damaged too much, another test was performed on the same triplet at a different level of lateral compressive load, with the bed-joints already cracked. These data were recorded in order to evaluate the shear strength of the mortar bed-joints in the case in which only the friction contribution is acting, and not the cohesive strength.

The shear strength, f_{vi} , of a specimen i , for a given compressive stress σ_i (imposed by the lateral load on the specimen, F_{pi}), is equal to:

$$f_{vi} = \frac{F_{i,max}}{2A_i}$$

with:

$$\sigma_i = \frac{F_{pi}}{A_i}$$

$F_{i,max}$ maximum shear force

F_{pi} compressive lateral load

A_i transversal area of the specimens, parallel to the bed-joints

For each value of pressure F_{pi} , and then σ_i , there is a corresponding value of f_{vi} .

By plotting the couples (σ_i, f_{vi}) it is possible to obtain a graph like the one shown as example in Figure 55.

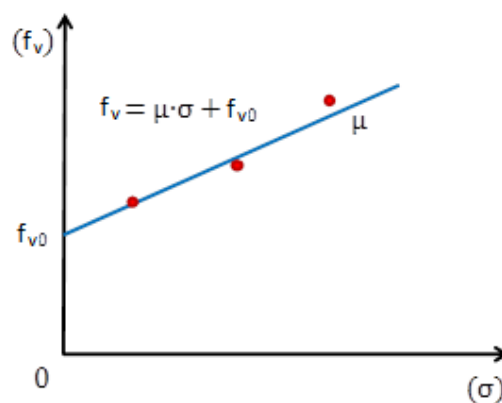


Figure 55. Triplet Test, σ - f_v strength envelope.

Coulomb's law resulted to be the more representative of the results. The shear strength of the mortar bed-joints of the specimens (f_v) depends on three parameters: the cohesion, the friction coefficient and the transversal compression. The cohesion contributes to the strength only if the mortar bed-joints are not cracked, while the friction force acts also after the formations of the fractures, till there is contact between the two materials. The shear strength (f_v), according to Coulomb's law, is linear depending on the compression (σ):

$$f_v = f_{v0} + \mu \cdot \sigma$$

with:

f_{v0} cohesion (or adhesion stress)

μ coefficient of friction

Test Results

The tests have been performed on 12 triplets, show in Figure 56, built in the DICAr laboratory using the same materials and Dutch construction techniques of the in-plane and out-of-plane test walls and the full-scale test house.



Figure 56. View of the specimens for triplet test.

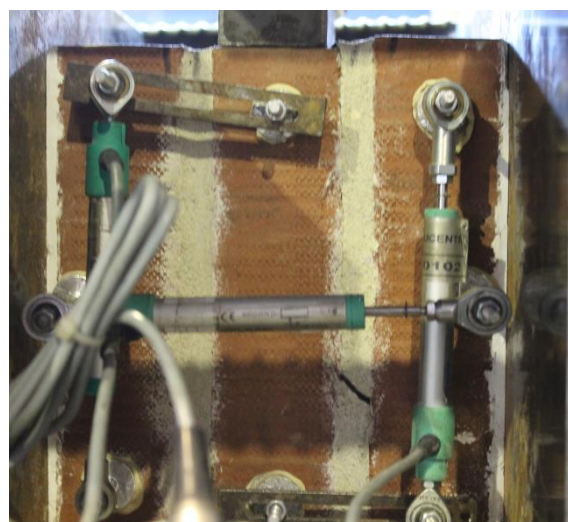


Figure 57. Test on specimen #3. Failure of the right bed-joint.



Figure 58. Tested specimen #2.

Performing the test, the contribution to shear strength is given by both cohesion and friction force as long as the specimen is intact. After the bed-joint fails, cohesion is no longer acting and the only one contribution is provided by the friction inside the failure surface, therefore useful information are also obtained after failure.

Figure 60 shows the results of the tests. The square-dots represent all the couples (σ_1 , f_{v1}) relative to the maximum value of shear strength of mortar bed-joints reached in correspondence of failure for each performed test. The cross-dots represent all the couples (σ , f_v) collected after failure, that suggest the values of residual contribution due to the only friction.

The black continuous line represents the failure couples weighted linear regression, the black dashed line represents the residual couples linear regression.

Data in terms of shear strength f_v were obtained from each test as shown in Figure 59 for the case of the specimen 5.

The values of the (σ_1 , f_{v1}) couples are shown in Table 18.

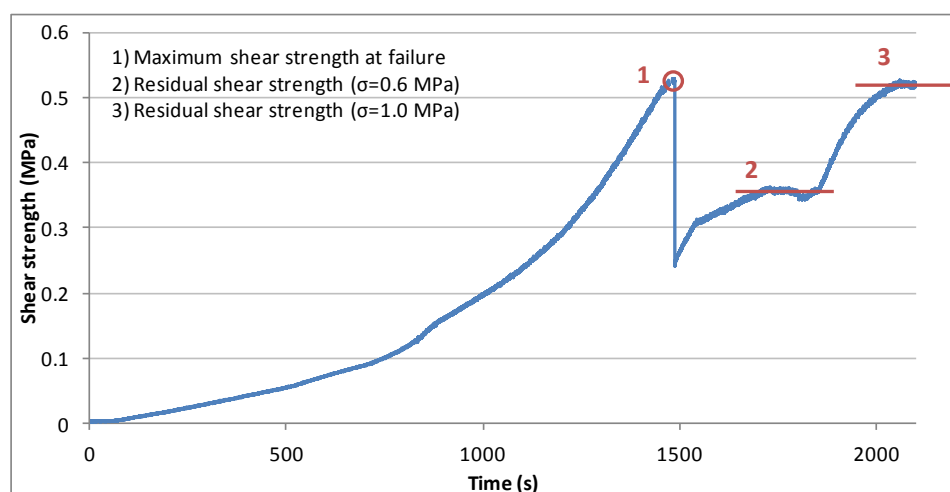


Figure 59. Identification of useful data from f_v results of specimen #5.

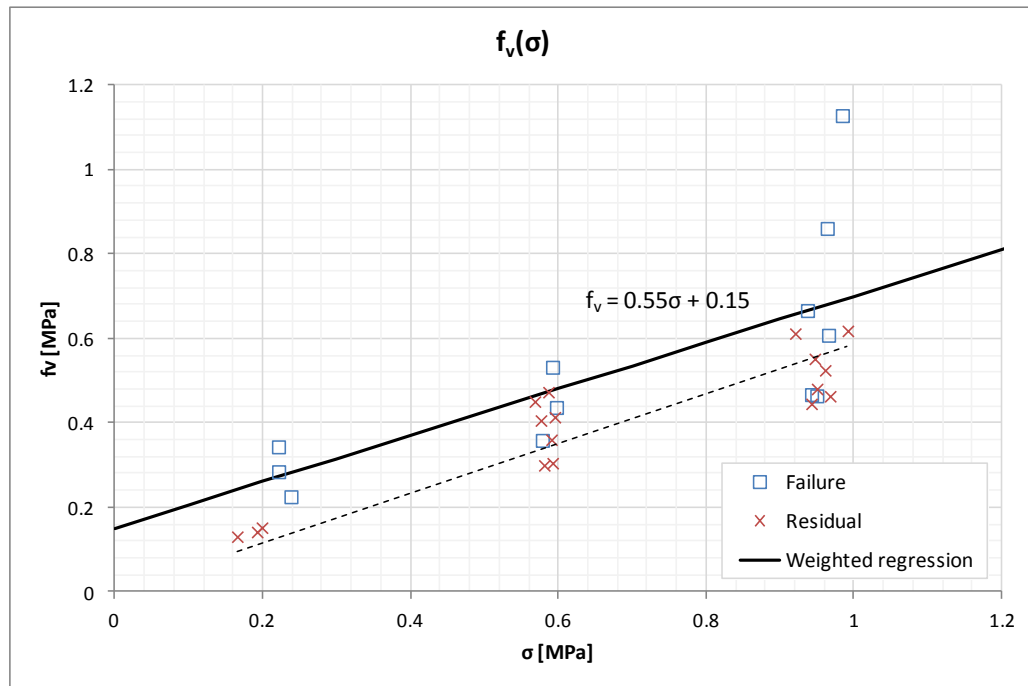


Figure 60. Triplet test results, failure and residual strength envelopes.

Table 18. Triplet test results, (σ_1, f_{v1}) couples.

Specimen	σ [MPa]	$f_{v,1}$ [MPa]
T1	0.22	0.28
T2	0.24	0.22
T3	0.22	0.34
T4	0.60	0.44
T5	0.59	0.53
T6	0.58	0.36
T7	0.95	0.46
T8	0.94	0.67
T9	0.94	0.47
T10	0.97	0.61
T11	0.96	0.86
T12	0.98	1.13

Analyzing each triplet result singularly, a great variation of the cohesion between a lower value of 0.03 MPa and a higher value of almost 0.5 MPa was noticed.

Since 5 tests have been performed at a value of σ of about 1.0 MPa, as shown in Figure 60, the proposed shear strength model is obtained from the linear regression of the couples (σ, f_v) which represent the means for each level of compression σ (0.2; 0.6 and 1.0 MPa), in order to confer the same weight to each series of test.



The defined model determines the values of the cohesion f_{v0} and of the friction μ , respectively as the Y-intercept and the slope of the linear regression line:

$$\text{Cohesion: } f_{v0} = 0.15 \text{ [MPa]}$$

$$\text{Friction: } \mu = 0.55 \text{ [-]}$$

3.8 Summary table

<i>Material property</i>	<i>Symbol</i>	<i>UM</i>	<i>Mean</i>	<i>C.o.V.</i>
Density	ρ	[kg/m ³]	1979	0.01
Compressive strength of mortar	f_c	[MPa]	4.12	0.24
Tensile strength of mortar	f_t	[MPa]	1.20	0.33
Compressive strength of masonry unit	f_b	[MPa]	46.8	0.11
Flexural strength of masonry unit	$f_{b,t}$	[MPa]	8.5	0.05
Compressive strength of masonry in the direction perpendicular to bed joints	f_m	[MPa]	11.22 ^[1]	0.07
Elastic modulus of masonry in the direction perpendicular to bed joints (33% f_m)	E_1	[MPa]	9833 ^[1]	0.25
Elastic modulus of masonry in the direction perpendicular to bed joints (10% f_m)	E_2	[MPa]	13023 ^[1]	0.39
Elastic modulus of masonry in the direction perpendicular to bed joints given by the secant line passing through the points (33% f_m , $\epsilon_{33\% f_m}$) and (10% f_m , $\epsilon_{10\% f_m}$)	E_3	[MPa]	8905 ^[1]	0.24
Flexural bond strength	f_w	[MPa]	0.23	0.60
In-plane flexural strength of masonry	f_{x3}	[MPa]	0.44	0.19
Out-of-plane flexural strength of masonry	f_{x2}	[MPa]	0.64	0.15
Masonry (bed joint) initial shear strength	f_{v0}	[MPa]	0.15	-
Masonry (bed joint) shear friction coefficient	μ	[-]	0.55	-

^[1] Test on bigger specimens considered more reliable due to dimensions and mortar characteristics more similar to the full-scale specimens.



4 In-plane component tests

The experimental test program administrated by EUCENTRE and carried out at the Laboratory in Pavia, included five in-plane cyclic shear-compression tests on full scale clay unreinforced masonry walls under varied combinations of vertical compression, boundary conditions and slenderness ratios. The study of the cyclic in-plane behaviour of these piers will be fundamental in calibrating reliable numerical models in term of force-displacement response, as well as defining proper limit states.

4.1 Test Specimens

The tested specimens are:

- 3 slender walls tested under double-fixed boundary conditions, at different levels of vertical compression stress: 0.52 MPa, 1.2 MPa and 0.86 MPa, labeled as EC_COMP2_1, EC_COMP2_2 and EC_COMP2_3, respectively;
- 2 squat walls tested under double-fixed boundary conditions and mean vertical compression stress of 0.3 MPa, labelled as EC_COMP2_4 and EC_COMP2_5; in order to investigate the influence of the testing protocol on the results, for the last test (EC_COMP2_5) only one cycle for each target displacement has been performed.

The characteristics of the tested specimens are summarised in Table 19, Figure 61 and Figure 62.

Table 19. Characteristics of the tested specimens.

Specimen name	l [m]	t [m]	h [m]	σ_v [MPa]	Unit size ($l \times t \times h$) [mm]	# of bricks ($l \times t \times h$) [-]	Boundary conditions
EC_COMP2_1	1.2	0.208	2.71	0.52	208x100x50	5.5x2x45	Double-fixed
EC_COMP2_2	1.2	0.208	2.71	1.20	208x100x50	5.5x2x45	Double-fixed
EC_COMP2_3	1.2	0.208	2.71	0.86	208x100x50	5.5x2x45	Double-fixed
EC_COMP2_4	2.74	0.208	2.71	0.3	208x100x50	12x2x45	Double-fixed
EC_COMP2_5	2.74	0.208	2.71	0.3	208x100x50	12x2x45	Double-fixed

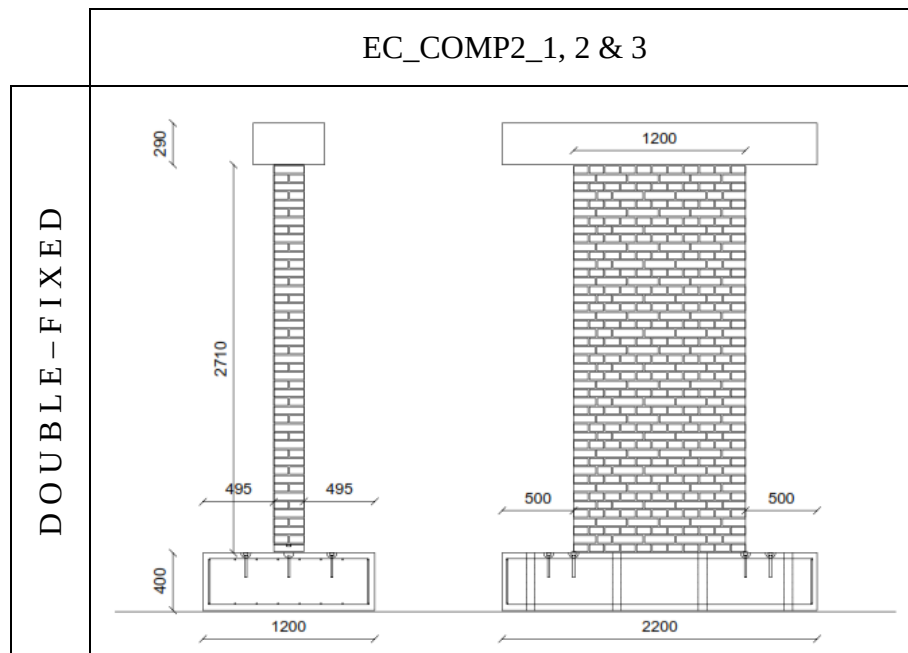


Figure 61. Geometry of the tested slender specimens.

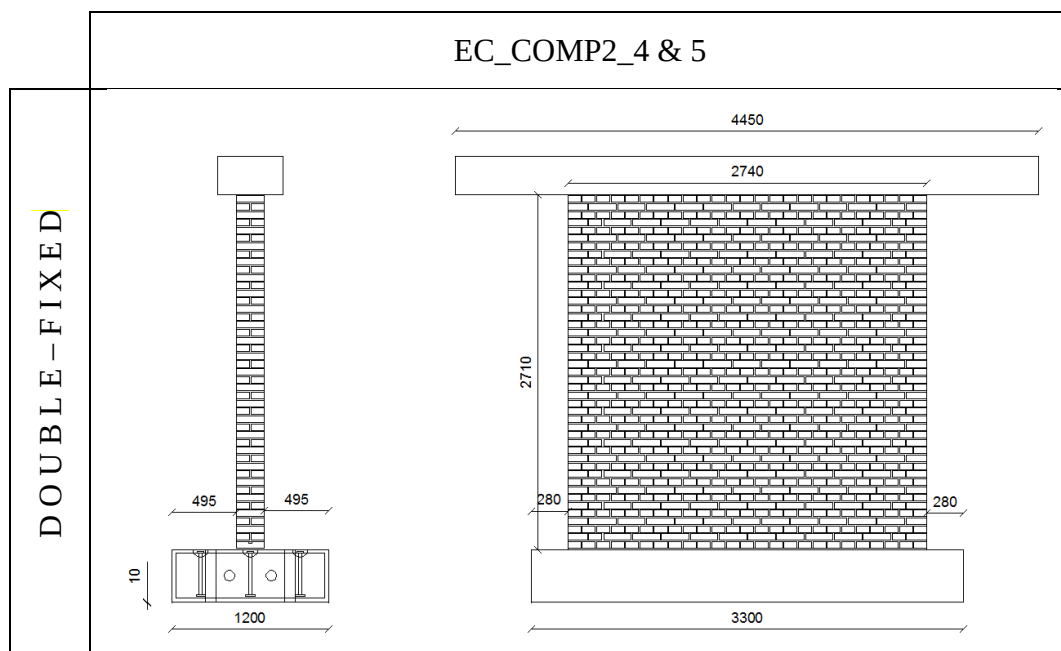


Figure 62. Geometry of the tested squat specimens.

The following pictures (Figure 63 and Figure 64) illustrate the two types of tested specimens during the construction phase took place on 1st and 2nd of March 2016.



Figure 63. Slender walls during their construction.



Figure 64. Squat walls during their construction.

4.2 Test Set-up and Instrumentation

4.2.1 Test set-up

The experimental set-up adopted for the in-plane component tests is illustrated in Figure 65 and Figure 66. In particular, it consists of:

- A horizontal servo-hydraulic actuator, able to apply a horizontal shear force at the top of the specimen equal to its maximum shear strength and a maximum displacement higher than 2 % of the height of the specimen; the actuator is fixed in order to guarantee a sufficiently rigid reaction in one end and it is connected to the top of the wall through a loading beam;
- Two vertical servo-hydraulic actuators able to apply simultaneously a force equal to the required vertical load and a moment corresponding to the maximum resisting moment at the top section; the actuators are fixed in order to guarantee a sufficiently rigid vertical reaction in one end and they are connected to the loading beam;

A restraining system to prevent out-of-plane deflection of the loading beam connected to the top of the specimen.

The horizontal servo-hydraulic actuator applies a horizontal shear force to the top of the wall through a composite steel/R.C. loading beam. The steel beam is stiffened with steel plates positioned orthogonally to the axis of the beam. A layer of self-levelling high strength shrinkage-controlled gypsum is placed between the steel/R.C. composite beam and the top of the masonry specimen, which can present an irregular surface, to allow a continuous and uniform support.

Two vertical servo-hydraulic actuators apply the vertical load on the wall, reacting on a steel frame fixed on a strong wall. The two vertical actuators allow to control the vertical load applied and to provide different boundary conditions (e.g. cantilever and double-fixed boundary conditions). The reinforced concrete footing is bolted to a strong floor. The horizontal actuator allows to apply horizontal forces and displacements according to the test protocol. The wall is restrained from out-of-plane deflections by a low-friction sliding restraining system.

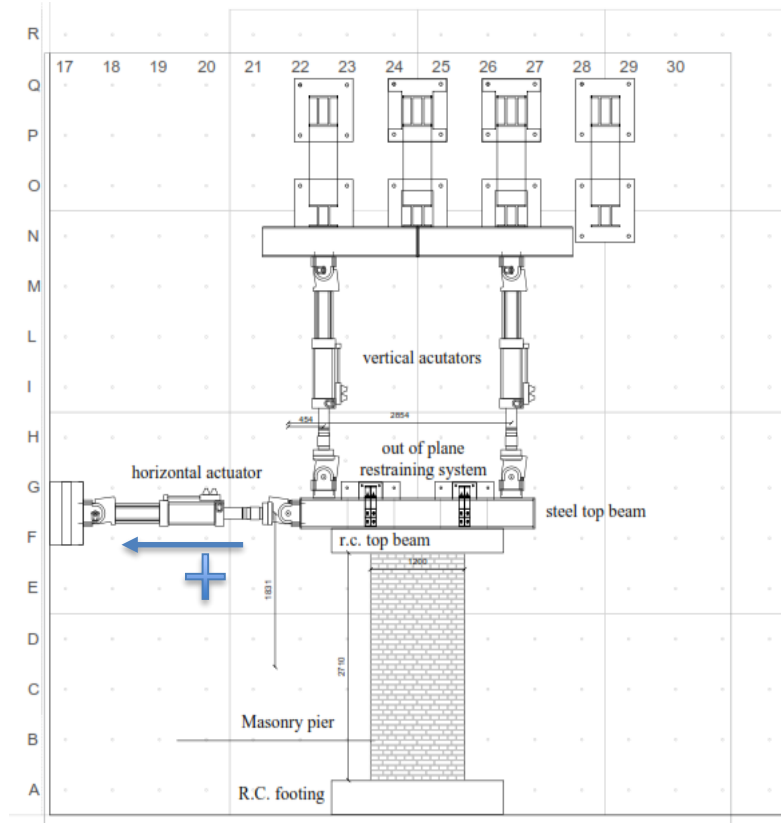


Figure 65. Test set-up for the slender piers.

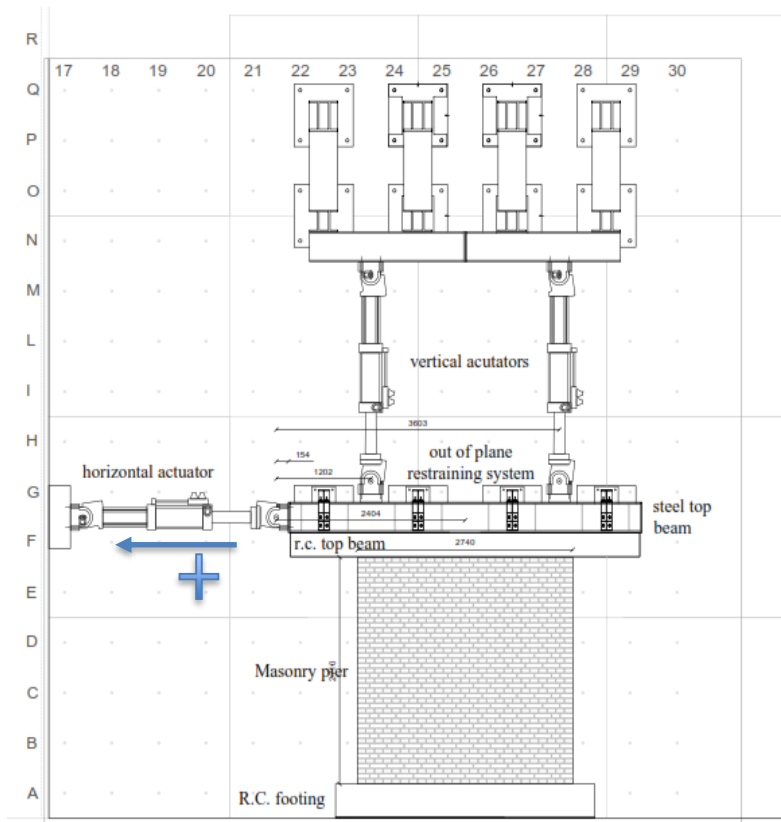


Figure 66. Test set-up for the squat piers.

4.2.1.1 Instrumentation

The instrumentation consists of a set of instruments recording the following quantities:

- forces in the actuators (horizontal and vertical forces applied to the top of the specimen);
- horizontal displacement of the beam on top of the specimen;
- vertical displacements of at least two different points at the top of the specimen (symmetric with respect to the central axis of the wall, in order to detect possible in-plane rotations);
- possible out-of-plane displacement of the wall, monitored by means of potentiometers;
- possible sliding between the various mechanical parts involved in the test, as well as possible sliding of the specimen with respect to the test apparatus (e.g. sliding of the top beam or sliding with respect to the foundation).

The instrumentation for all the specimens is reported in Figure 67 and Figure 68. All the displacement transducers are placed at middle stroke, fixed to the specimen with flat head nail anchored in the masonry and a plate with threaded rod is glued at the head of the nail. In this case, if the end scale of the transducer is reached, failure of the bonding of the two parts prevents the instrument from getting damaged. The instruments indicated with the letter “T” are spring-loaded potentiometers.

The initial distance between the measure bases, with eventual extension of the transducers, has been measured after the installation of the instruments (for details refer to the “Guidelines report” shared with the experimental raw data). Detailed info about the instrumentation and instructions regarding the use of the recordings are reported in Appendix-D.

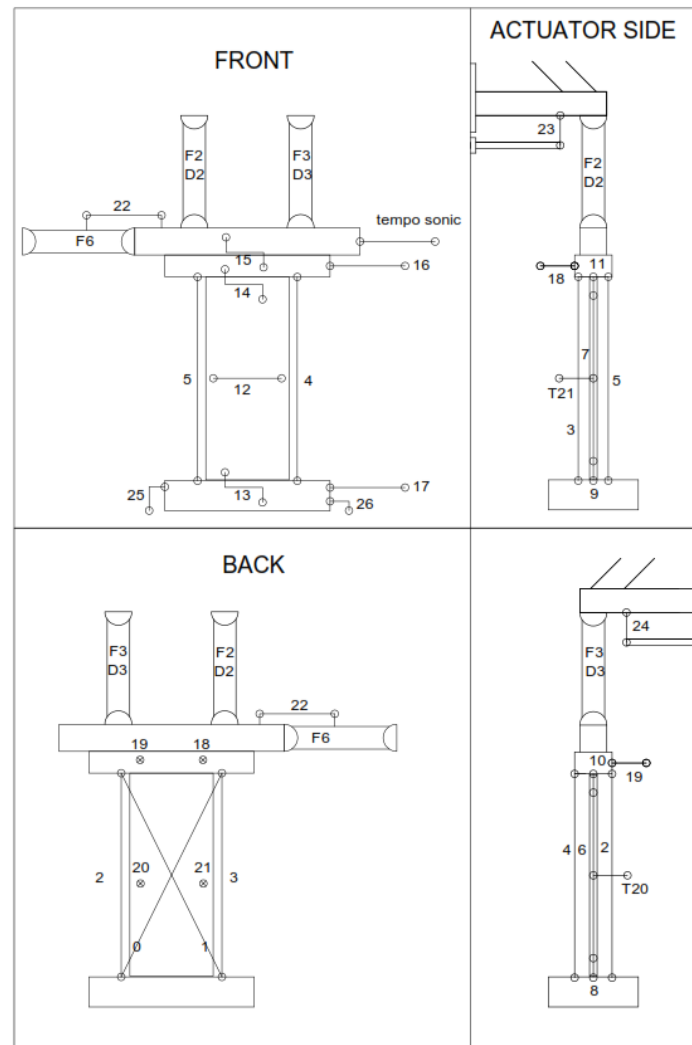


Figure 67. Location of the instruments mounted on the slender piers.

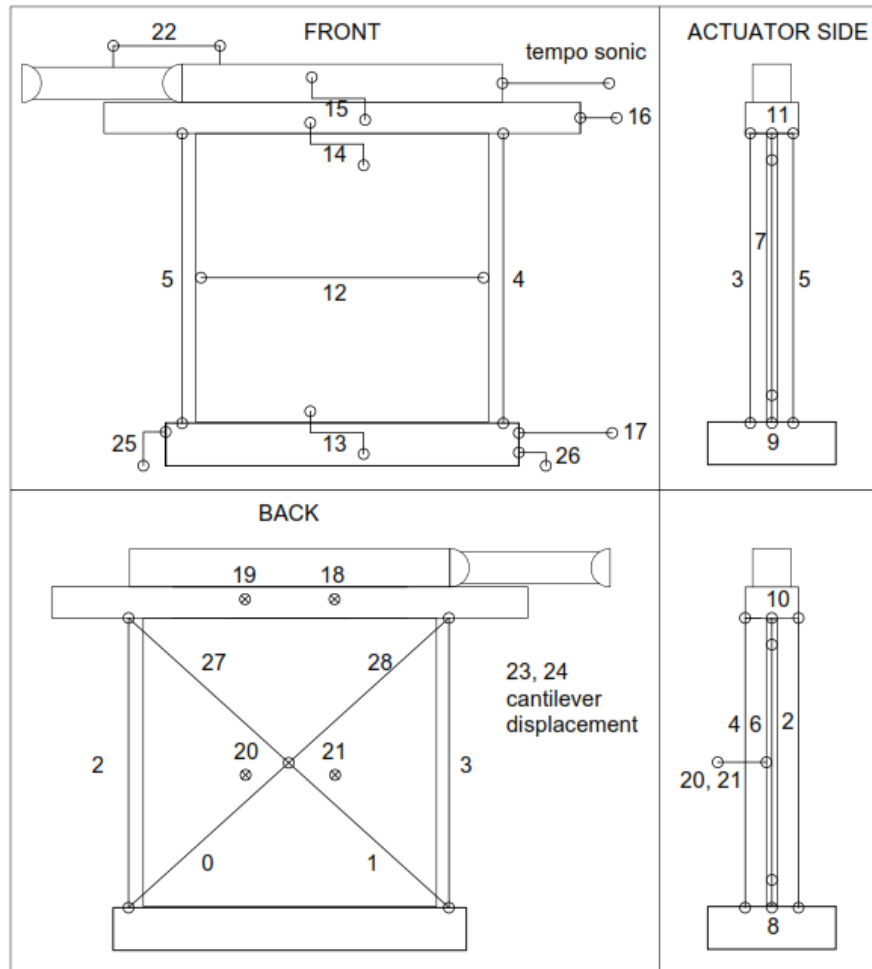


Figure 68. Location of the instruments mounted on the squat piers.

In addition to the traditional acquisition system, each specimen has been monitored by an optical acquisition system which consists of high resolution cameras able to follow and record the time history of the coordinates of several point of the panel. The 2D optical acquisition system has been adopted only for the first component test (EC_COMP2-1), while data coming from a more advanced 3D optical acquisition system has been available for the other four specimens.

4.3 Testing Procedure

The testing procedure envisaged a “double-fixed” boundary condition system: rotation restrained at the top beam with a constant vertical load.

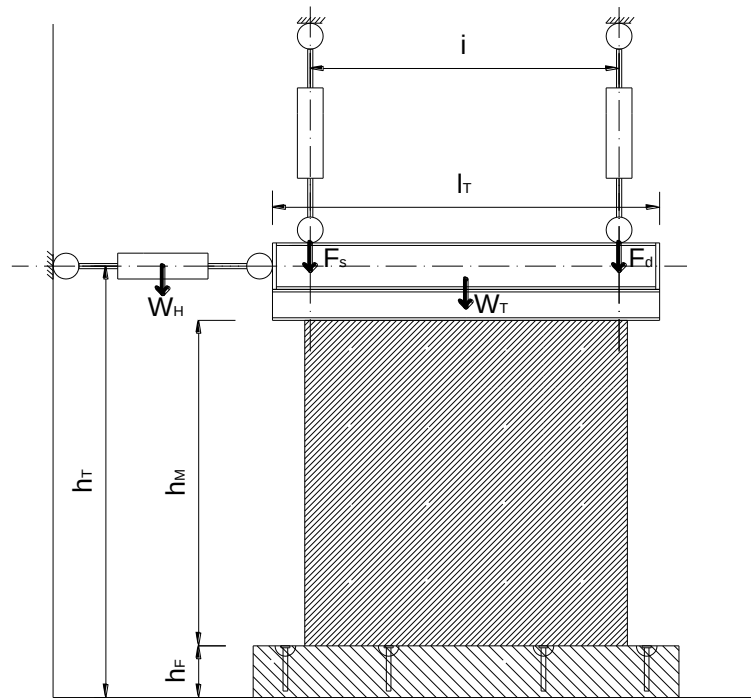


Figure 69. Sketch of the acting forces on the test set-up.

The double fixed boundary conditions can be obtained by two alternative settings of the actuator control. The first one is based on a “static” criterion (force control), the second one consists of a “kinematic” criterion (mixed control). The “kinematic” criterion allows a better control of the tests and hence it is strongly recommended, provided the testing controller allows a mixed force-displacement control.

The “static criterion” consists in imposing the condition of zero bending moment at mid-height of the wall. Therefore, the following expressions are implemented in the test procedure:

$$\begin{cases} F_D + F_S = P - \frac{W_H}{2} - W_T \\ F_S - F_D = F_H \frac{2}{i} \left(h_H - \frac{h_M}{2} - h_F \right) - \frac{l_T W_H}{2i} \end{cases}$$

Where P is the desired total load, F_D and F_S are the applied vertical forces of the right and left actuator respectively; F_H is the applied horizontal force of the horizontal actuator, W_H is the weight of the horizontal actuator, W_T is the weight of top beam, i is the horizontal distance between the 2 vertical actuators, l_T is the length of the top beam, $h_H = h_T$ is the height of the axis of the horizontal actuator, h_M is the clear height of the wall, h_F is the height of the foundation.

The “kinematic” criterion involves a mixed force-displacement control, imposing both a constant vertical load and a condition of free translation with no rotation of the top beam.

Therefore, the following expressions are implemented in the test procedure:

$$\begin{cases} F_D + F_S + \frac{W_H}{2} + W_T = P \\ u_D = u_S \end{cases}$$

where u_D and u_S are the vertical displacements of the right and left actuator respectively, F_D and F_S are the applied forces of the right and on the left actuator respectively, W_H is the weight of the horizontal actuator, W_T is the weight of top beam, i is the horizontal distance between the 2 vertical actuators, l_T is the length of the top beam.

4.4 Execution of the Test and Horizontal Loading History

Initially, the horizontal actuator was kept unloaded and the vertical load was applied by the vertical actuators with constant velocity. The horizontal loading history is then applied in a displacement-controlled procedure. The duration of each cycle is kept almost constant, incrementing the actuator displacement rate proportionally to the cycle target displacement. The tests were stopped when the first of the following conditions was met: inability of the wall to sustain the vertical load (sudden increase of vertical velocity of the top beam), sudden drop of maximum lateral force, 4% drift.

The cycles of horizontal displacement imposed to the top of the walls are summarized in the tables and figures below (from Figure 70 to Figure 79, and Table 20 to Table 24).

Table 20. Loading protocol for component EC_COMP2_1: horizontal displacement imposed on the top of the pier.

<i>N Cycles</i>	<i>Drift [%]</i>	<i>Amplitude [mm]</i>	<i>Velocity [mm/s]</i>
1_1 *	0.016	0.42	-
1_3 *	0.027	0.74	-
1_4 *	0.051	1.38	-
1_D	0.055	1.50	0.025
2_D	0.077	2.09	0.0375
3_D	0.108	2.93	0.05
4_D	0.155	4.19	0.0625
5_D	0.209	5.66	0.08
6_D	0.260	7.03	0.1
7_D	0.310	8.41	0.12
8_D	0.414	11.21	0.16
9_D	0.515	13.96	0.2
10_D	0.619	16.77	0.24
12_D	0.819	22.20	0.32
13_D	1.022	27.70	0.4
14_D	1.276	34.59	0.5
15_D	1.530	41.46	0.5
17_D	2.036	55.18	0.6
18_D	2.545	68.97	0.65
19_D	3.050	82.64	0.8
21_D	4.068	110.24	1
* The cycle is conducted in force control			

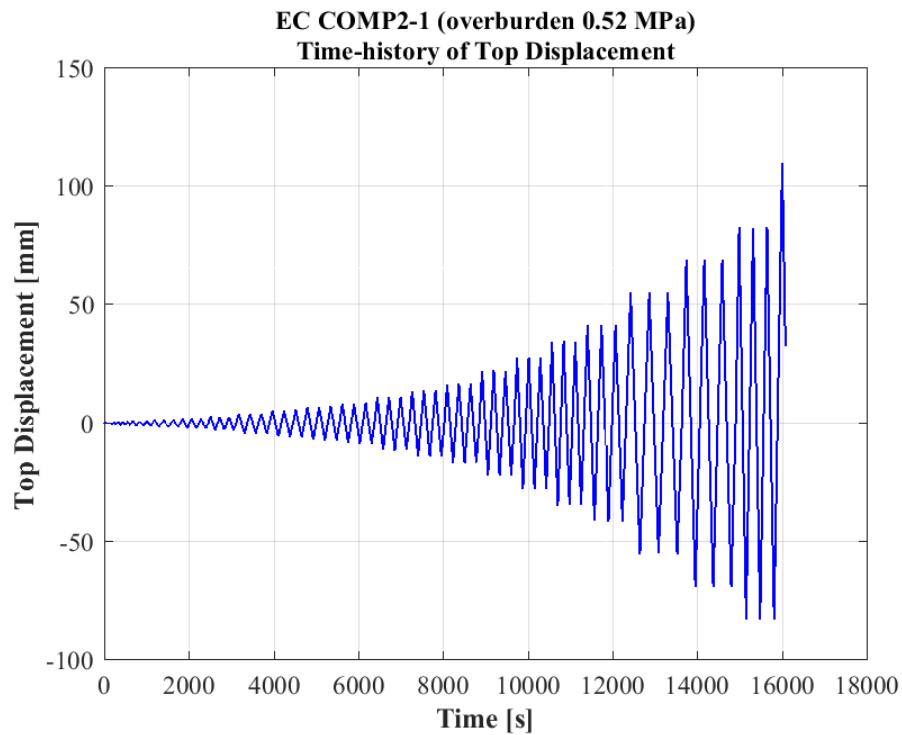


Figure 70. EC_COMP2_1: Time history of the horizontal displacement of the top of the pier.

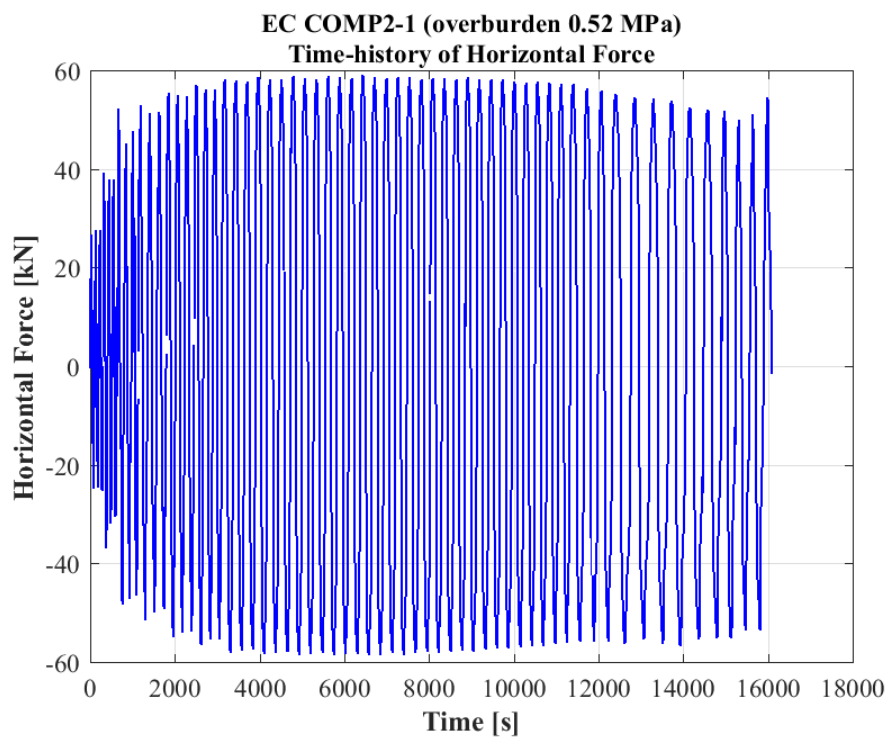


Figure 71. EC_COMP2_1: Time history of the applied horizontal force.

Table 21. Loading protocol for component EC_COMP2_2: horizontal displacement imposed on the top of the pier.

<i>N Cycles</i>	<i>Drift [%]</i>	<i>Amplitude [mm]</i>	<i>Velocity [mm/s]</i>
1_1 *	0.013	0.36	-
1_3 *	0.031	0.83	-
1_4	0.039	1.07	0.025
1_D	0.053	1.42	0.025
2_D	0.078	2.10	0.0375
3_D	0.102	2.77	0.05
4_D	0.153	4.14	0.0625
5_D	0.204	5.53	0.08
6_D	0.255	6.91	0.1
7_D	0.306	8.29	0.12
8_D	0.410	11.1	0.16
9_D	0.511	13.84	0.2
10_D	0.612	16.57	0.24
12_D	0.814	22.05	0.32
13_D	1.018	27.59	0.4
14_D	1.273	34.38	0.5
* The cycle is conducted in force control			

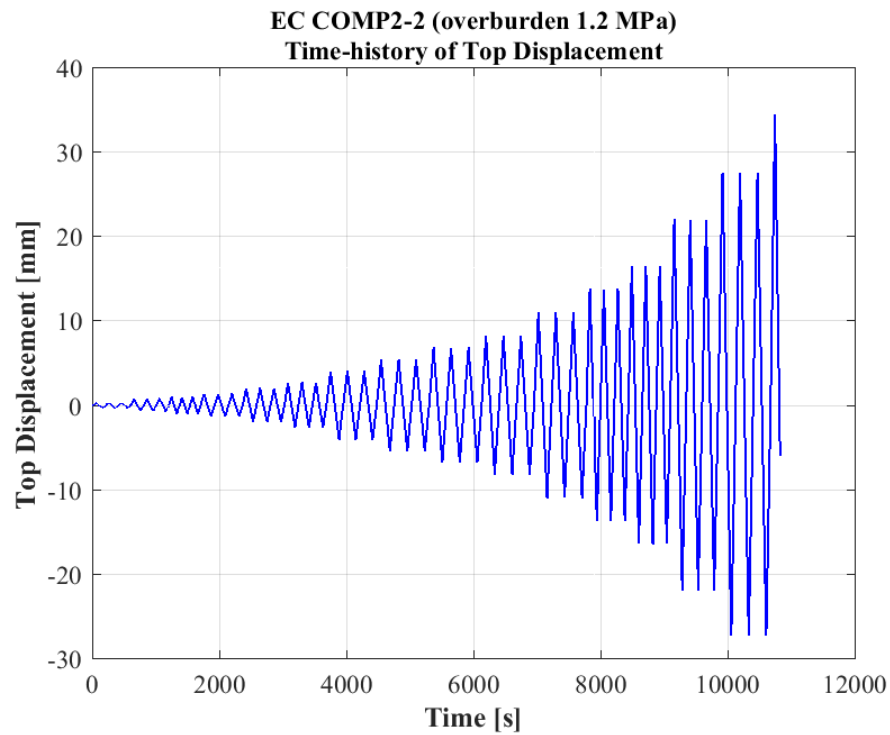


Figure 72. EC_COMP2_2: Time history of the horizontal displacement of the top of the pier.

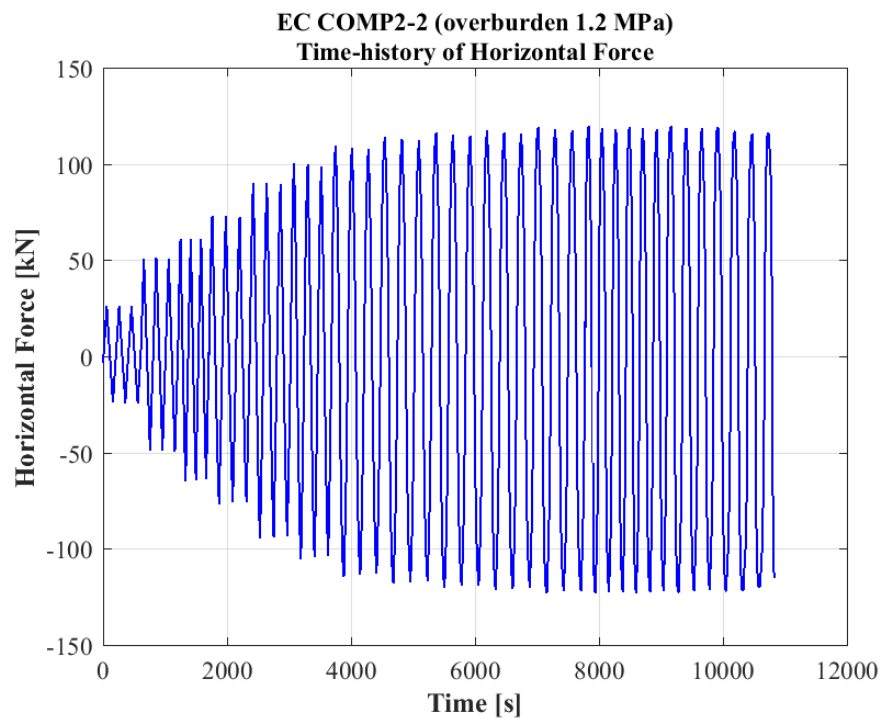


Figure 73. EC_COMP2_2: Time history of the applied horizontal force.

Table 22. Loading protocol for component EC_COMP2_3: horizontal displacement imposed on the top of the pier.

<i>N Cycles</i>	<i>Drift [%]</i>	<i>Amplitude [mm]</i>	<i>Velocity [mm/s]</i>
1_1 *	0.015	0.41	-
1_3 *	0.031	0.85	-
1_4	0.037	0.99	0.025
1_D	0.049	1.32	0.025
2_D	0.074	2.10	0.0375
3_D	0.099	2.68	0.05
4_D	0.151	4.10	0.0625
5_D	0.201	5.44	0.08
6_D	0.252	6.82	0.1
7_D	0.303	8.20	0.12
8_D	0.402	10.89	0.16
9_D	0.502	13.59	0.2
10_D	0.604	16.35	0.24
12_D	0.808	21.89	0.32
13_D	1.011	27.40	0.4
14_D	1.265	34.28	0.5
* The cycle is conducted in force control			

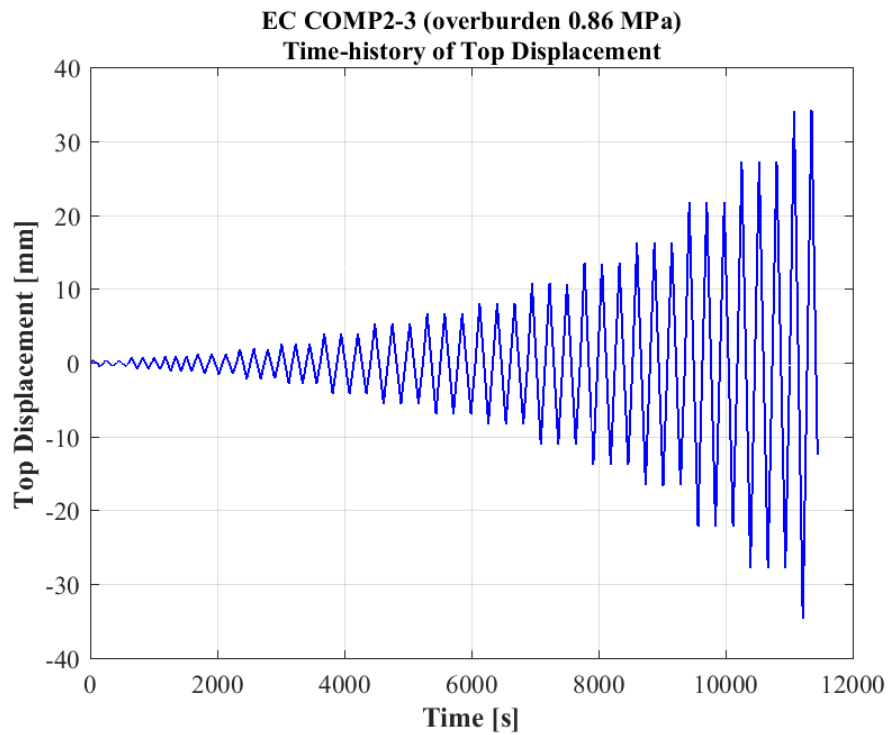


Figure 74. EC_COMP2_3: Time history of the horizontal displacement of the top of the pier.

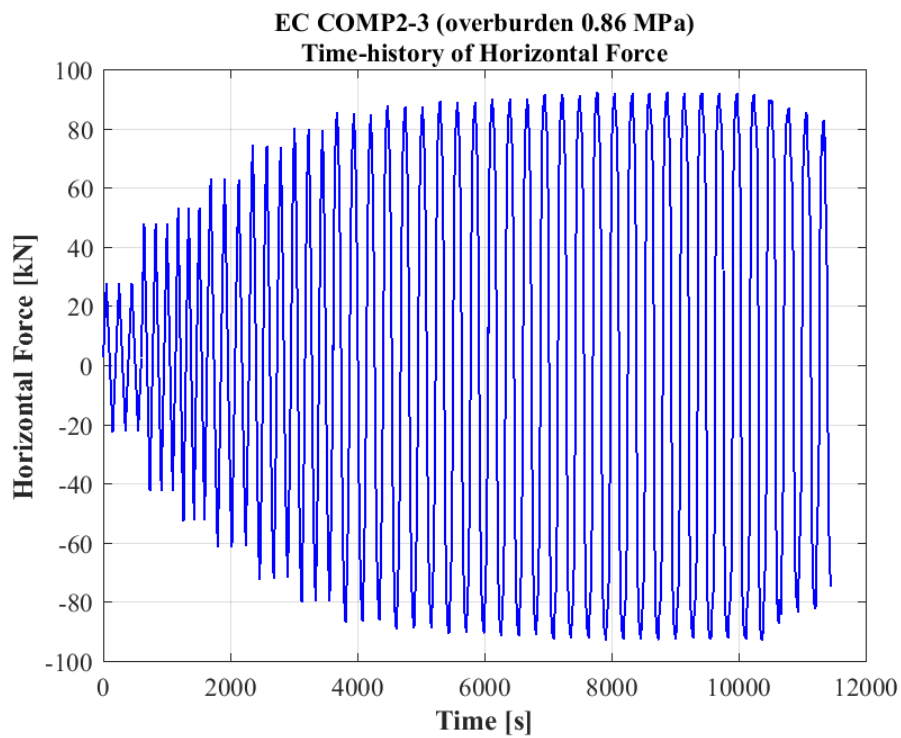


Figure 75. EC_COMP2_3: Time history of the applied horizontal force.

Table 23. Loading protocol for component EC_COMP2_4: horizontal displacement imposed on the top of the pier.

<i>N Cycles</i>	<i>Drift [%]</i>	<i>Amplitude [mm]</i>	<i>Velocity [mm/s]</i>
1_1*	0.003	-0.09	-
1_2*	0.004	-0.11	-
1_3*	0.008	-0.25	-
1_4	0.016	-0.45	0.02
1_5	0.020	-0.56	0.02
1_6	0.029	-0.79	0.02
1_7	0.038	-1.04	0.02
1_D	0.051	-1.40	0.025
2_D	0.077	-2.08	0.0375
3_D	0.103	-2.79	0.05
4_D	0.153	-4.16	0.0625
5_D	0.204	-5.53	0.08
6_D	0.255	-6.92	0.1
7_D	0.319	-8.64	0.12
* The cycle is conducted in force control			

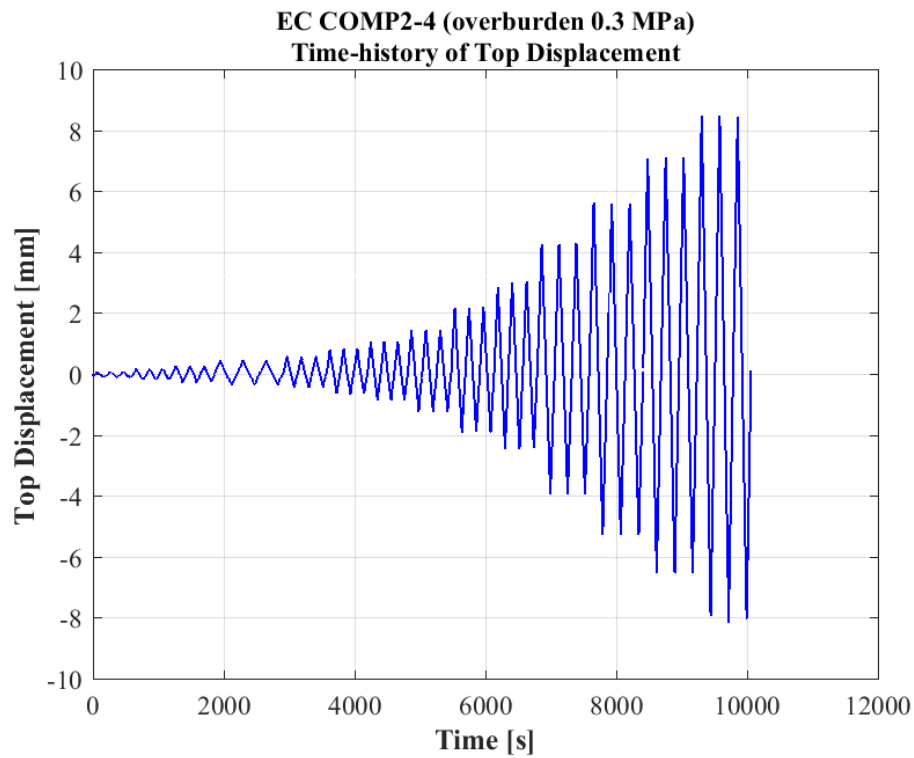


Figure 76. EC_COMP2_4: Time history of the horizontal displacement of the top of the pier.

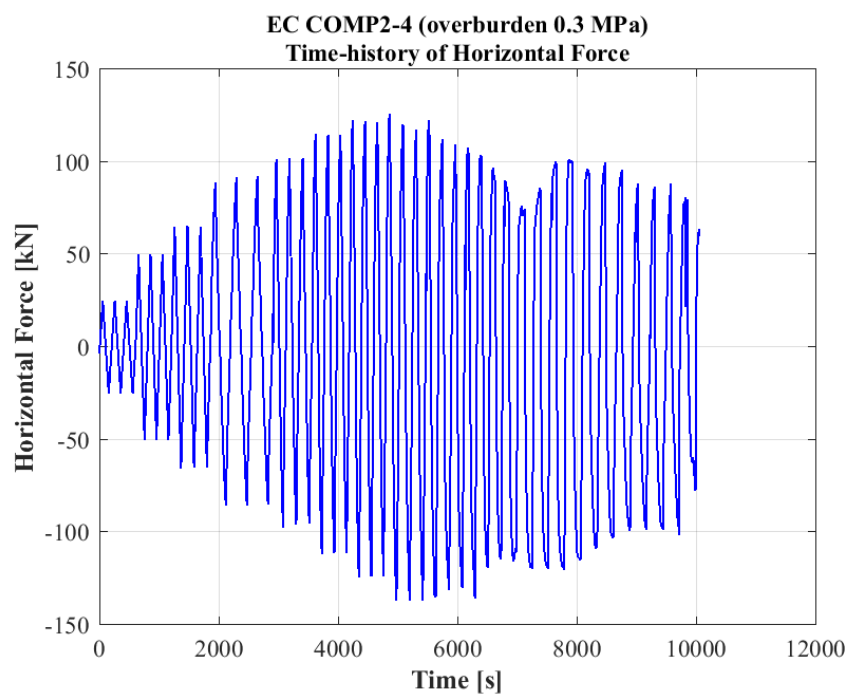


Figure 77. EC_COMP2_4: Time history of the applied horizontal force.

Table 24. Loading protocol for component EC_COMP2_5: horizontal displacement imposed on the top of the pier.

<i>N Cycles</i>	<i>Drift [%]</i>	<i>Amplitude [mm]</i>	<i>Velocity [mm/s]</i>
1_1*	0.003	0.07	-
1_2*	0.006	0.17	-
1_3*	0.013	0.36	-
1_5	0.020	0.55	0.02
1_6	0.030	0.83	0.02
1_D	0.051	1.38	0.025
3_D	0.101	2.74	0.05
5_D	0.201	5.45	0.08
7_D	0.301	8.17	0.12
8_D	0.402	10.90	0.16
10_D	0.603	16.35	0.24
12_D	0.804	21.79	0.32
13_D	1.005	27.23	0.4
15_D	1.528	41.40	0.5
17_D	1.434	38.85	0.6
* The cycle is conducted in force control			

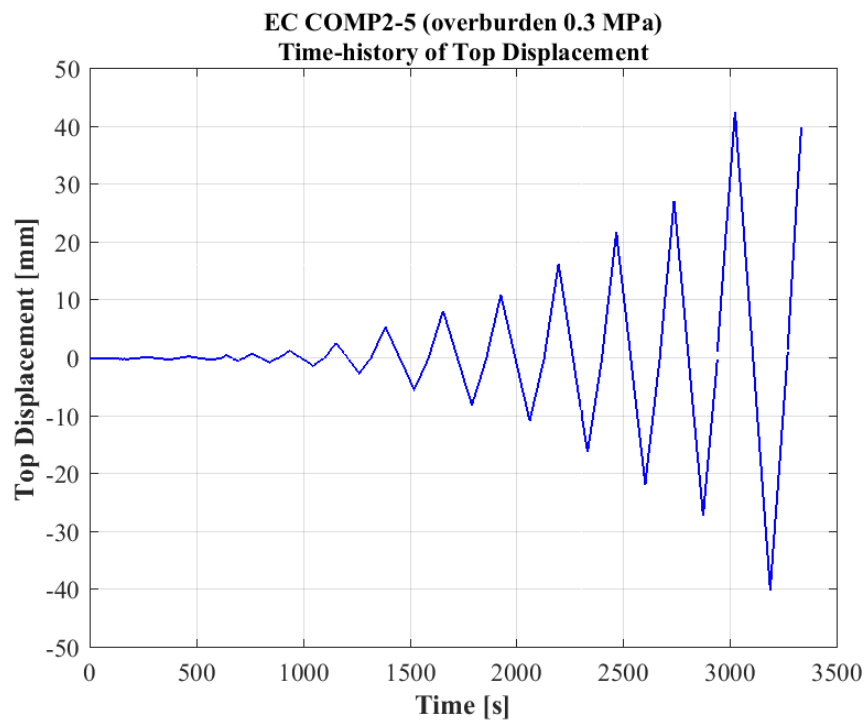


Figure 78. EC_COMP2_5: Time history of the horizontal displacement of the top of the pier.

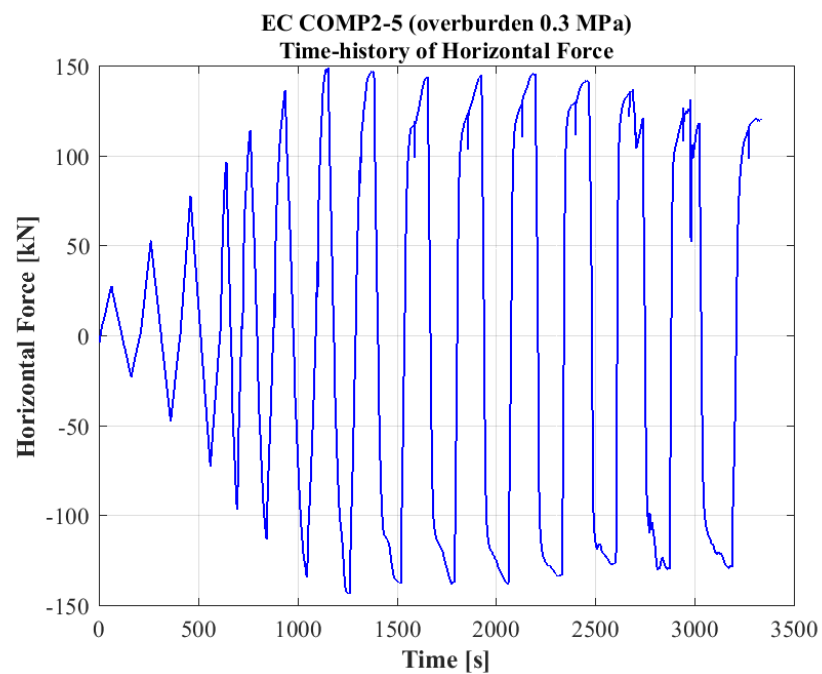


Figure 79. EC_COMP2_5: Time history of the applied horizontal force.

4.5 Experimental Results

4.5.1 EC_COMP2-1

EC_COMP2-1 is the first of the five full-scale components tested at the EUCENTRE Lab facilities. The specimen, 1.2 m long and 2.71 m high, was subjected to in-plane cyclic quasi-static tests performed under an overburden of 0.52 MPa and double-fixed boundary conditions. The experimental set-up of the component EC_COMP2-1 is illustrated in Figure 80.

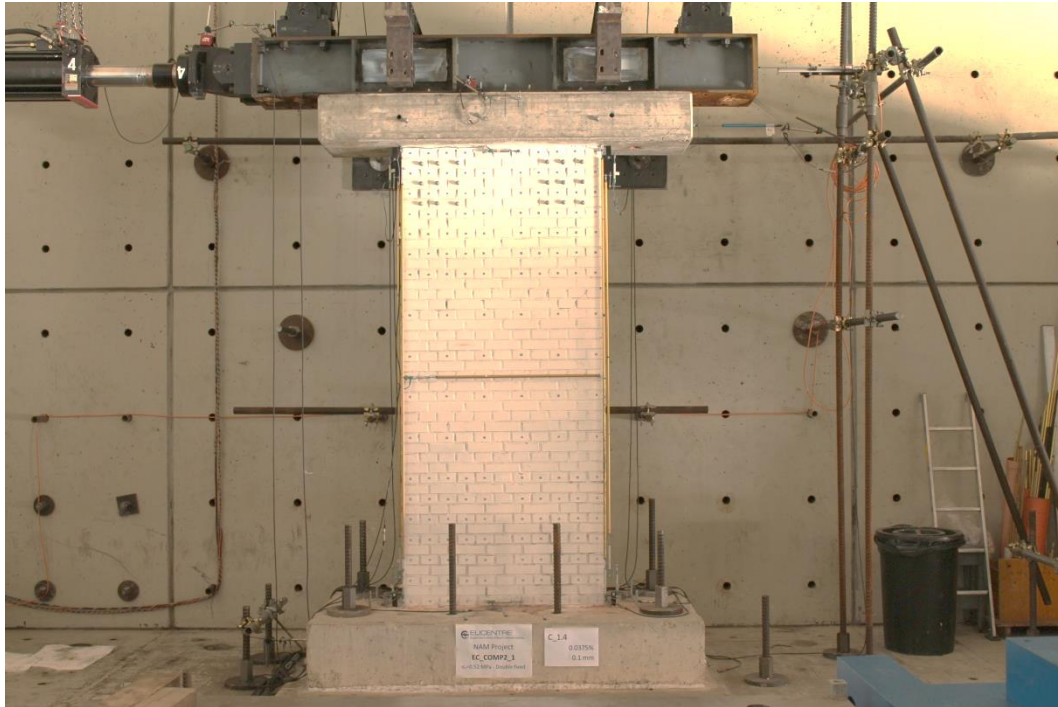


Figure 80. Test set-up of EC_COMP2-1.

As expected, the specimen exhibited a pure rocking behavior during the whole test. The first horizontal cracks appeared one and two layers of bricks above the base and below the top of the wall (Figure 82), during loading cycle 5_D (0.2% drift - 5.5 mm top displacement). During the subsequent loading cycles no new cracks were observed, while the deformations were accommodated by opening of the pre-existing cracks. The masonry panel did not suffer any shear damage and the test was stopped during loading cycle 21_D, as soon as a maximum drift of 4% was attained (top displacement equal to 110 mm), with the development of a severe toe crushing mechanism (Figure 83). At the end of the test, the wall was still able to sustain the applied vertical load.

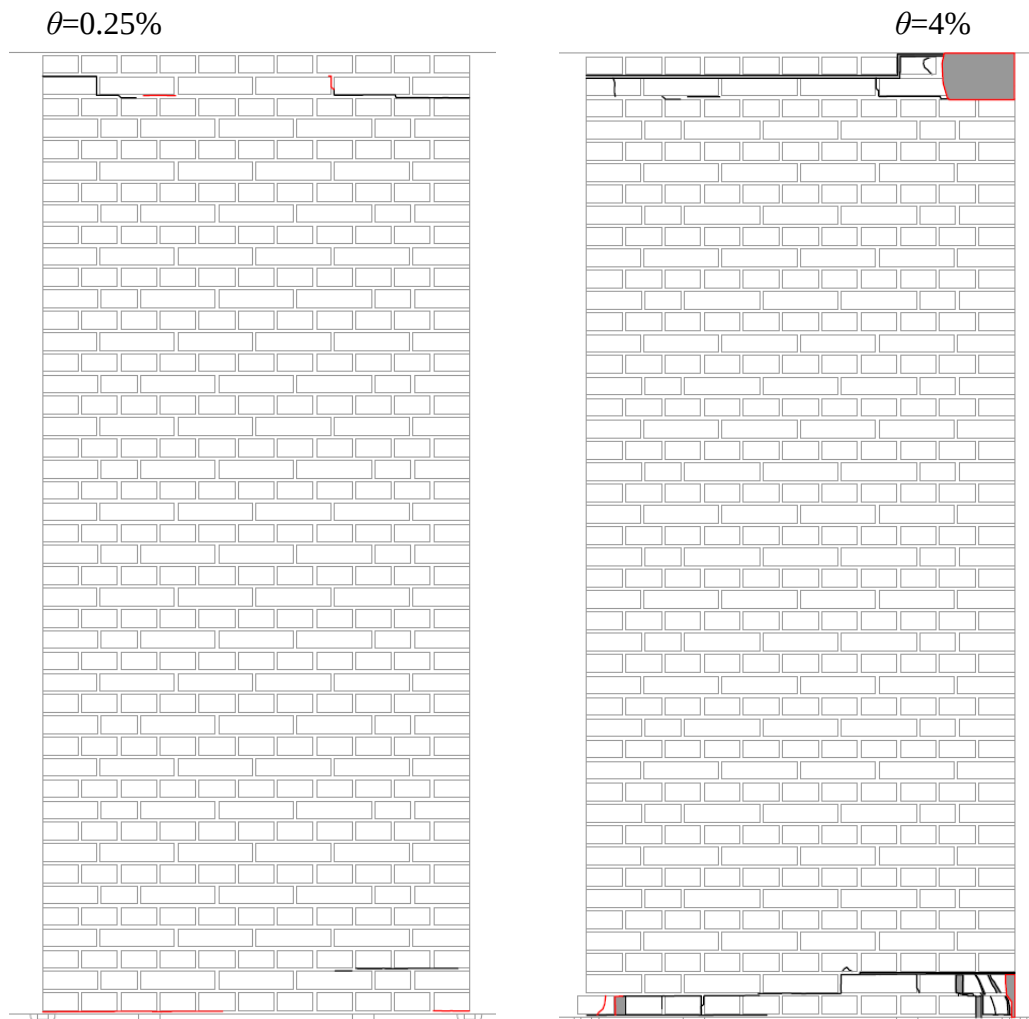


Figure 81. Illustration of the observed crack pattern for specimen EC_COMP2-1 after loading cycles 6_D and 21_D.



Figure 82 EC_COMP2-1: Horizontal crack at the top of the pier, associated with its flexural behaviour.

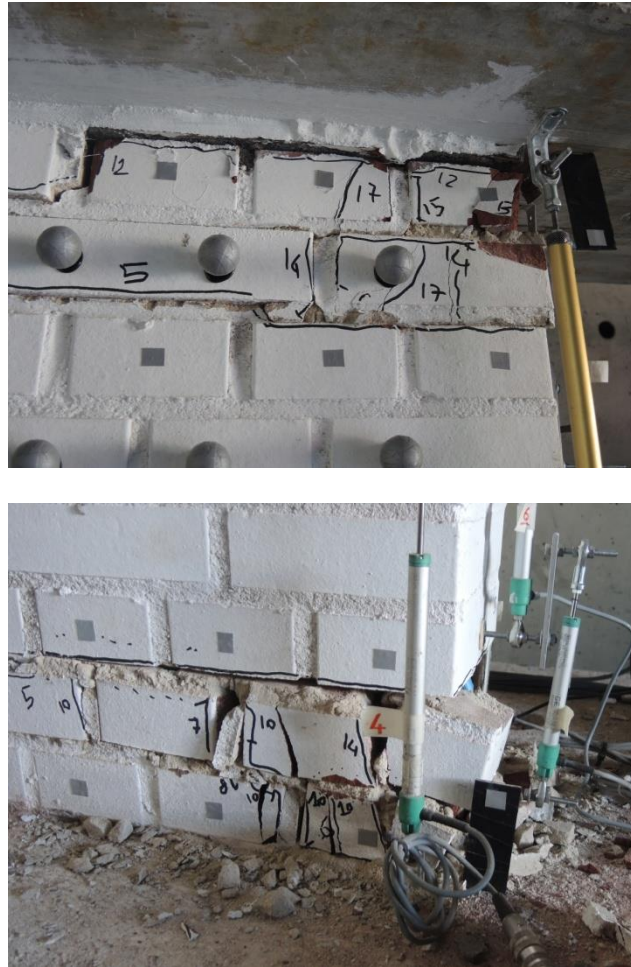


Figure 83. EC_COMP2-1: Development of toe crushing mechanism at both top and bottom sections of the pier.

The hysteretic response of the specimen EC_COMP2-1 is reported in Figure 84. The initial lateral stiffness of the wall was 65.5 kN/mm and the shear capacity 59.15 kN in the positive direction (e.g. actuator pulling) and 58.45 in negative direction.

According to the results obtained during the vertical load application phase, an evaluation of the uncracked Young Moduli has been carried out (considering the shortening of the LVDT #6 and #7 under the imposed increasing load on top). The computation leads to a value of 9667 MPa.

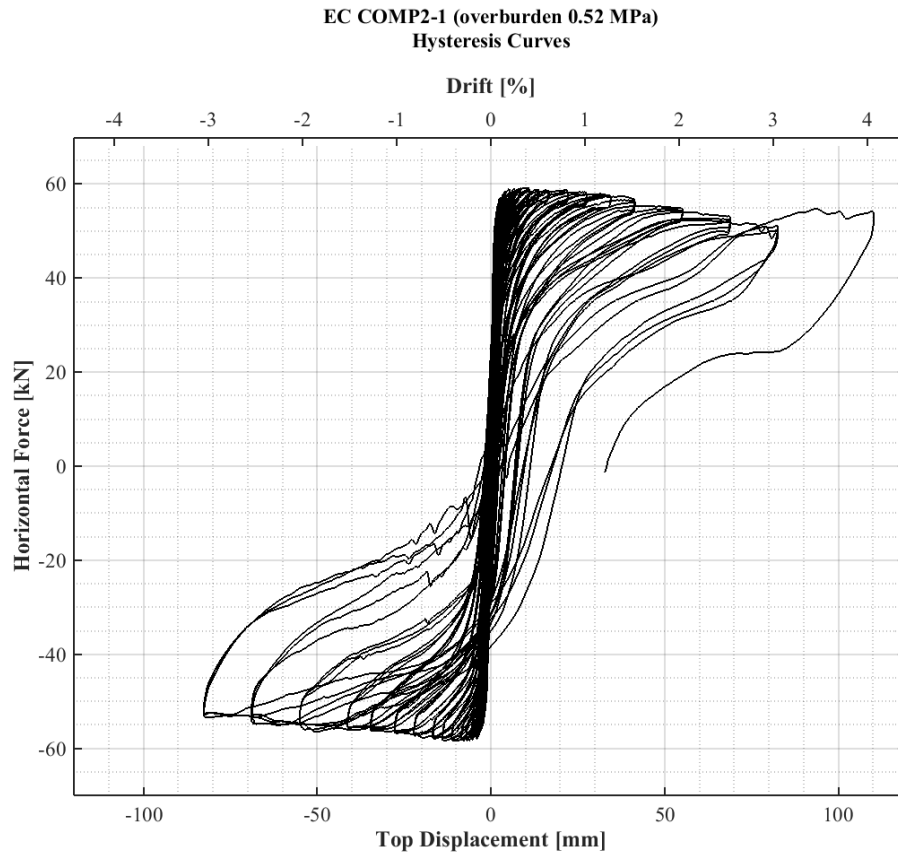


Figure 84. Hysteretic behaviour of component EC_COMP2-1.

Figure 85 reports the total work computed during each cycles (the red curve represents the dissipated energy).

The dissipated hysteretic energy was evaluated also in terms of equivalent viscous damping coefficient:

$$\xi_{hyst} = A_h / (2\pi F_m \Delta_m)$$

where A_h is the area within one complete cycle of stabilized force-displacement response, F_m and Δ_m are the maximum force and displacement achieved in the stabilized loops. The results of the calculated equivalent viscous damping are plotted in Figure 86 as a function of displacement ductility of each cycle (calculated considering the bilinear approximation).

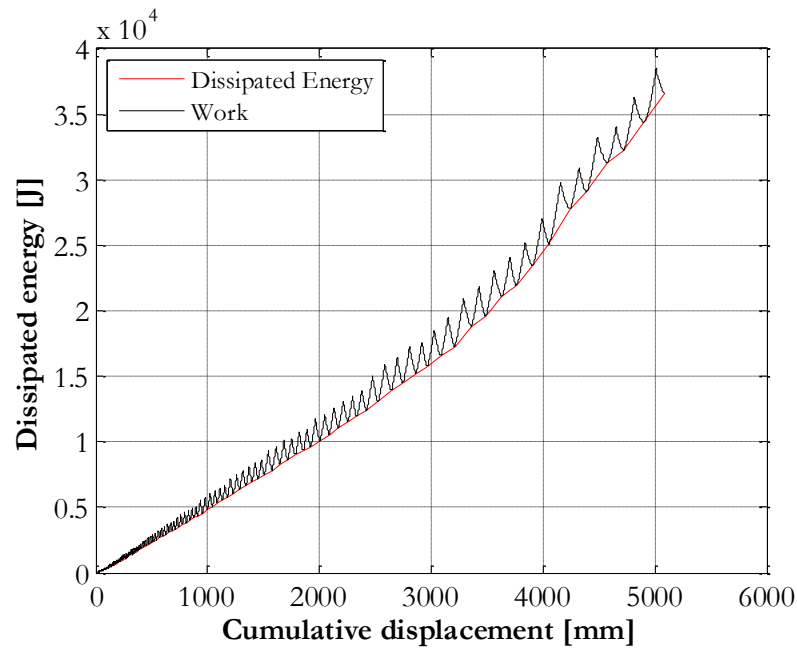


Figure 85. EC_COMP2-1: Work of horizontal force and dissipated energy plot.

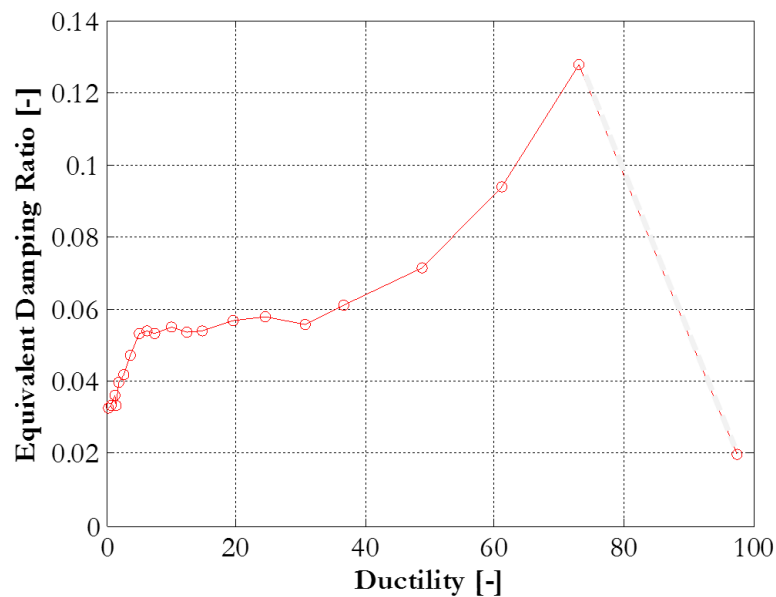


Figure 86. EC_COMP2-1: Equivalent viscous damping ratio vs. Displacement ductility.

According to the results obtained from the optical acquisition system (in particular using the coordinate time histories of the marker located at the edges of the panel), an investigation about the deformation modes of the masonry panel have been carried out. Figure 87 to Figure 89 report the results in terms of shear angular deformation (γ), flexural angular deformation ($\Delta\theta_y$) and vertical elongation (Δy), respectively.

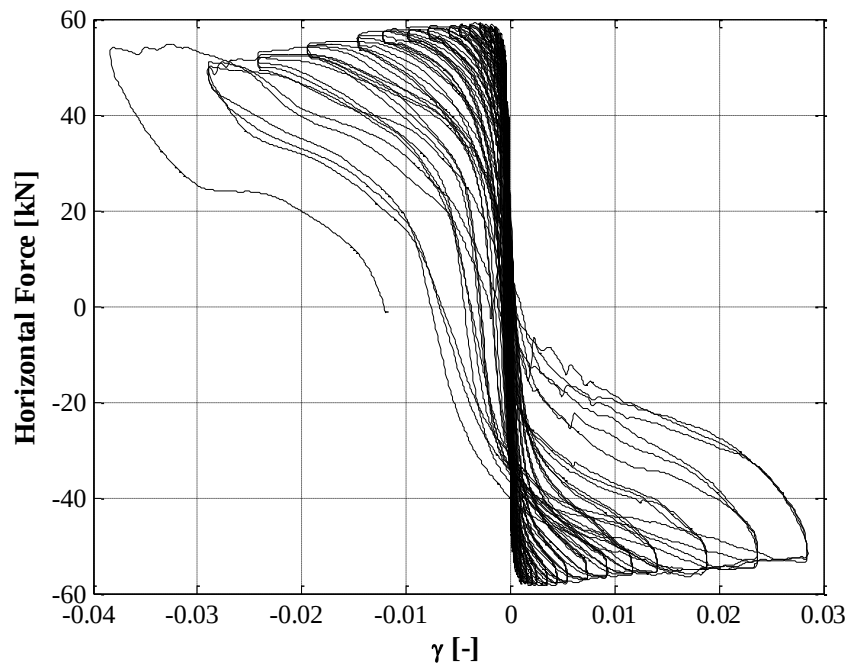


Figure 87. EC_COMP2-1: Shear deformation plot.

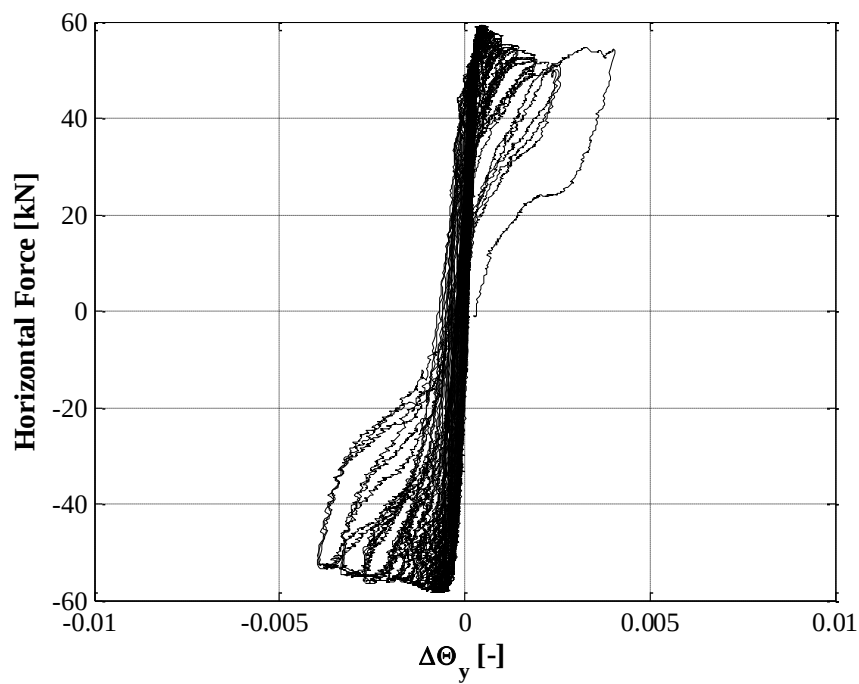


Figure 88. EC_COMP2-1: Flexural deformation plot.

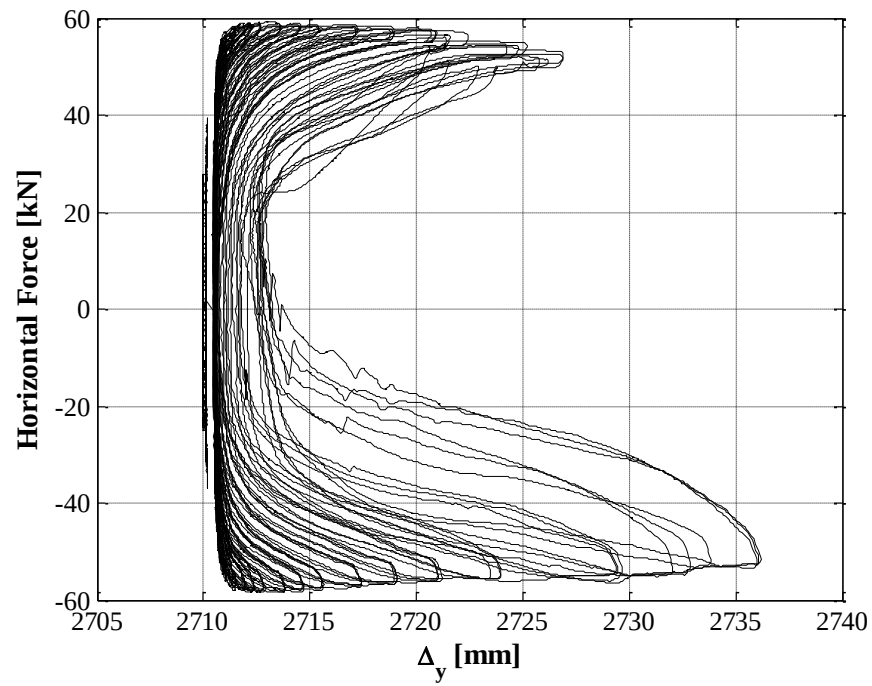


Figure 89. EC_COMP2-1: Vertical elongation plot.

4.5.2 EC_COMP2-2

EC_COMP2-2 is the second of the three slender piers to be tested. It was subjected to in-plane cyclic quasi-static tests performed under overburden of 1.2 MPa. The specimen was 1.2 m long and 2.71 m high, with fixed boundary conditions at both top and base. Figure 90 reports the test set-up of EC_COMP2-2.



Figure 90. Test set-up of EC_COMP2-2.

Initially, this pier exhibited rocking behaviour, since horizontal cracks formed during loading cycle 5_D (0.2% drift - 5.5 mm top displacement), one at the base, along the wall-foundation interface, and another one at the top, one brick layer below the top beam (Figure 91). During loading cycle 9_D (0.5% drift - 13.75 mm top displacement) the first signs of toe crushing became visible, identified mainly at the edges of the base (Figure 93). This is due to the quite high compressive load under which the specimen was tested.

While testing at target drift 1% (corresponding to top displacement 27.5 mm – loading cycle 13_D), a diagonal crack at an angle of 45° formed in the bottom half of the pier, for pushing toward left. Within the same loading cycle, but for pushing towards the opposite direction (right) a similar stair-stepped crack appeared, leading the specimen to collapse, since it was unable to sustain the high vertical load applied at the top (Figure 94). The finally attained top displacement was 34.38 mm, corresponding to a drift value of 1.25%. The failure of the specimen was a typical example of a hybrid failure (shear failure occurred after the attainment of the maximum flexural capacity of the pier).

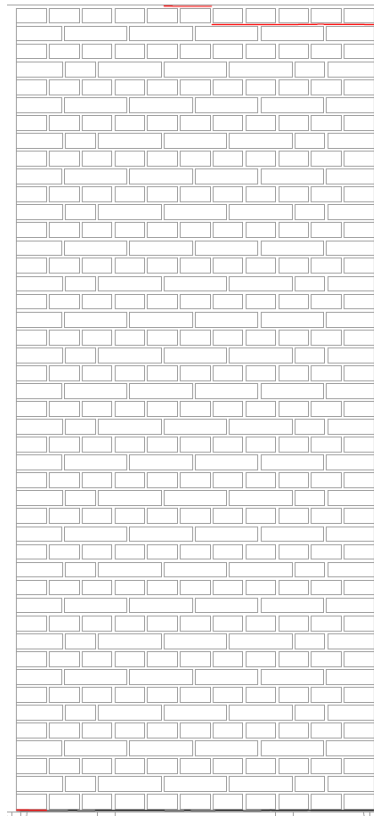
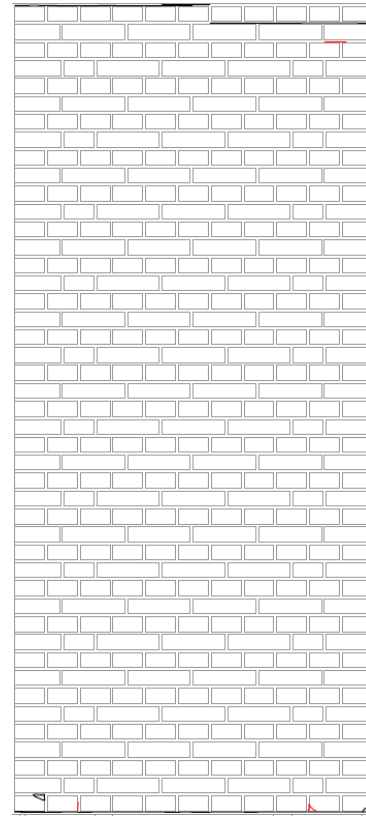
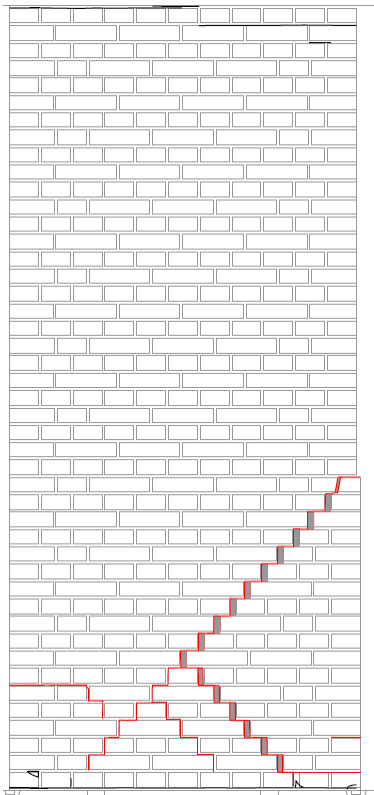
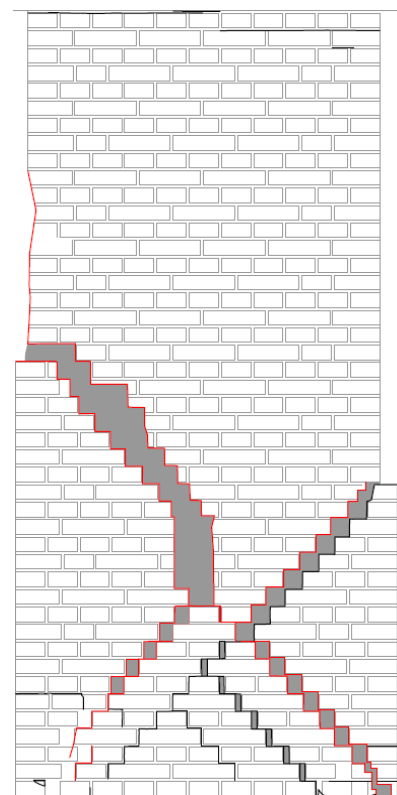
$\theta=0.25\%$  $\theta=0.4\%$  $\theta=1\%$  $\theta=1.25\%$ 

Figure 91. Illustration of the observed crack pattern for specimen EC_COMP2-2 after loading cycles 6_D, 12_D, 13_D and 14_D



Figure 92. EC_COMP2-2: Horizontal crack along the base of the specimen.



Figure 93. EC_COMP2-2: Damage associated with toe crushing at the base of the pier.



Figure 94. EC_COMP2-2: Diagonal cracks in the masonry panel and collapse of the specimen.

The hysteretic response of the pier EC_COMP2-2 is reported in Figure 95. The initial lateral stiffness of the wall was 74.4 kN/mm and the shear capacity 119.93 kN in the positive direction (e.g. actuator pulling) and 122.67 kN in negative direction. According to the results obtained during the vertical load application phase, an evaluation of the uncracked Young Moduli has been carried out (considering the shortening of the LVDT #6 and #7 under the imposed increasing load on top). The computation leads to a value of 8586 MPa.

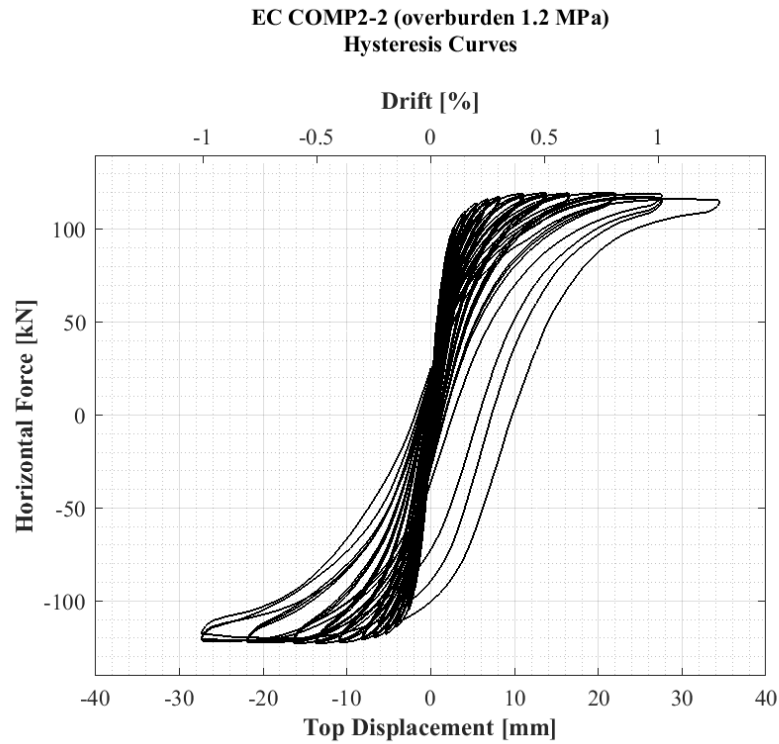


Figure 95. Hysteretic behaviour of component EC_COMP2-2.

As for the previous test, Figure 96 reports the total work computed during each cycles (the red curve represents the dissipated energy). The results of the calculated equivalent viscous damping are plotted in Figure 97 as a function of displacement ductility of each cycle.

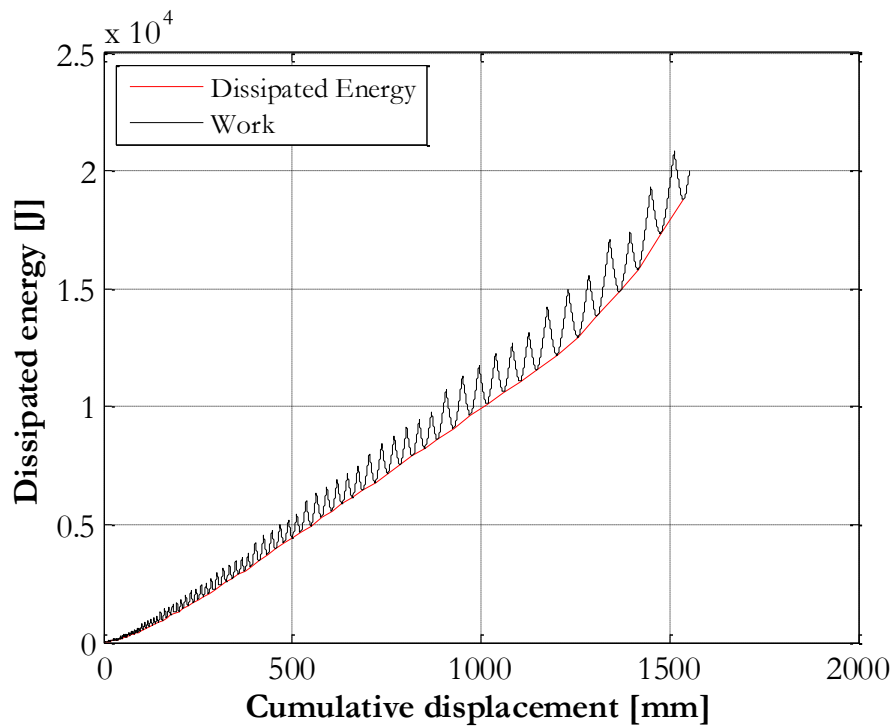


Figure 96. EC_COMP2-2: Work of horizontal force and dissipated energy plot.

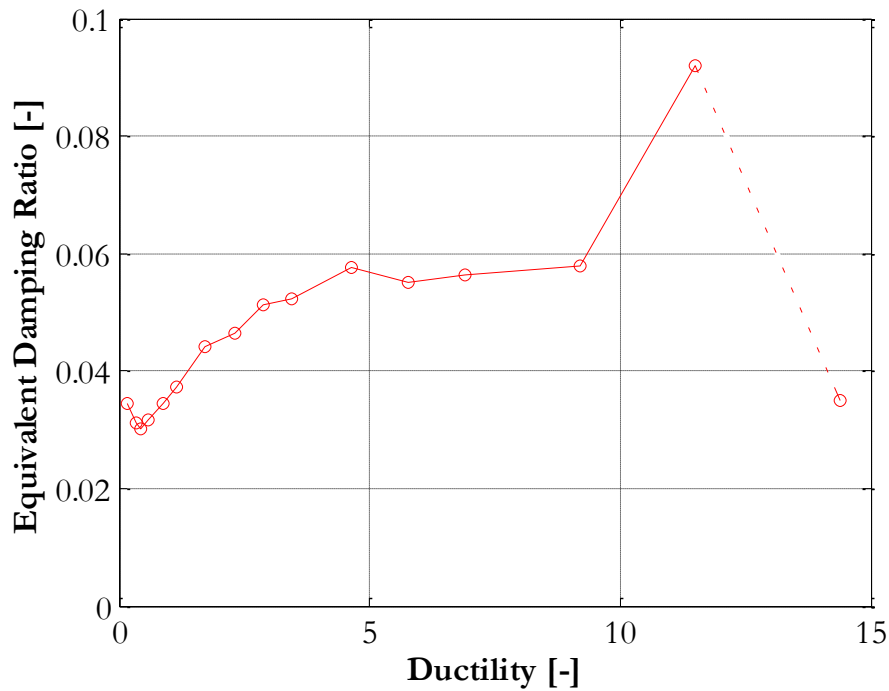


Figure 97. EC_COMP2-2: Equivalent viscous damping ratio vs. Displacement ductility.

According to the results obtained from the optical acquisition system (in particular using the coordinate time histories of the marker located at the edges of the panel), an investigation about the deformation modes of the masonry panel have been carried out. Figure 98 to Figure 100 report the results in terms of shear angular deformation (γ), flexural angular deformation ($\Delta\theta_y$) and vertical elongation (Δ_y), respectively.

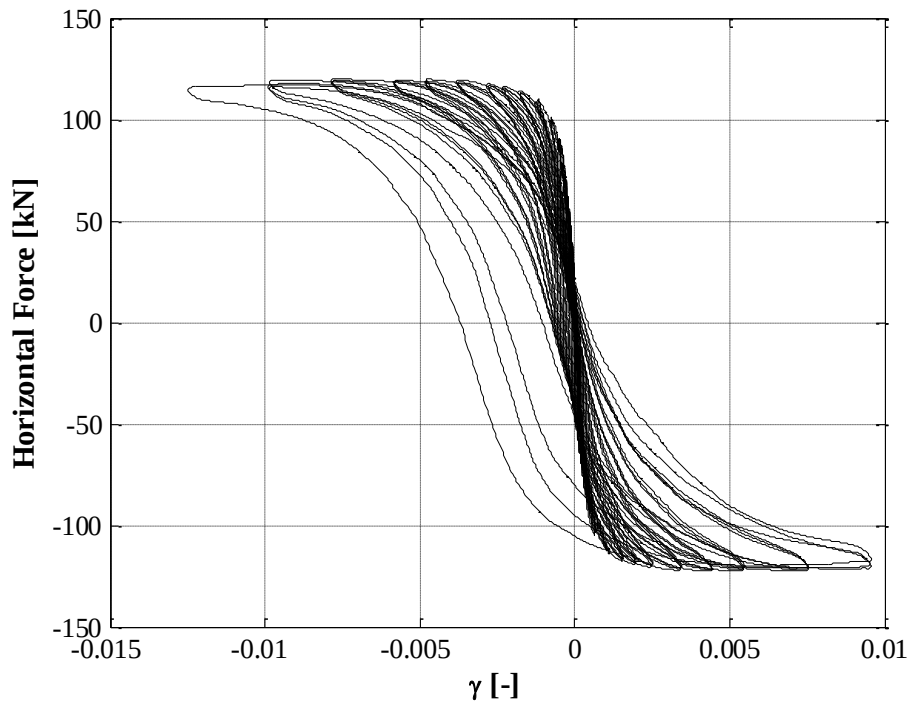


Figure 98. EC_COMP2-2: Shear deformation plot.

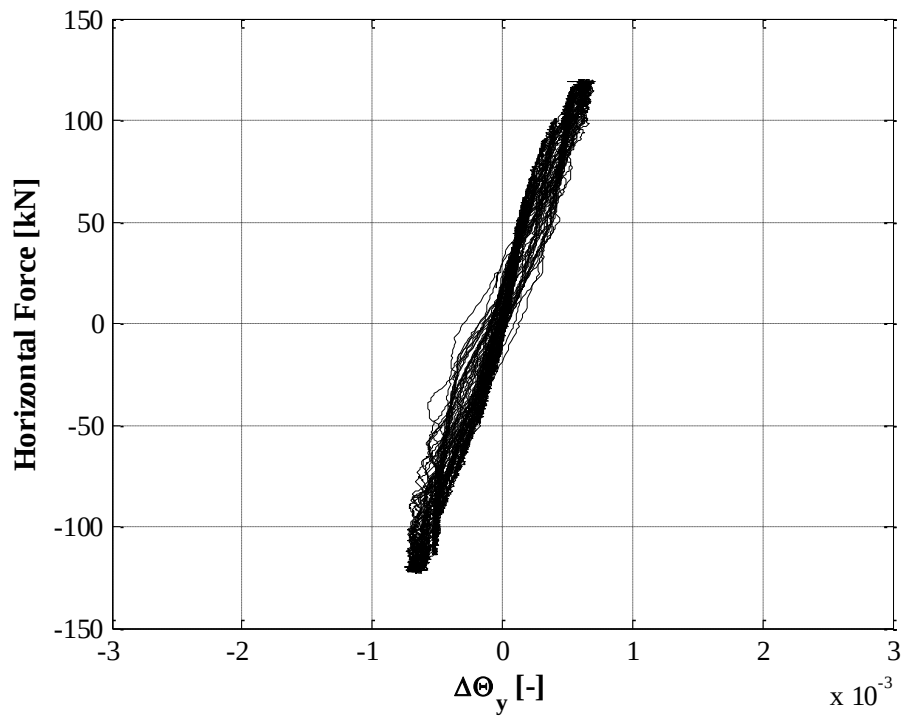


Figure 99. EC_COMP2-2: Flexural deformation plot.

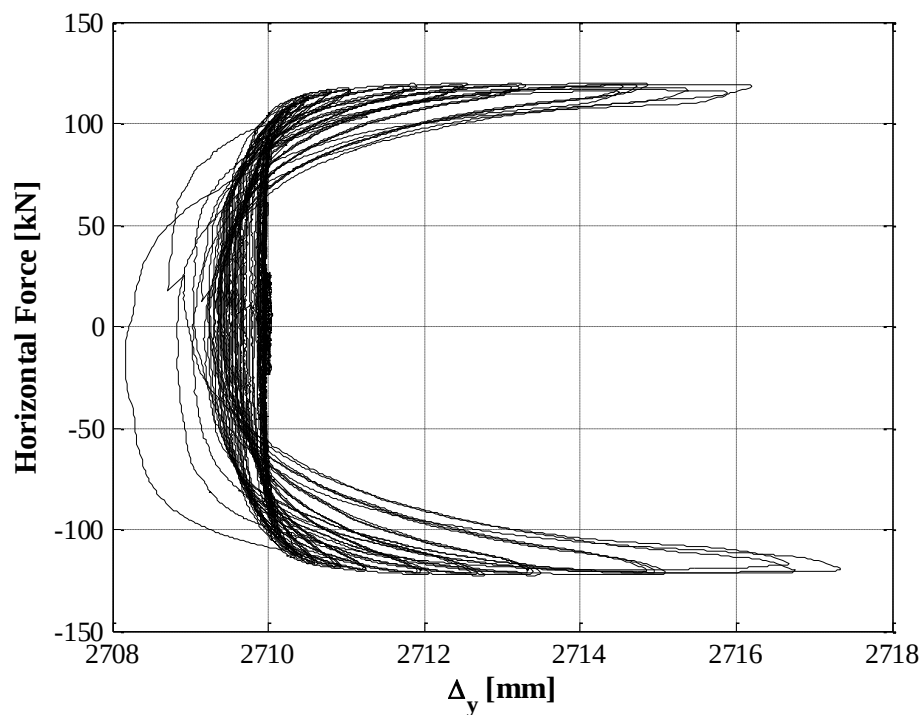


Figure 100. EC_COMP2-2: Vertical elongation plot.

Figure 101 reports the width development of the diagonal crack formed during the loading cycle 13D (target drift 1%, corresponding to top displacement 27.5 mm), in the bottom half of the pier, for pushing toward left.

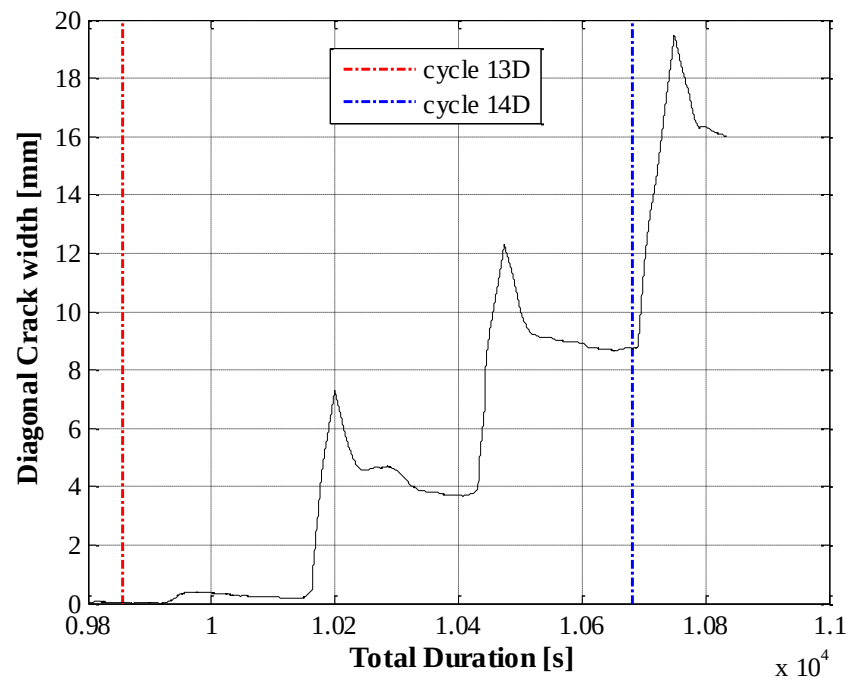


Figure 101. Width development of the diagonal crack formed in the masonry panel during cycle 13D.

4.5.3 EC_COMP2-3

EC_COMP2-3 is the last of the three slender piers to be tested. It was subjected to in-plane cyclic quasi-static tests performed under overburden of 0.86 MPa. The specimen was 1.2 m long and 2.71 m high, with double-fixed boundary conditions. Figure 102 shows the test set-up of EC_COMP2-3.

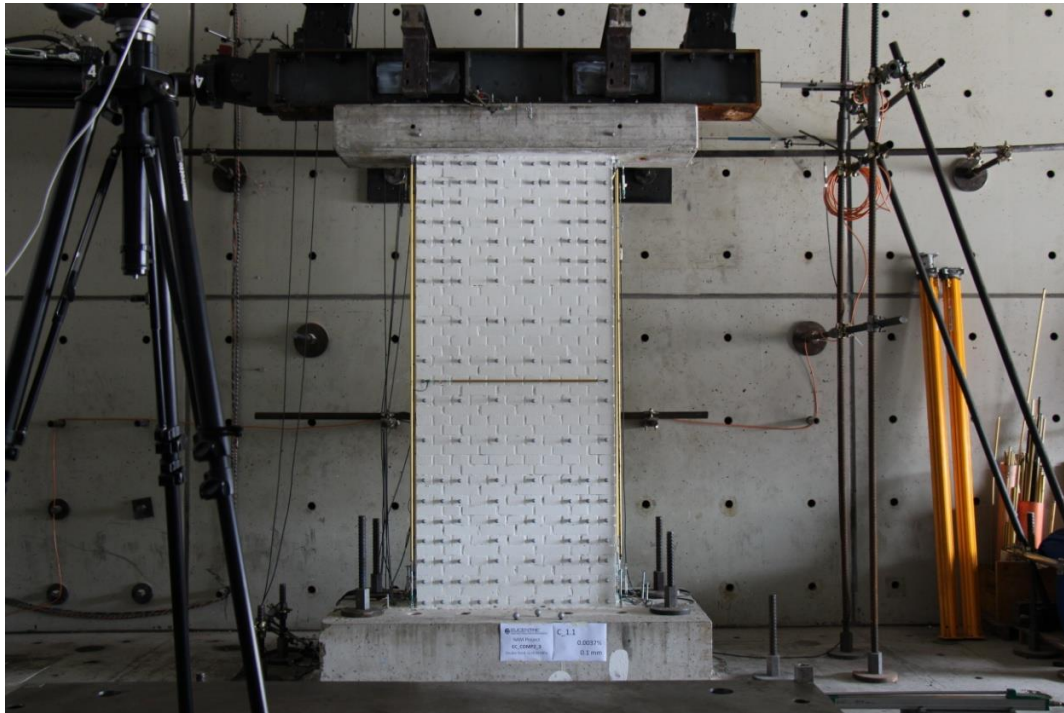


Figure 102. Test set-up of EC_COMP2-3.

The first reported cracks were identified at the wall-foundation interface, during loading cycle 4_D (0.15% drift - 4.13 mm top displacement). The same cracks were elongated all along the wall base (Figure 104) during the following cycle 5_D (0.2% drift - 5.5 mm top displacement), while horizontal cracks also appeared two layers of bricks below the wall-top beam interface (Figure 104 - residual width of 0.1 mm). After this point, the pier exhibited pure rocking behaviour until subject to loading cycle 13_D (1% drift - 27.5 mm top displacement), when the first diagonal cracks appeared a few centimetres above the base (associated with the wall's flexural behaviour), and the specimen exhibits intensive toe crushing mechanism at the base (Figure 105). During loading cycle 14_D (1.25% drift - 34.38 mm top displacement, second repetition), diagonal cracks appear in the top half of the pier, rendering the wall unable to carry the applied vertical load anymore (i.e. increase in vertical velocity of the top beam) (Figure 106). As specimen EC_COMP2-2, the failure of the specimen was a hybrid failure (shear failure occurred after the attainment of the maximum flexural capacity of the pier).

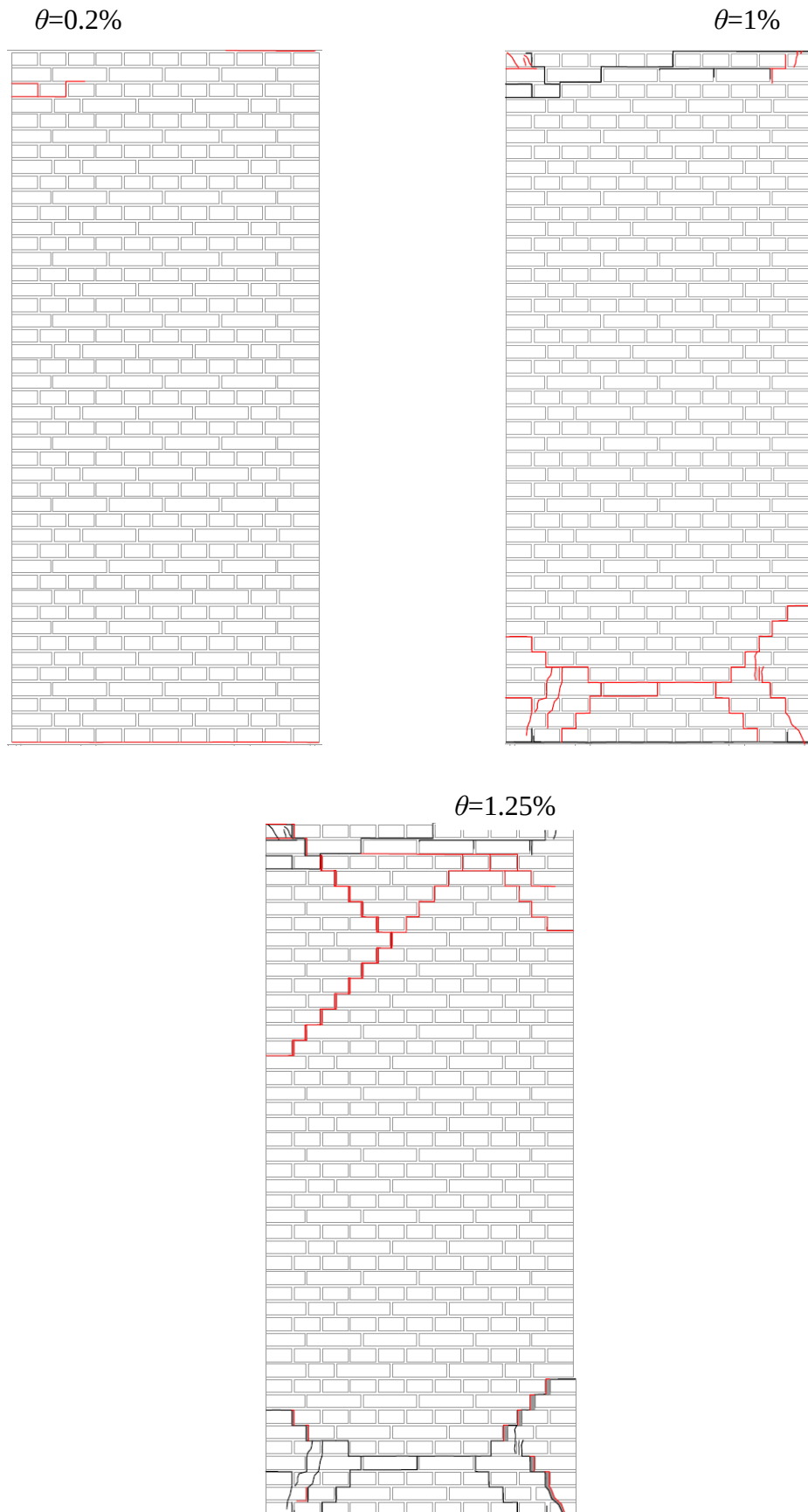


Figure 103. Illustration of the observed crack pattern for specimen EC_COMP2-3 after loading cycles 5_D, 13_D and 14_D



Figure 104. EC_COMP2-3: Horizontal cracks along the base and the top of the pier, associated with its flexural behaviour.



Figure 105. EC_COMP2-3: Development of toe crushing mechanism at the base of the pier.



Figure 106. EC_COMP2-3: Diagonal cracks appear in the top half of the pier after loading cycle 14_D.

The hysteretic response of the pier EC_COMP2-3 is reported in Figure 107. The initial lateral stiffness of the wall was 68.3 kN/mm and the shear capacity 92.52 kN in the positive direction (e.g. actuator pulling) and 92.55 kN in negative direction. According to the results obtained during the vertical load application phase, an evaluation of the uncracked Young Moduli has been carried out. The computation leads to a value of 8386 MPa.

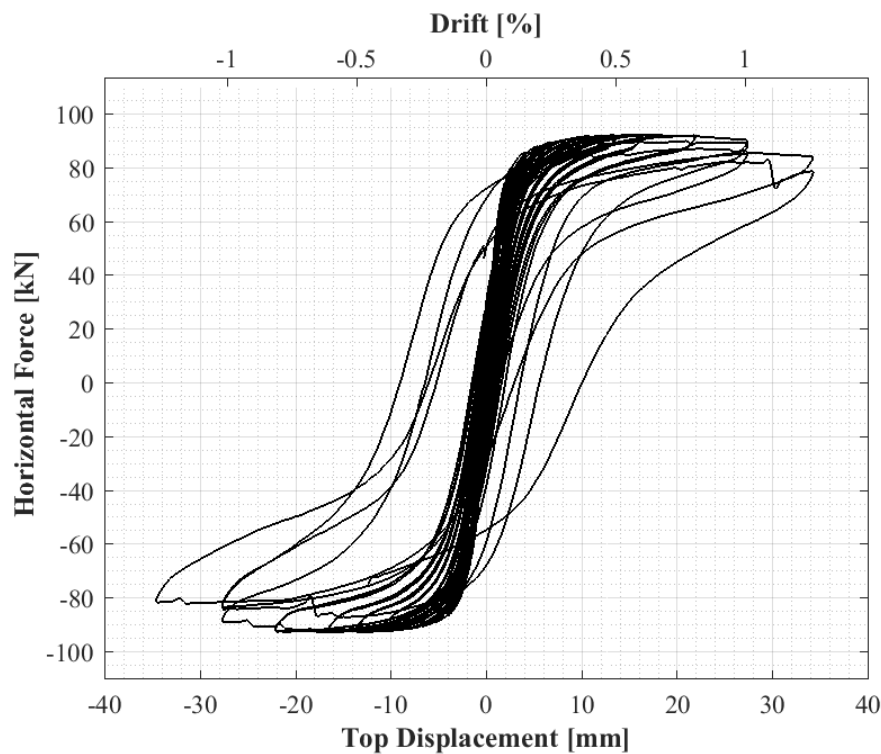


Figure 107. Hysteretic behaviour of component EC_COMP2-3.

As for the previous test, Figure 108 reports the total work computed during each cycles (the red curve represents the dissipated energy). The results of the calculated equivalent viscous damping are plotted in Figure 109 as a function of displacement ductility of each cycle.

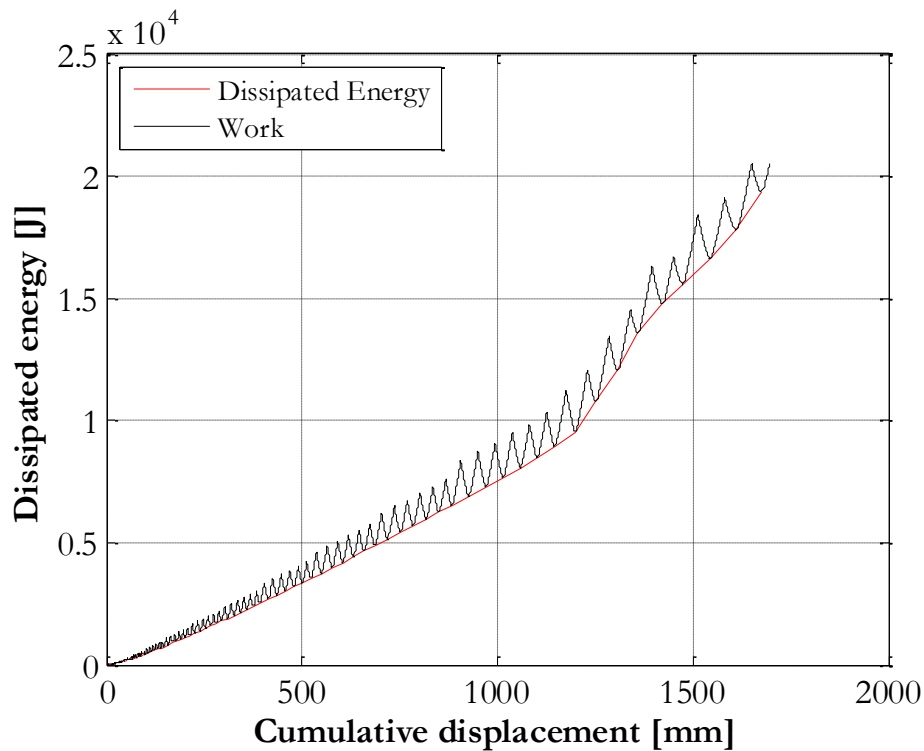


Figure 108. EC_COMP2-3: Work and dissipated energy plot.

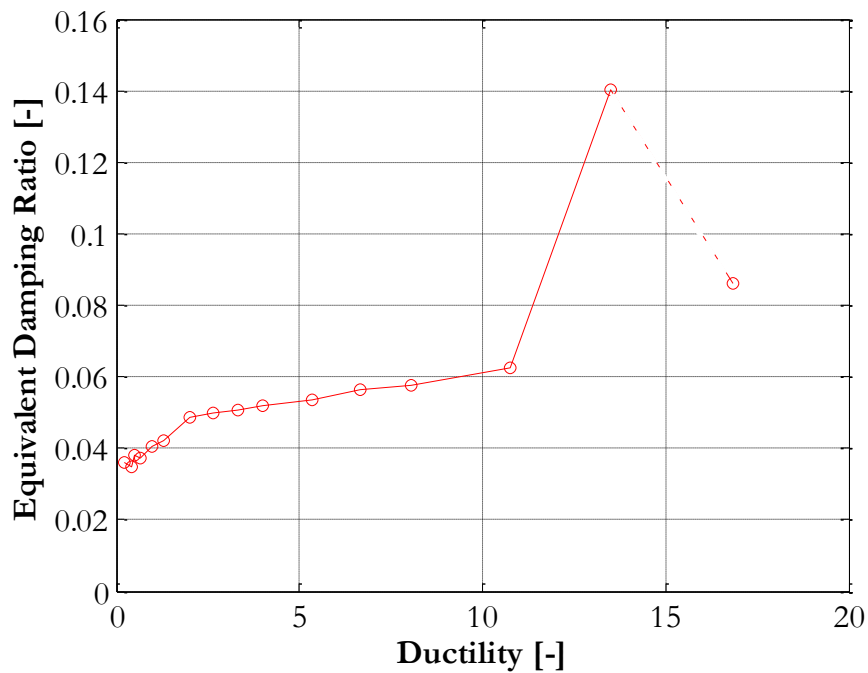


Figure 109. EC_COMP2-3: Equivalent viscous damping ratio vs. Displacement ductility.

According to the results obtained from the optical acquisition system (in particular using the coordinate time histories of the marker located at the edges of the panel), an investigation about the deformation modes of the masonry panel have been carried out. Figure 110 to Figure 112 report the results in terms of shear angular deformation (γ), flexural angular deformation ($\Delta\theta_y$) and vertical elongation (Δy), respectively.

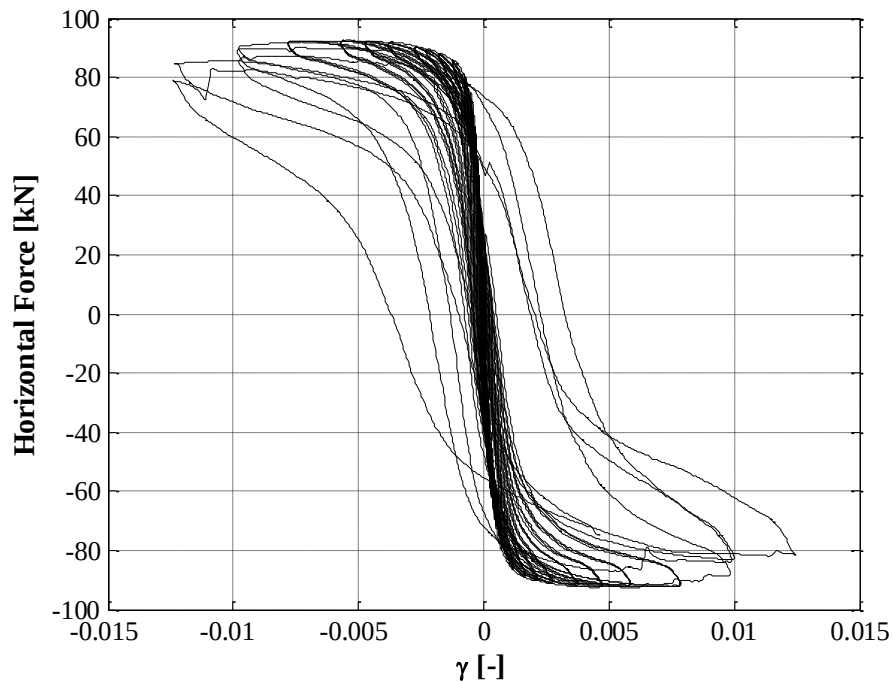


Figure 110. EC_COMP2-3: Shear deformation plot.

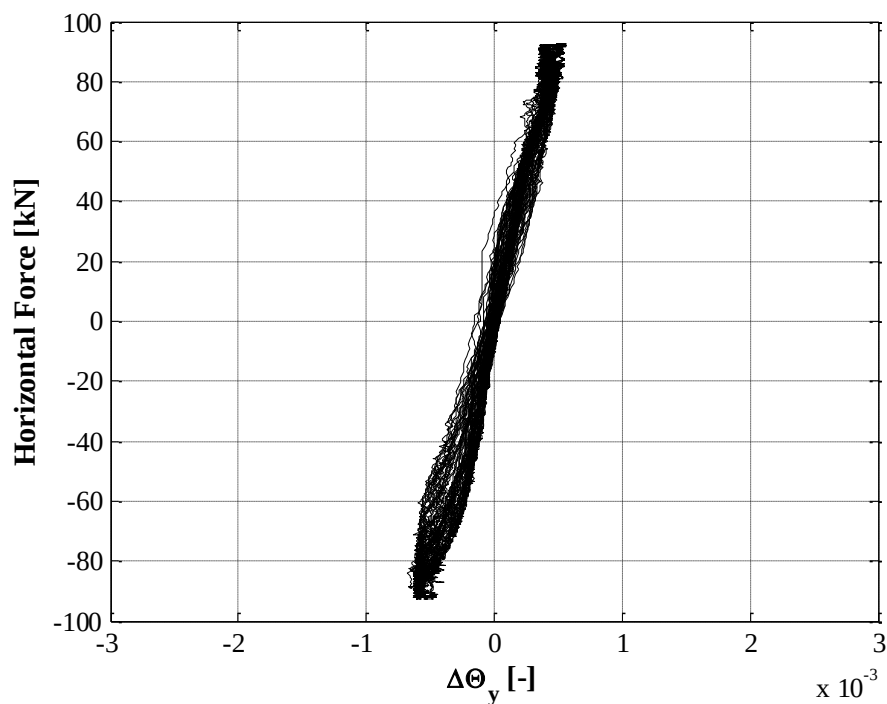


Figure 111. EC_COMP2-3: Flexural deformation plot.

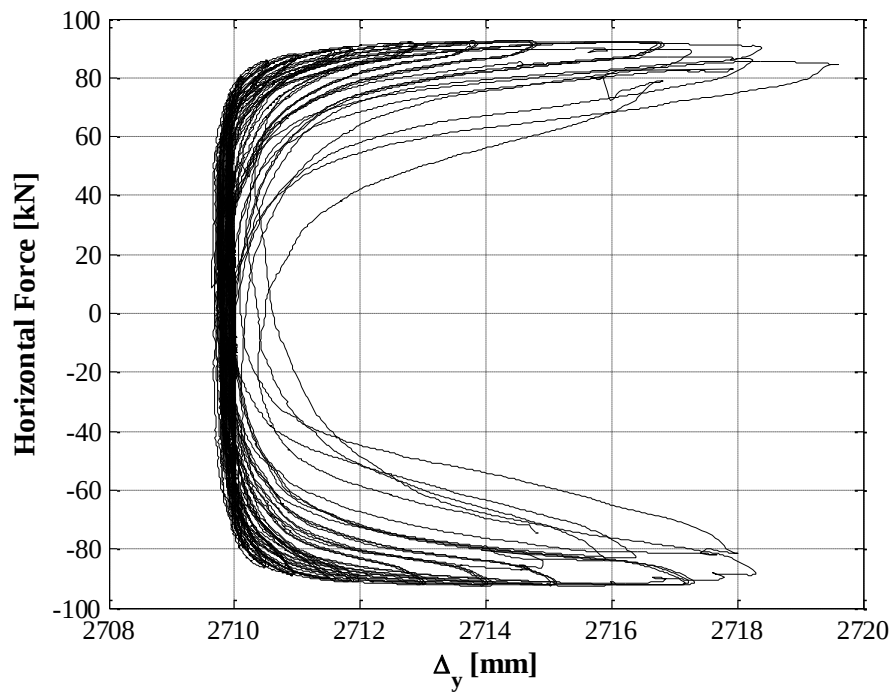


Figure 112. EC_COMP2-3: Vertical elongation plot.

4.5.4 EC_COMP2-4

EC_COMP2-4 is the first of the two squat piers to be tested. It was subjected to in-plane cyclic quasi-static tests performed under overburden of 0.3 MPa. The specimen was 2.74 m long and 2.71 m high, with double-fixed boundary conditions. Figure 113 shows the test set-up of EC_COMP2-4.

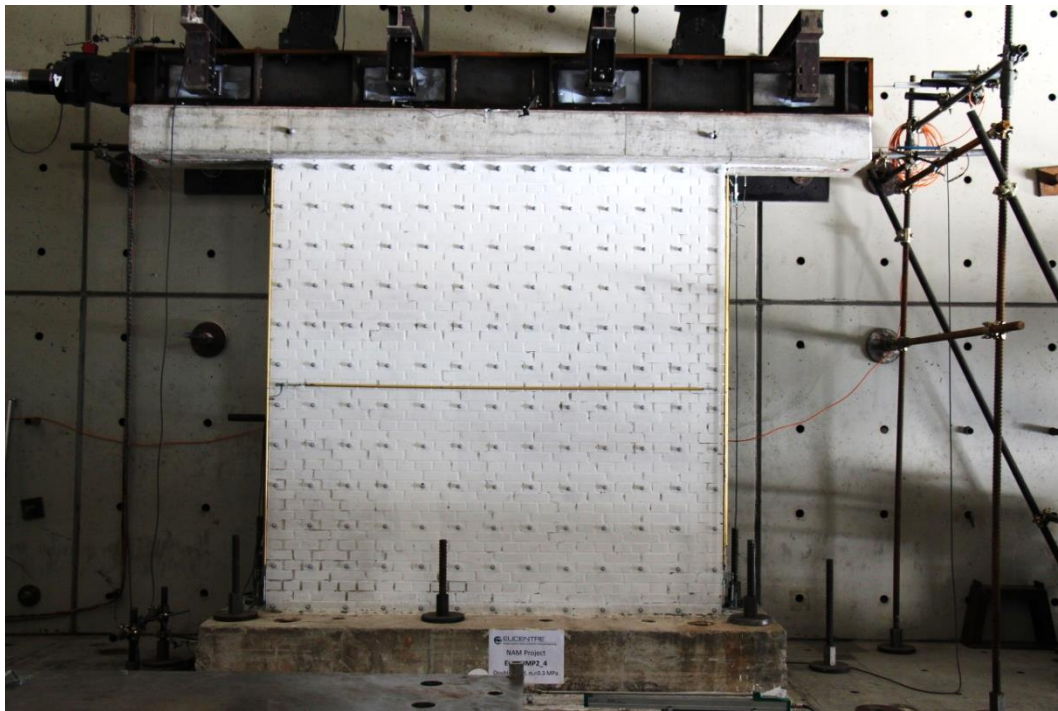
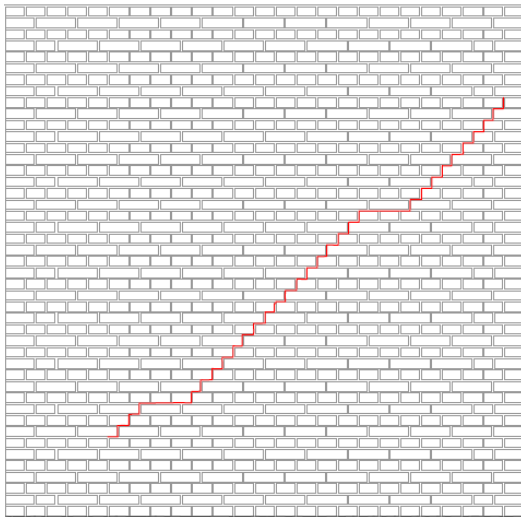


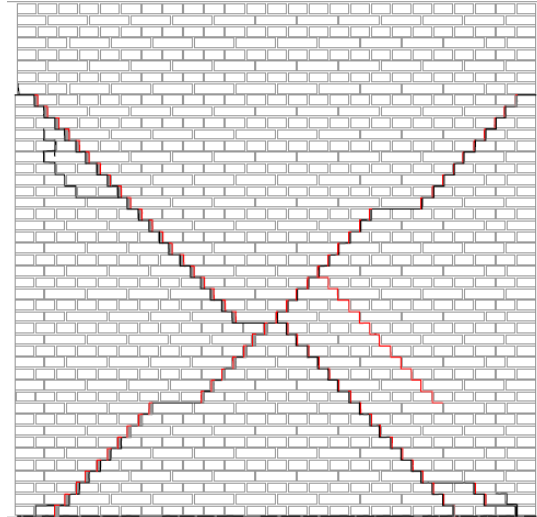
Figure 113. Test set-up of EC_COMP2-4.

The first reported cracks were identified at the wall-foundation interface (Figure 115), during loading cycle 1_5 (0.02% drift – 0.56 mm top displacement). The same cracks were elongated all along the wall base during the following cycle 1_6 (0.029% drift – 0.79 mm top displacement) and cycle 1_7 (0.038% drift – 1.04 mm top displacement). The first diagonal cracks appeared in the middle of the panel (Figure 116) during cycle 1D (0.051% drift – 1.4 mm top displacement). In the following cycles the specimen exhibits a shear behaviour with additional diagonal cracks visible in the central part of the panel. The test ended after 30% drop of the maximum lateral force, when the wall was not able anymore to carry the applied vertical load (cycle 7D drift 0.32% - top displacement 8.64 mm). The failure of the specimen was a typical example of a shear failure (Figure 117).

$\theta=0.05\%$



$\theta=0.2\%$



$\theta=0.3\%$

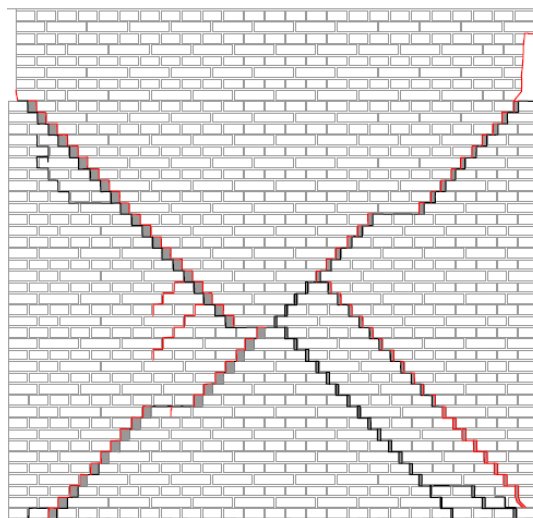


Figure 114. Illustration of the observed crack pattern for specimen EC_COMP2-4 after loading cycles 1_D, 5_D and 7_D.



Figure 115. EC_COMP2-4: Horizontal cracks along the base of the pier



Figure 116. EC_COMP2-4: Diagonal cracks appear in the middle of the pier after loading cycle 1_D.

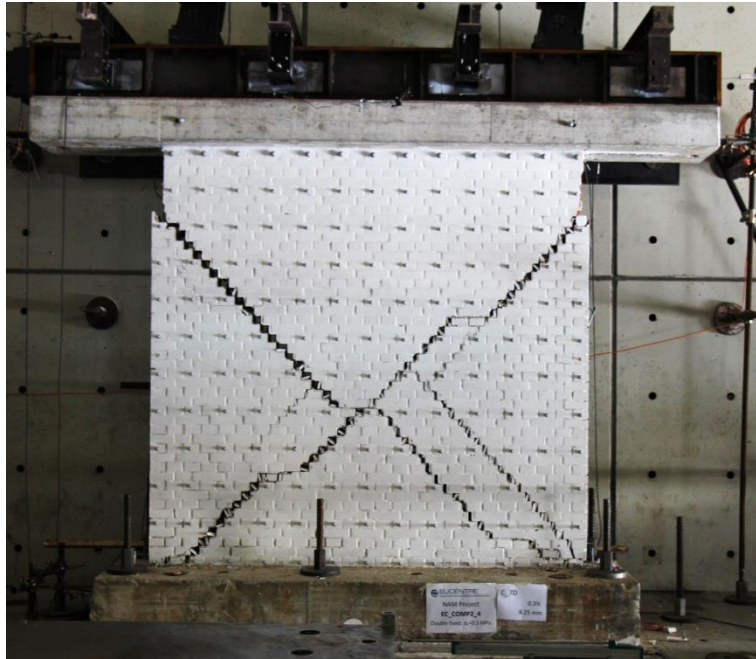


Figure 117. EC_COMP2-4: Damage pattern of the pier at the end of the test.

The hysteretic response of the pier EC_COMP2-4 is reported in Figure 118. The initial lateral stiffness of the wall was 297 kN/mm and the shear capacity 126.08 kN in the positive direction (e.g. actuator pulling) and 137.08 kN in negative direction. According to the results obtained during the vertical load application phase, an evaluation of the uncracked Young Moduli has been carried out. The computation leads to a value of 8847 MPa.

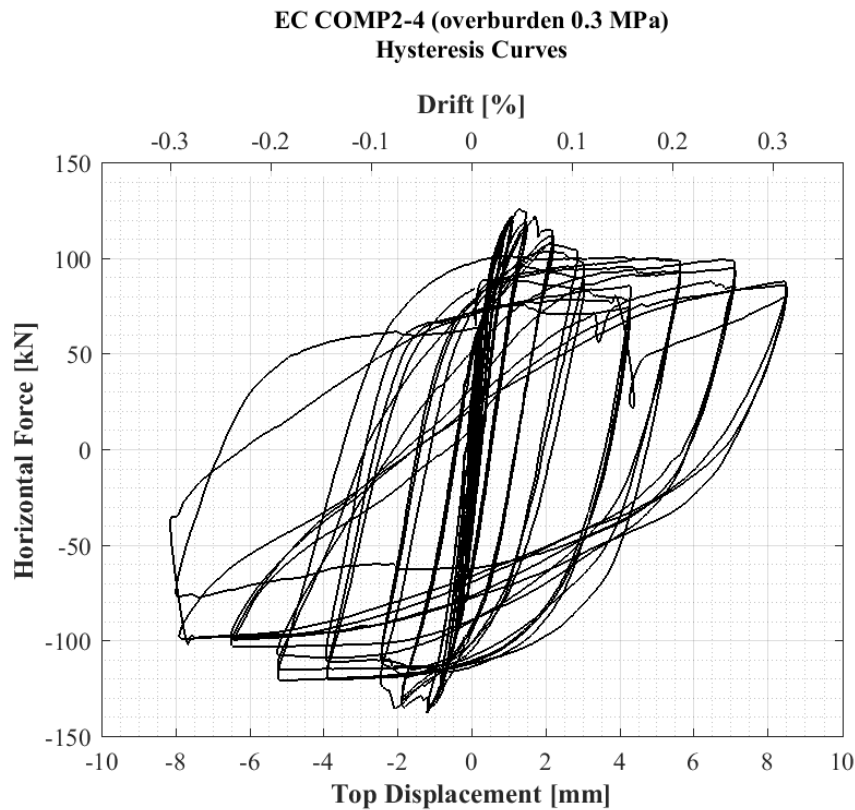


Figure 118. Hysteretic behaviour of component EC_COMP2-4.

As for the previous test, Figure 119 reports the total work computed during each cycles (the red curve represents the dissipated energy). The results of the calculated equivalent viscous damping are plotted in Figure 120 as a function of displacement ductility of each cycle.

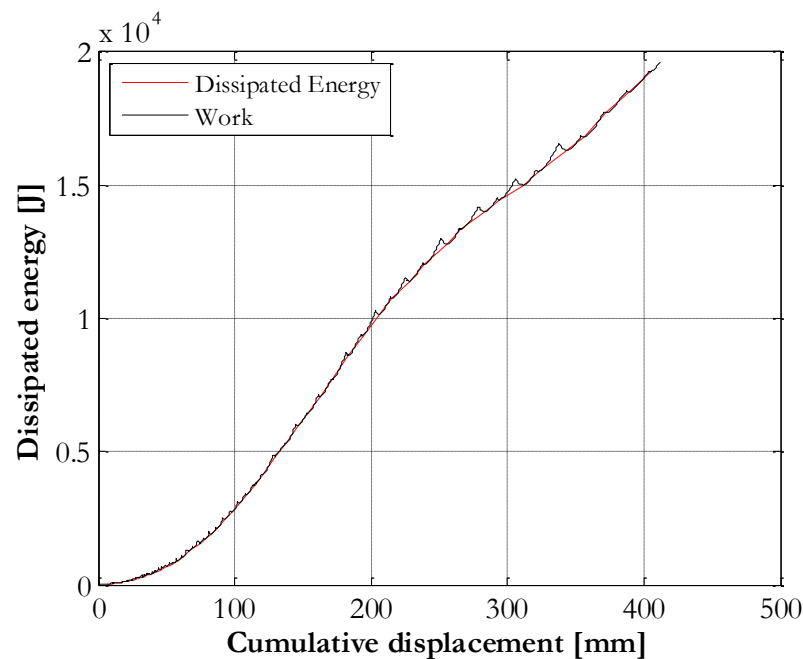


Figure 119. EC_COMP2-4: Work of horizontal force and dissipated energy plot.

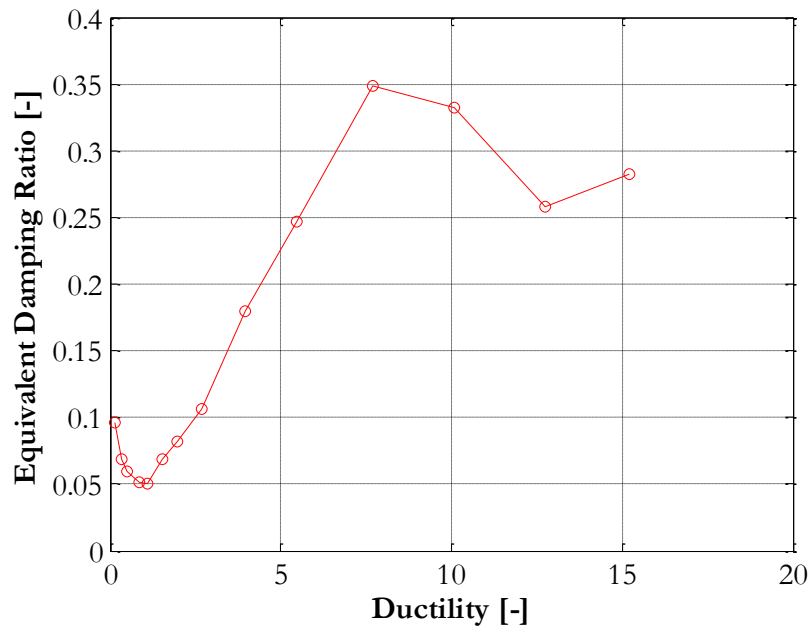


Figure 120. EC_COMP2-4: Equivalent viscous damping ratio vs. Displacement ductility.

According to the results obtained from the optical acquisition system (in particular using the coordinate time histories of the marker located at the edges of the panel), an investigation about the deformation modes of the masonry panel have been carried out. Figure 121 to Figure 123 report the results in terms of shear angular deformation (γ), flexural angular deformation ($\Delta\theta_y$) and vertical elongation (Δy), respectively.

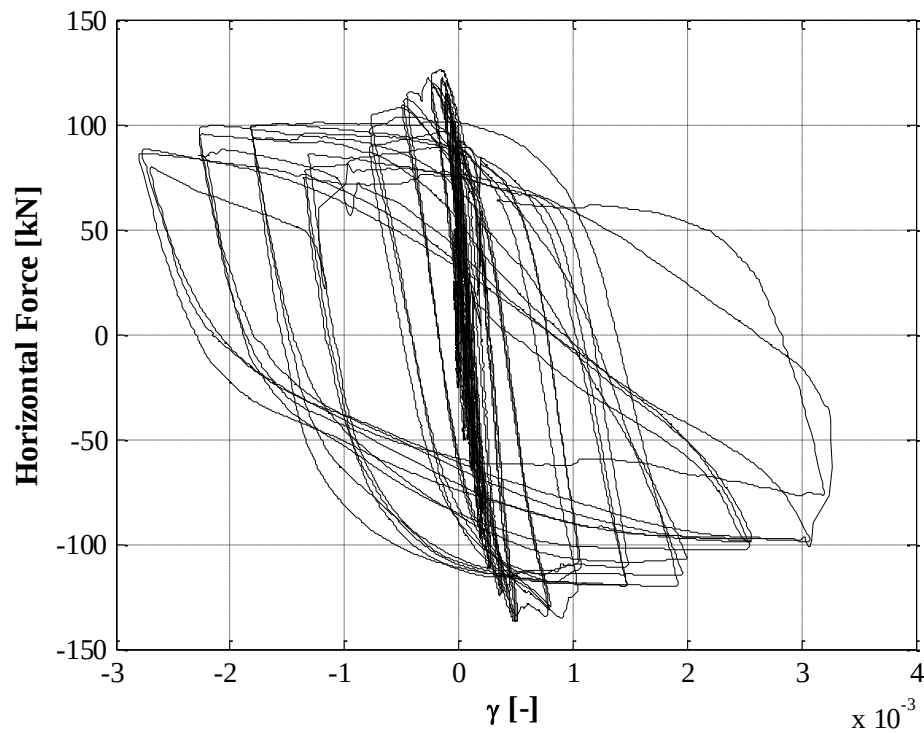


Figure 121. EC_COMP2-4: Shear deformation plot.

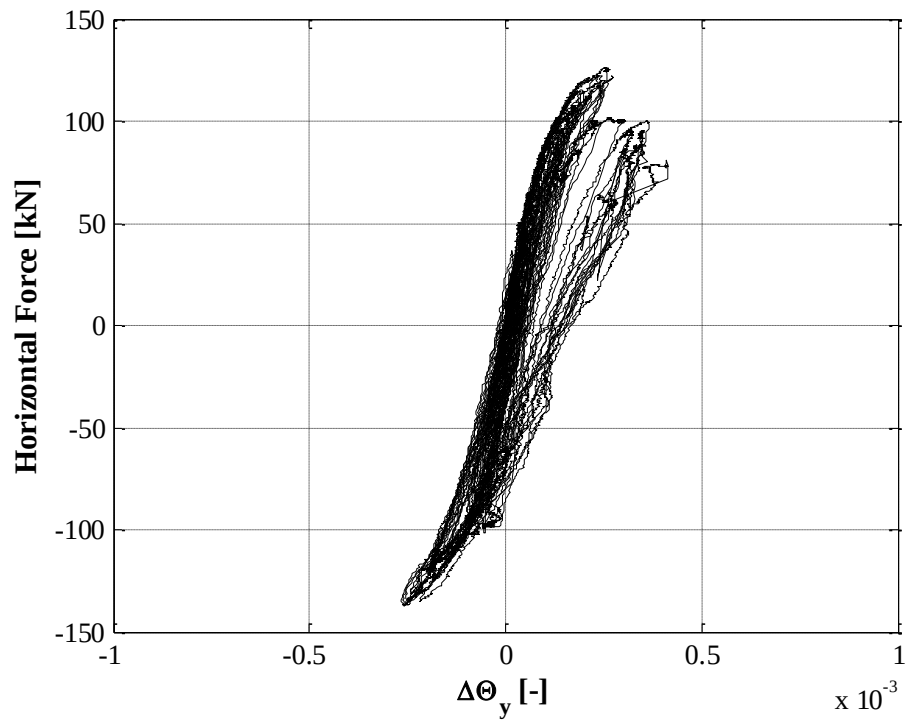


Figure 122. EC_COMP2-4: Flexural deformation plot.

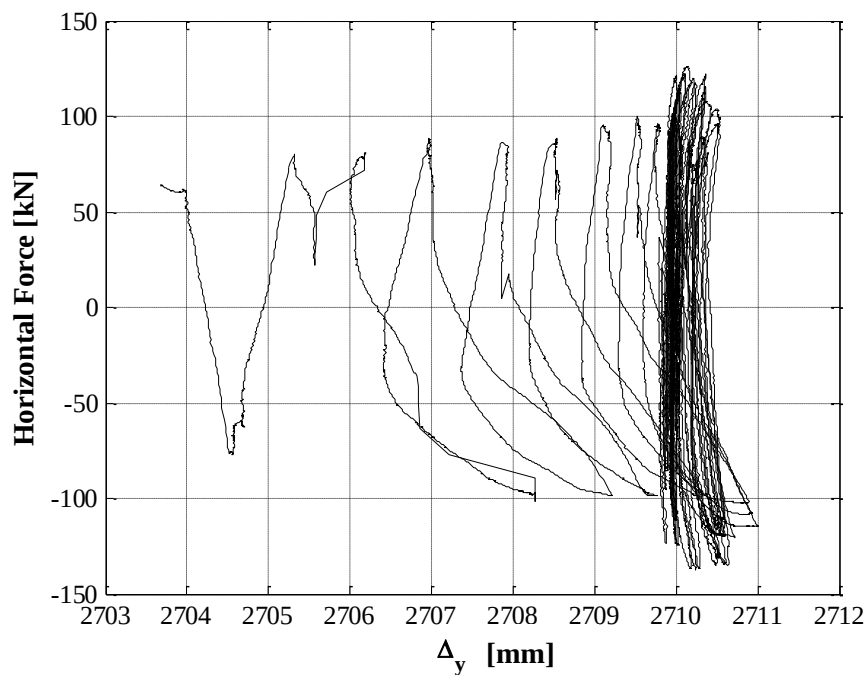


Figure 123. EC_COMP2-4: Vertical elongation plot.

Figure 124 reports the width development of the first diagonal crack formed during the loading cycle 1D, in the middle of the pier, for pushing toward left. It is a time-history plot, different cycles are identified with coloured vertical lines.

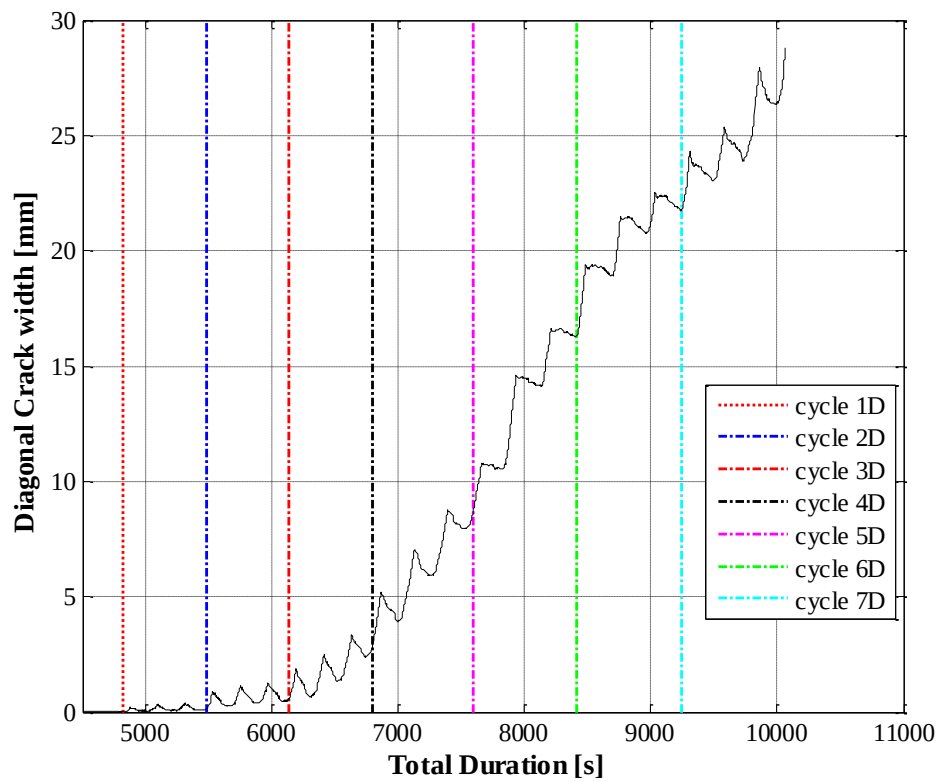


Figure 124. EC_COMP2-4: Width development of the first diagonal crack formed in the masonry panel during cycle 1D

4.5.5 EC_COMP2-5

EC_COMP2-5 is the second (last) of the two squat piers to be tested. It was subjected to in-plane cyclic quasi-static tests performed under overburden of 0.3 MPa. The specimen was 2.74 m long and 2.71 m high, with double-fixed boundary conditions. Figure 125 shows the test set-up of EC_COMP2-5.

In order to investigate the influence of the testing protocol on the results, for this last test only one cycle for each target displacement has been performed (in order to compare the results with the ones related to EC_COMP2-4).

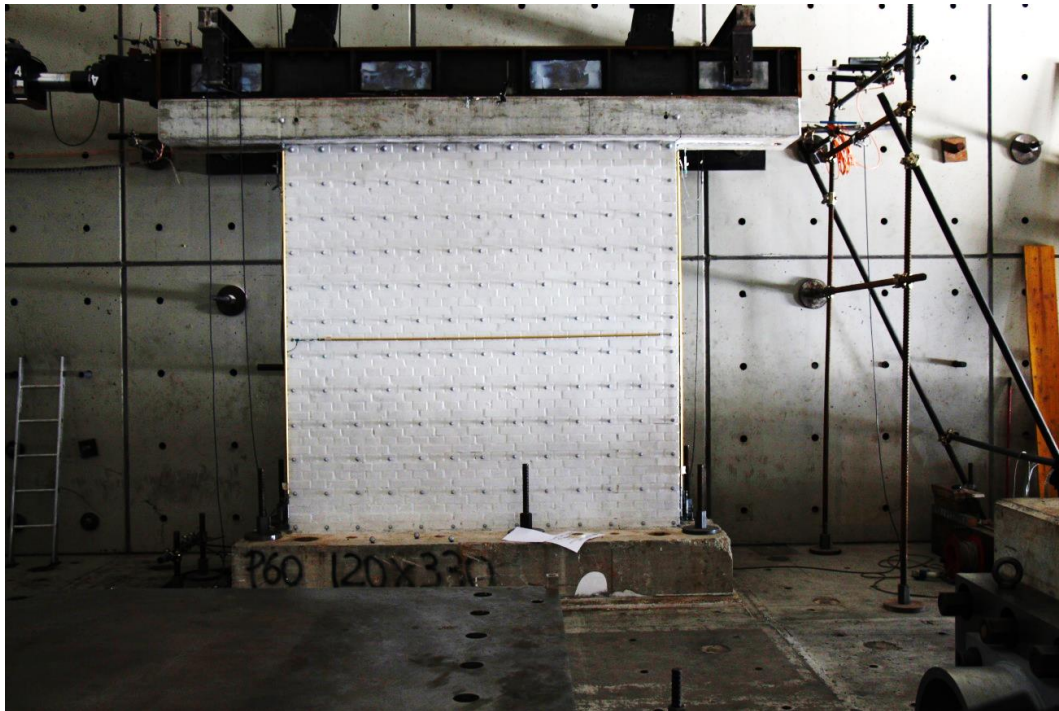


Figure 125. Test set-up of EC_COMP2-5.

A horizontal crack along the whole length of the wall, at a section just one layer of bricks above the base (Figure 127), was initially observed during loading cycle 3_D (0.05% drift - 2.71 mm top displacement). In the course of the test, the deformations were accommodated through sliding along the cracked section, a failure mode different from the one observed in EC_COMP2-4. There is no scientific evidence that the different failure mode is caused or not by the different loading protocol.

Shear failure occurred during cycle 13_D (Figure 128), while the wall could still sustain the applied vertical load after the end of cycle 15_D (drift 1.5% - 40.65 mm top displacement).

The test ended after partial collapse of the wall that occurred during cycle 17_D (2% drift - 54.2 mm top displacement), as shown Figure 129. This cycle was never completed, the procedure was stopped while pushing towards left and before reaching the peak displacement.

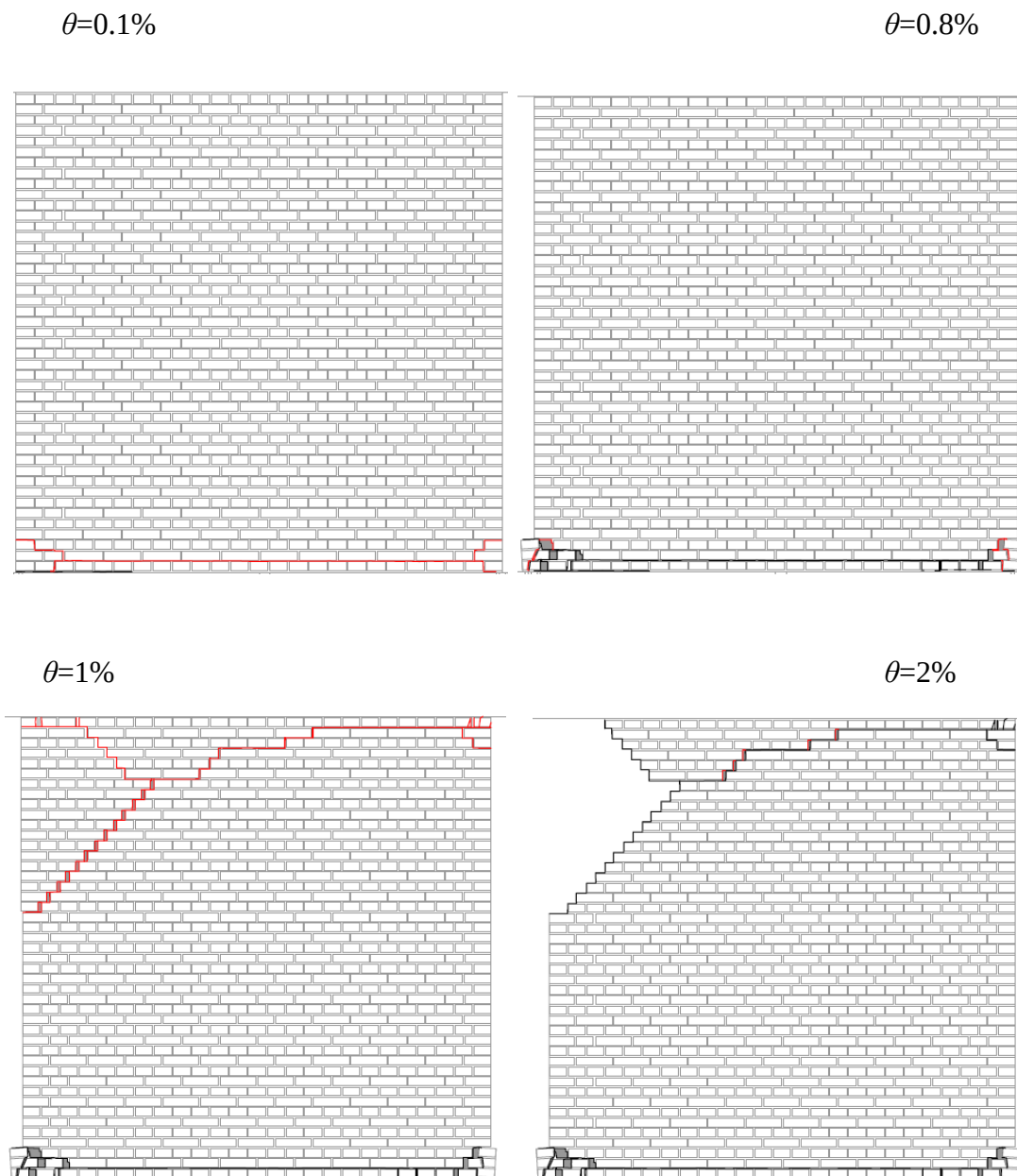


Figure 126. Illustration of the observed crack pattern for specimen EC_COMP2-5 after loading cycles 3_D, 12_D, 13_D and 17_D.



Figure 127. EC_COMP2-5: Horizontal cracks along the base of the pier.

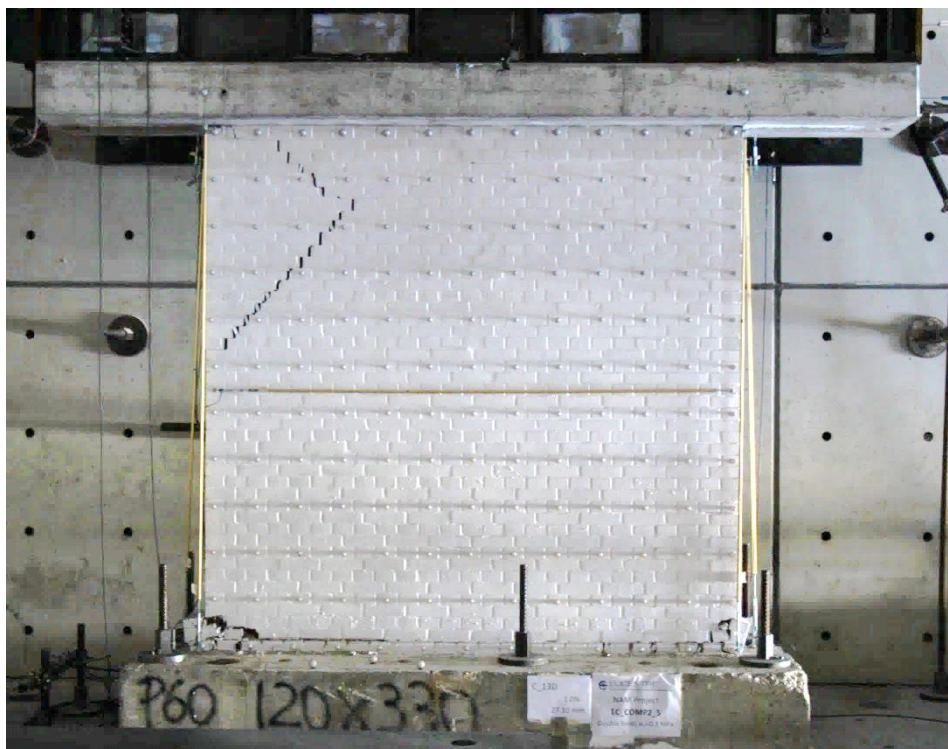


Figure 128. EC_COMP2-5: Shear failure of the pier during cycle 13D.



Figure 129. EC_COMP2-5: Damage pattern of the pier at the end of the test.

The hysteretic response of the pier EC_COMP2-5 is reported in Figure 130. The initial lateral stiffness of the wall was 324.7 kN/mm and the shear capacity 149.06 kN in the positive direction (e.g. actuator pulling) and 143.13 kN in negative direction. According to the results obtained during the vertical load application phase, an evaluation of the uncracked Young Moduli has been carried out. The computation leads to a value of 8663 MPa.

EC COMP2-5 (overburden 0.3 MPa)
Hysteresis Curves

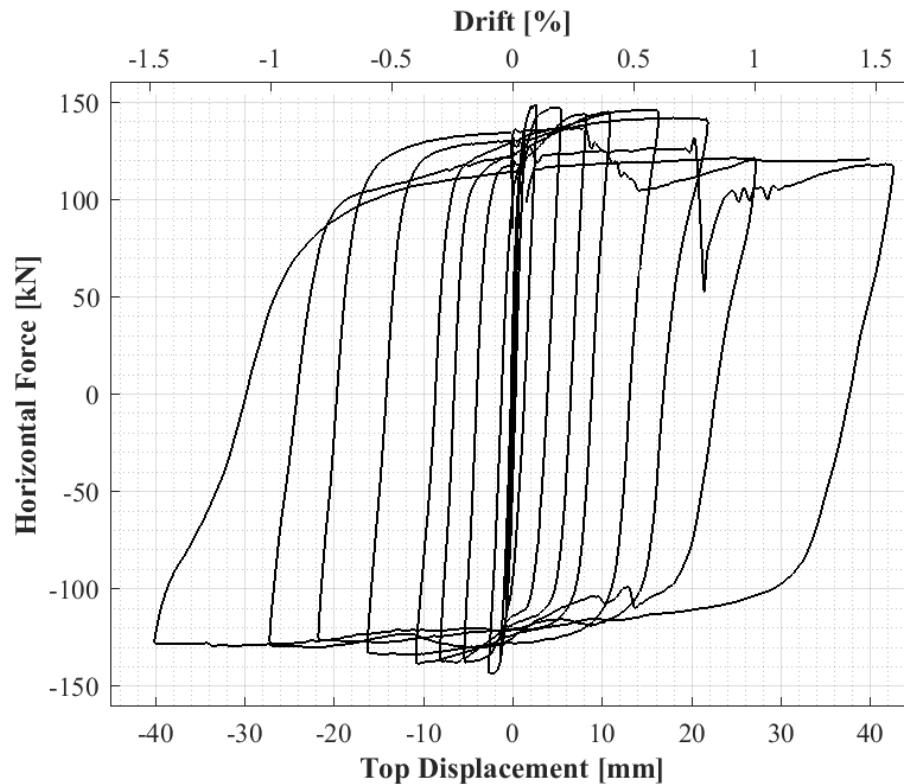


Figure 130. Hysteretic behaviour of component EC_COMP2-5.

As for the previous test, Figure 131 reports the total work computed during each cycles (the dashed curve represents the dissipated energy). The results of the calculated equivalent viscous damping are plotted in Figure 132 as a function of displacement ductility of each cycle.

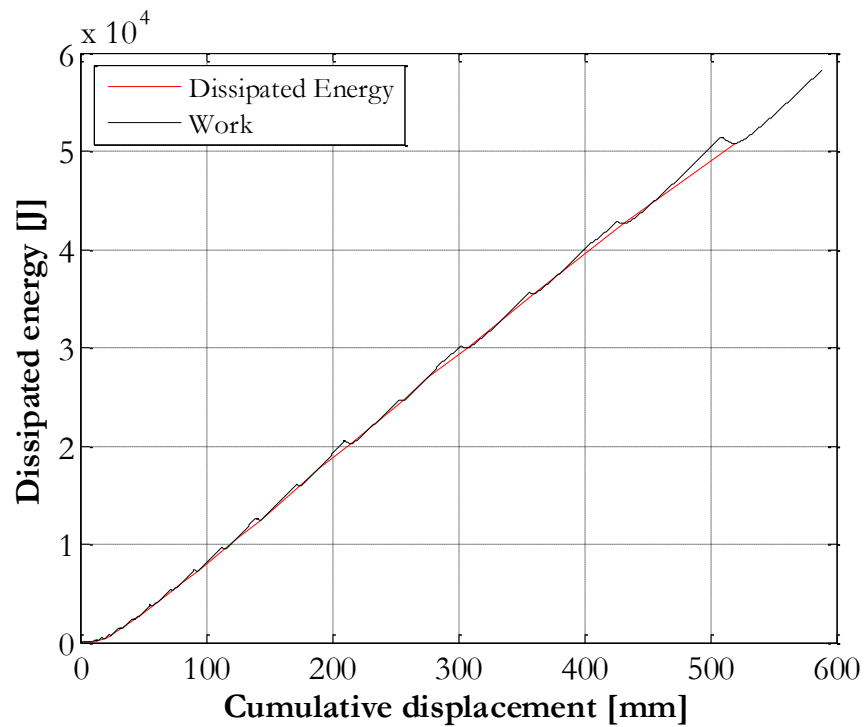


Figure 131. EC_COMP2-5: Work of the horizontal force and dissipated energy plot.

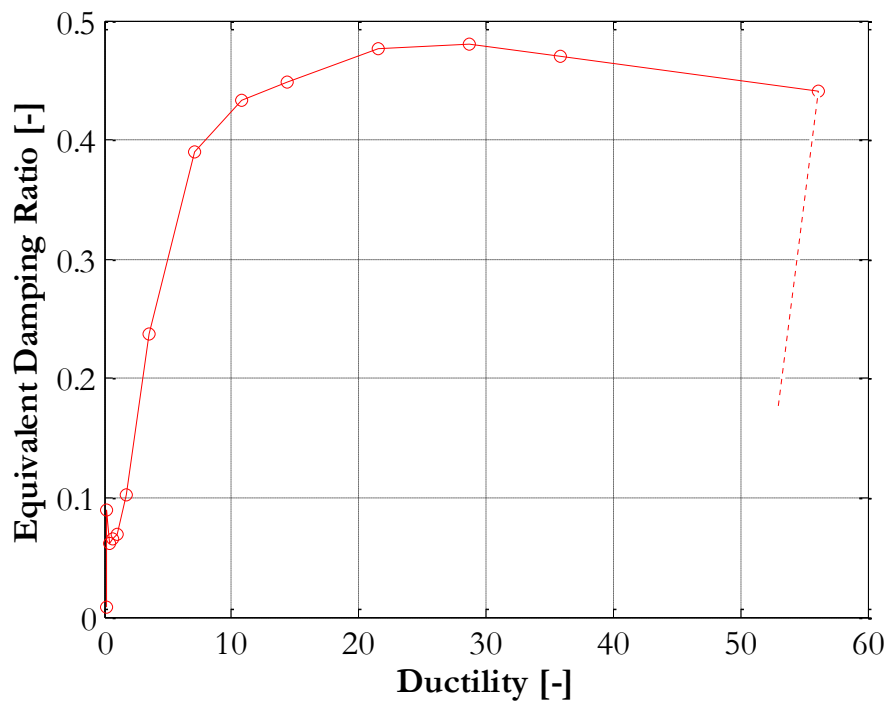


Figure 132. EC_COMP2-5: Equivalent viscous damping ratio vs. Displacement ductility.

According to the results obtained from the optical acquisition system (in particular using the coordinate time histories of the marker located at the edges of the panel), an

investigation about the deformation modes of the masonry panel have been carried out. Figure 133 to Figure 135 report the results in terms of shear angular deformation (γ), flexural angular deformation ($\Delta\theta_y$) and vertical elongation (Δy), respectively.

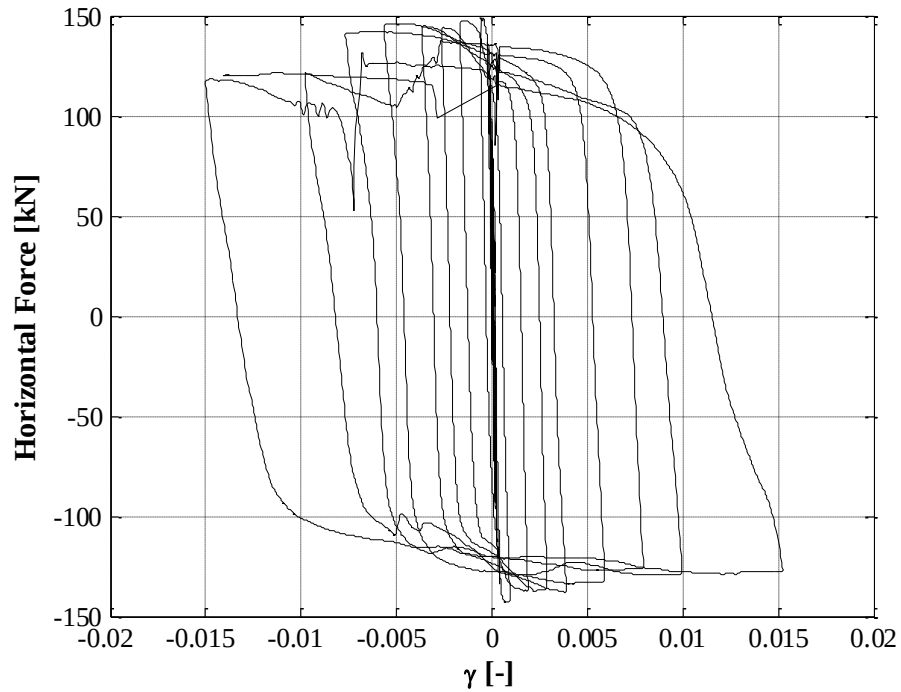


Figure 133. EC_COMP2-5: Shear deformation plot.

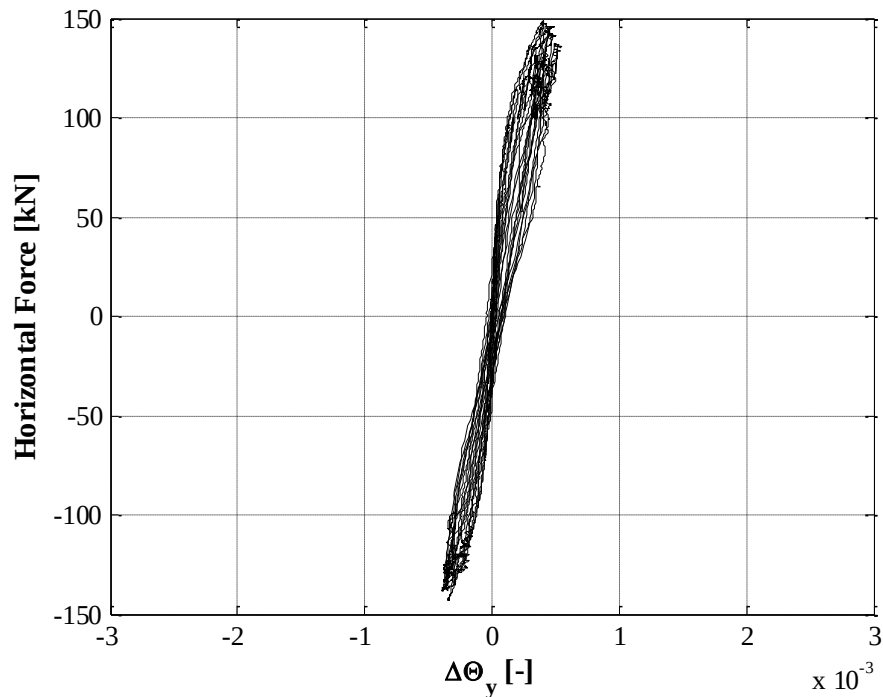


Figure 134. EC_COMP2-5: Flexural deformation plot.

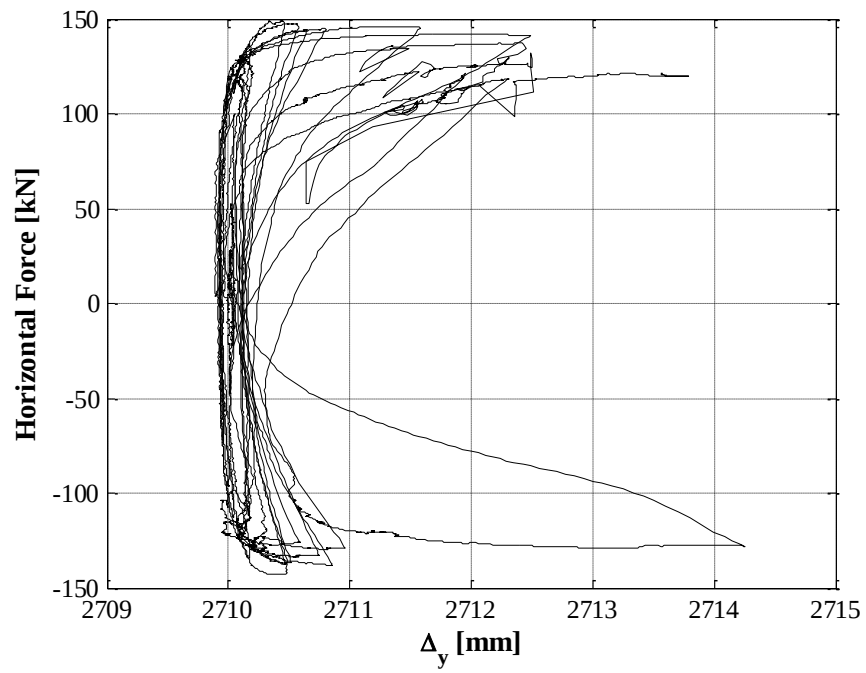


Figure 135. EC_COMP2-5: Vertical elongation plot.

4.6 Bilinear Envelopes

A common approach to interpret the in-plane response of masonry walls is to idealize the envelope of the cyclic hysteresis loops with a bilinear envelope.

The experimental envelope curve has been computed considering the attainment of the maximum absolute value of the shear force considering both positive and negative directions. Only for the very last cycle, the point of maximum lateral displacement has been included in the envelope curve.

In Figure 136 a possible definition of the parameters of the bilinear curve is given. The elastic stiffness k_{el} is obtained by drawing the secant to the experimental envelope at $0.70V_{max}$, where V_{max} is the maximum shear of the envelope.

The ultimate displacement δ_u can be evaluated as the displacement corresponding to strength degradation equal to 20% of V_{max} or as the displacement corresponding to an inability of the wall to sustain the applied vertical load. The value of the shear V_u corresponding to the horizontal branch of the bilinear curve can be found by ensuring that the areas below the cyclic envelope curve and below the equivalent bilinear curve are equal.

Knowing the elastic stiffness k_{el} and the value of V_u it is possible to evaluate the elastic displacement δ_e as V_u/k_{el} . The ultimate ductility is defined as $\mu_u = \delta_u/\delta_e$.

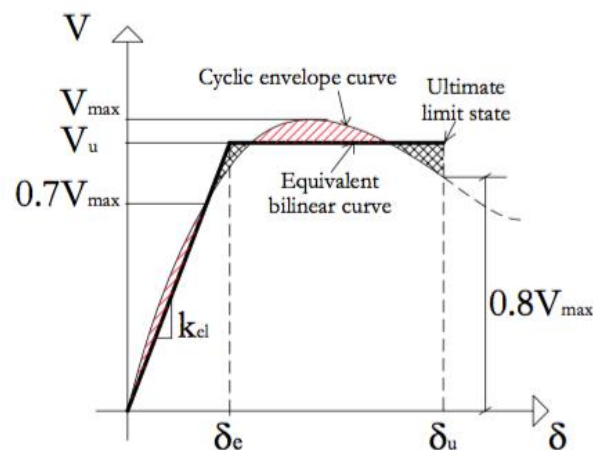


Figure 136. Bilinear idealisation of the hysteresis envelope.

The bilinear curves of the experimental global envelopes are reported in the following figures (from Figure 137 to Figure 141). The numerical values of the bilinear parameters are summarized in Table 25.

Table 25 .Parameters of the equivalent bilinear curves.

Test	V_{max}^+ [kN]	V_{min}^- [kN]	V_u^+ [kN]	V_u^- [kN]	K_{el}^+ [kN/mm]	K_{el}^- [kN/mm]	δ_{el}^+ [mm]	δ_{el}^- [mm]
EC_COMP2_1	59.1	-58.4	55.7	-56.0	49.8	49.3	1.119	-1.137
EC_COMP2_2	120.0	-122.7	117.9	-120.7	46.2	53.7	2.549	-2.246
EC_COMP2_3	92.6	-92.6	89.9	-89.2	46.5	41.1	1.932	-2.170
EC_COMP2_4	126.1	-137.1	111.4	-126.3	191.9	238.1	0.581	-0.530
EC_COMP2_5	149.0	-143.1	135.2	-132.4	161.9	193.1	0.835	-0.686
	δ_u^+ [mm]	δ_u^- [mm]	μ_u^+ [-]	μ_u^- [-]	$\mu_{u,min}$ [-]	δ_u^+/H [%]	δ_u^-/H [%]	$(\delta_u/H)_n^{mi}$ [%]
EC_COMP2_1	110.240	-82.156	98.5	72.2	72.2	4.1	-3.0	-3.0
EC_COMP2_2	34.333	-27.165	13.5	12.1	12.1	1.3	-1.0	-1.0
EC_COMP2_3	34.213	-34.492	17.7	15.9	15.9	1.3	-1.3	-1.3
EC_COMP2_4	6.452	-5.440	11.1	10.3	10.3	0.2	-0.2	-0.2
EC_COMP2_5	37.430	-39.939	44.8	58.2	44.8	1.4	-1.5	-1.5

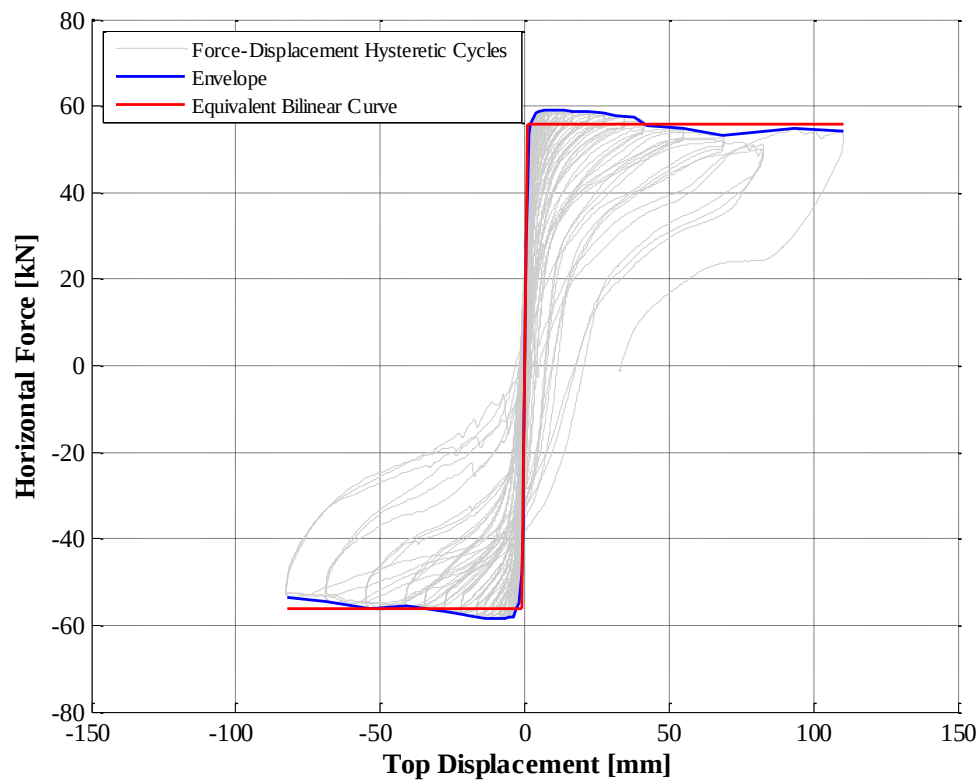


Figure 137. EC_COMP2-1: Bilinear idealisation of the hysteresis envelope

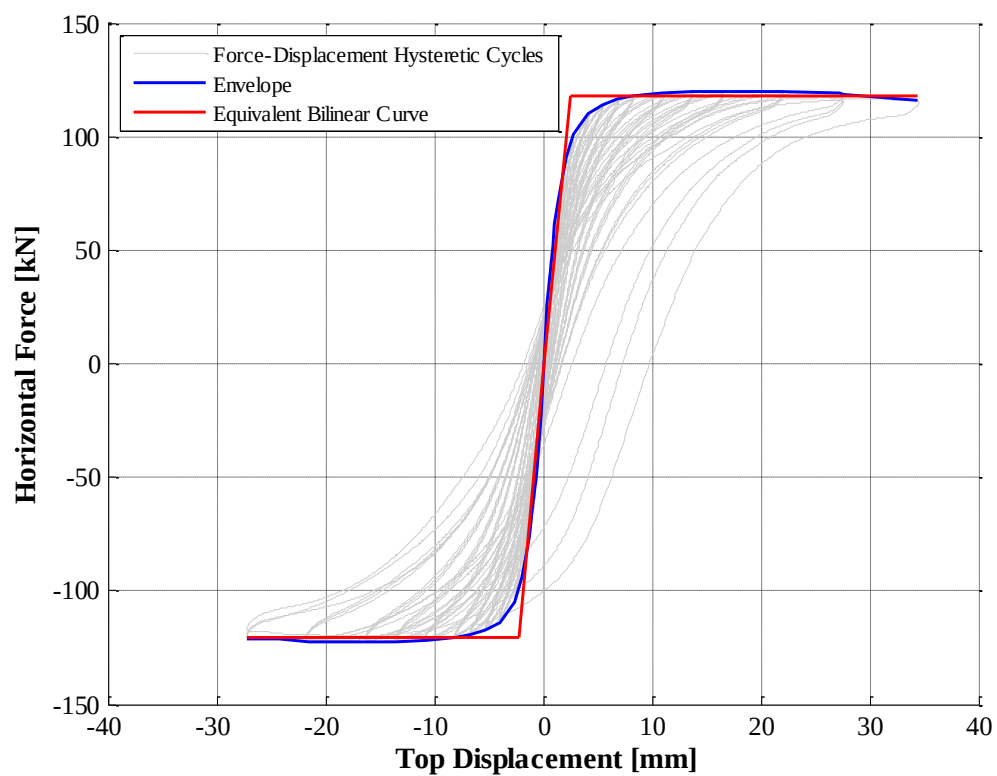


Figure 138. EC_COMP2-2: Bilinear idealisation of the hysteresis envelope

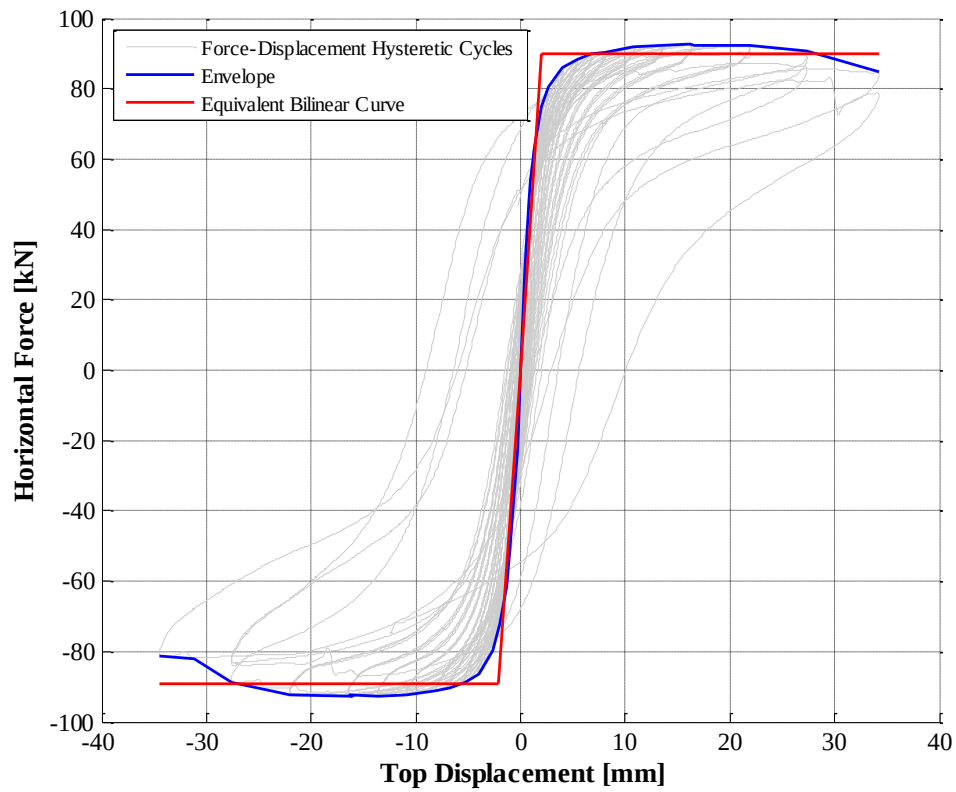


Figure 139. EC_COMP2-3: Bilinear idealisation of the hysteresis envelope

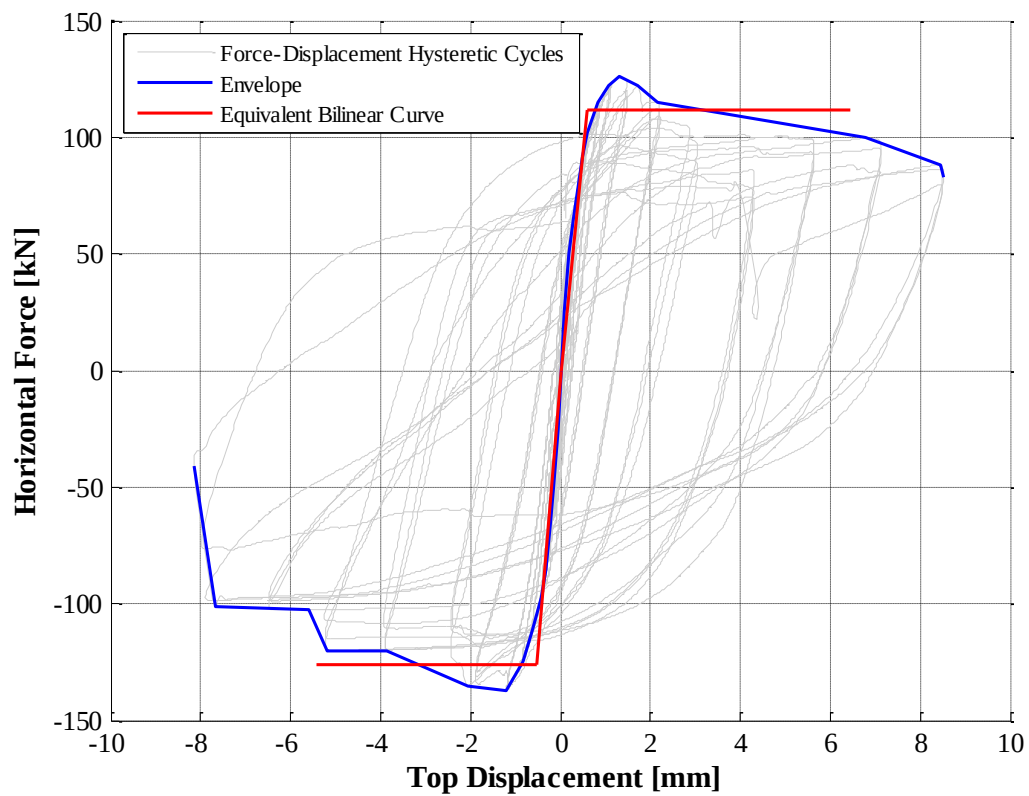


Figure 140. EC_COMP2-4: Bilinear idealisation of the hysteresis envelope

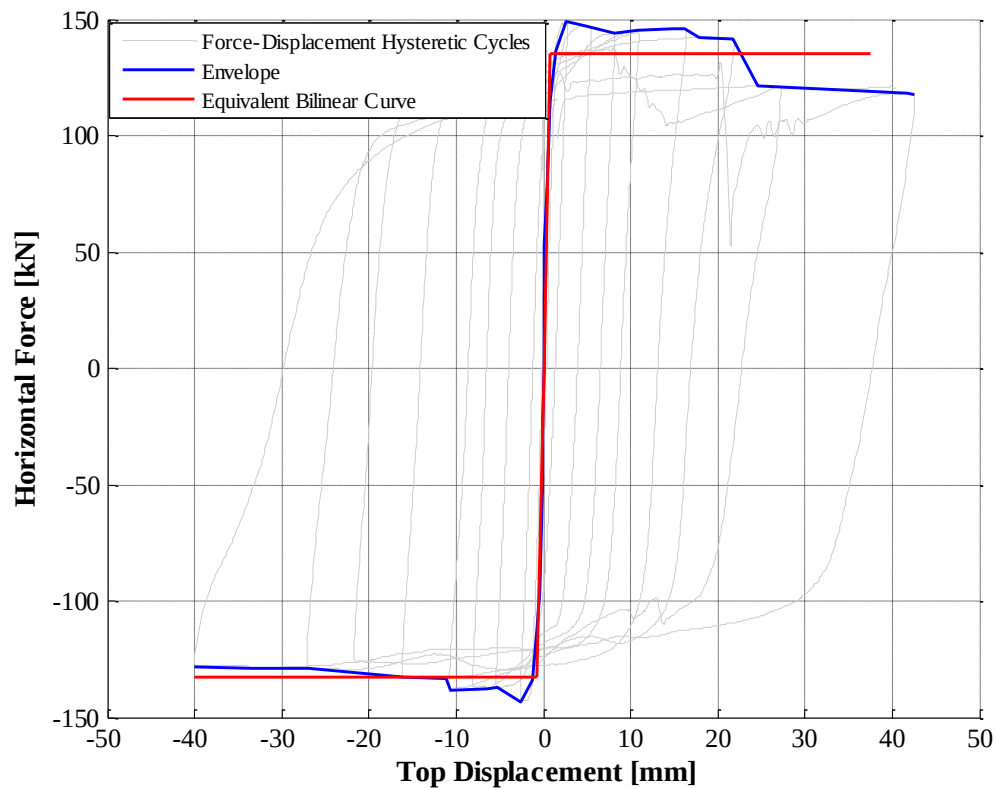


Figure 141. EC_COMP2-5: Bilinear idealisation of the hysteresis envelope

Figure 142 and Figure 143 report the comparison between the bilinear idealizations of the slender and squat walls, respectively. While the same comparison in terms of experimental envelope is reported in Figure 144 and Figure 145.

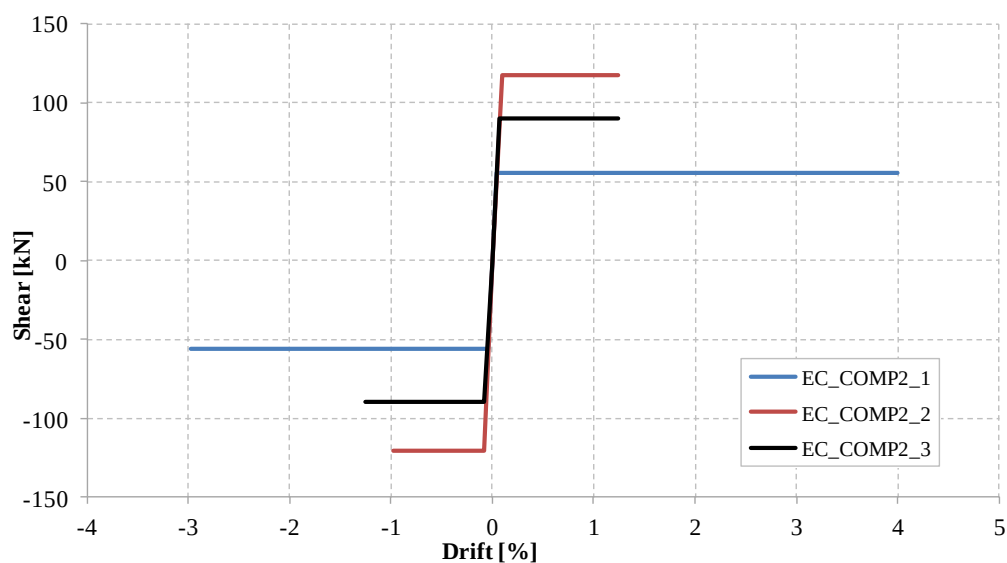


Figure 142. Comparison between the bilinear curves of the slender piers

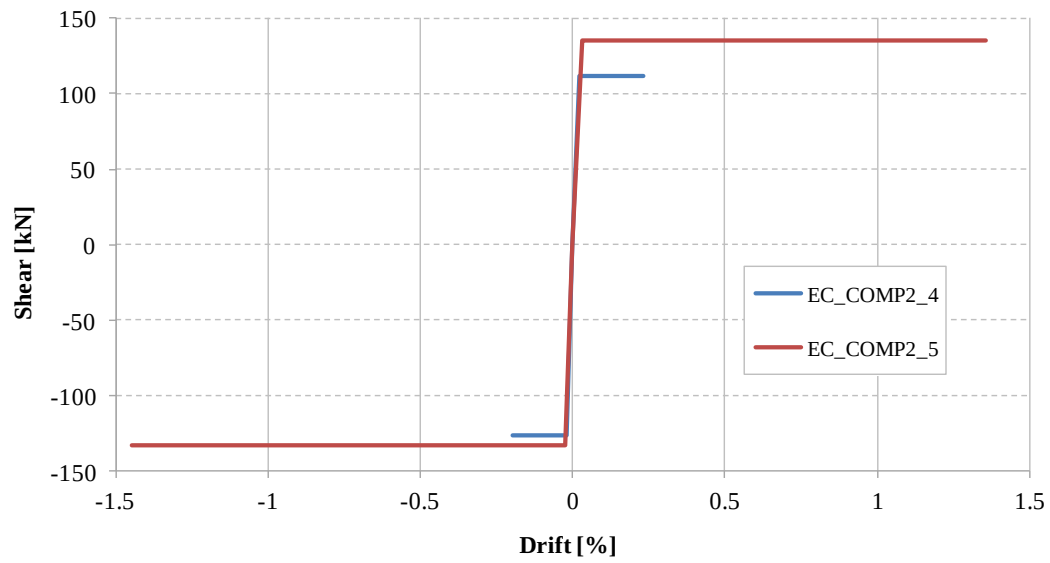


Figure 143. Comparison between the bilinear curves of the squat piers

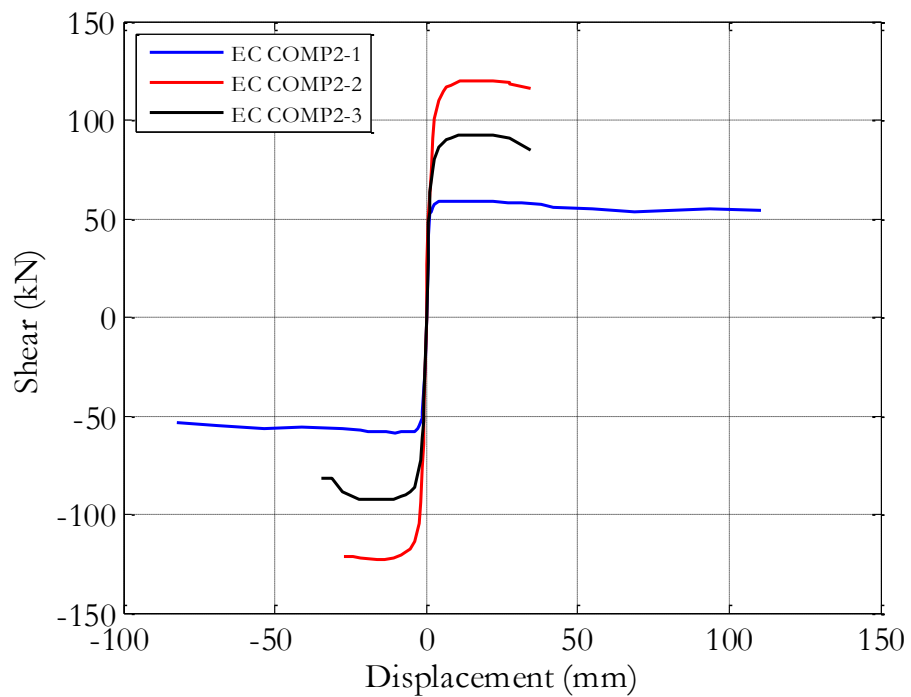


Figure 144. Comparison between the envelope curves of the slender piers

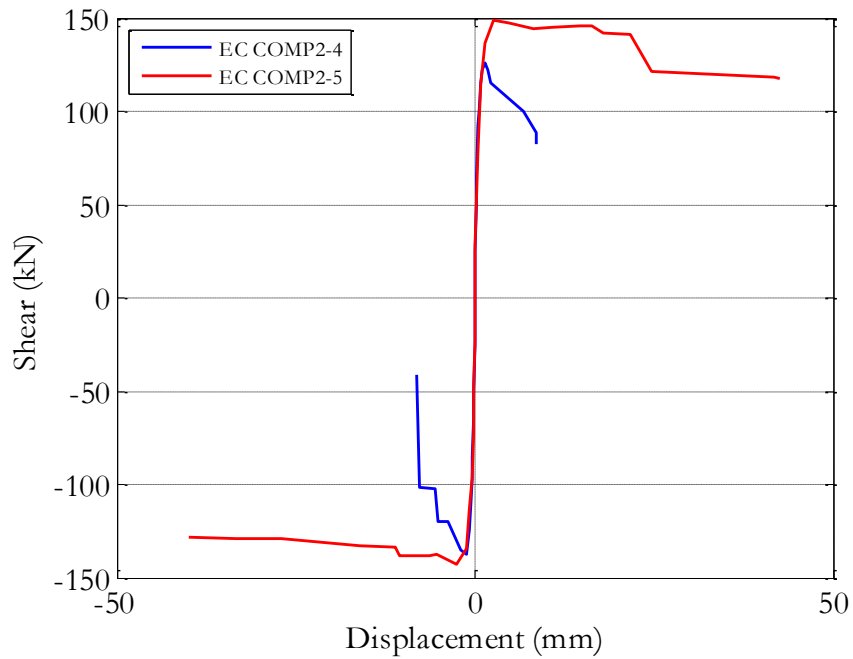


Figure 145. Comparison between the envelope curves of the squat piers

Figure 146 and Figure 147 report the comparison between the dissipated energy of the slender and squat walls, respectively.

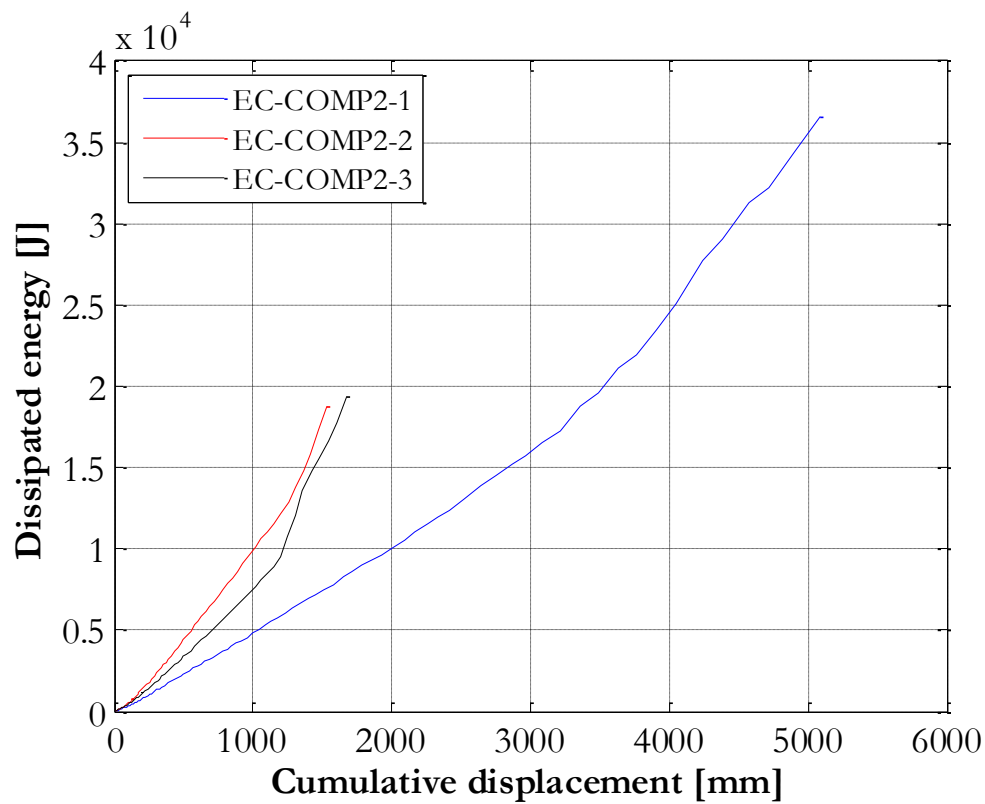


Figure 146. Comparison between the dissipated energy of the slender piers

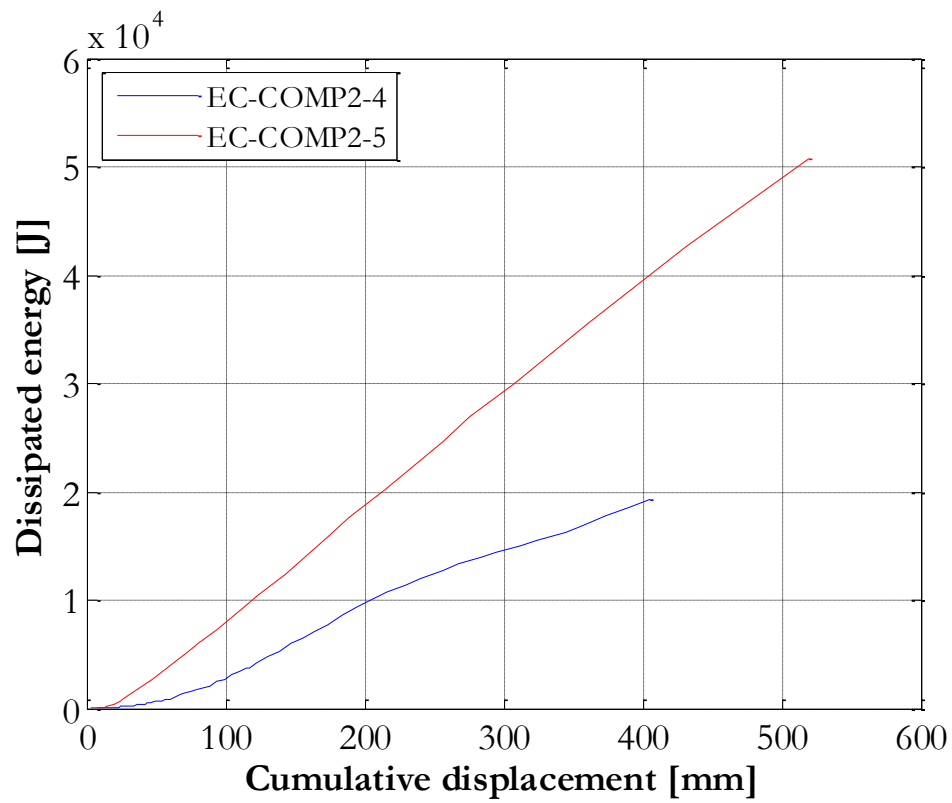


Figure 147. Comparison between the dissipated energy of the squat piers

5 Full-scale building test

This work aims to investigate the seismic behavior of a particular typology of Dutch residential building (a pre-1940 URM house). A shaking table test on a single-story full scale unreinforced masonry building was performed at the EUCENTRE laboratory within a comprehensive research programme on the seismic vulnerability of existing Dutch Unreinforced Masonry (URM) structures. The building specimen was meant to be representative a detached house built with clay solid walls and without any particular seismic design or detailing. The roof as well as the floor were composed by flexible timber diaphragms. Two different gables defined the roof ends, one full gable (South façade) and one half hipped gable (North façade). Both gables had very low vertical overburden force, being the roof carried by rafters spanning in transversal direction. An incremental dynamic test was carried out up to the near collapse limit state of the specimen. The input motions were selected to be representative of the dynamic characteristics of induced seismicity ground motions.

The report describes the characteristics of the building and presents the results obtained during the characterization and the shaking table tests, illustrating the response of the structure, the damage mechanism and its evolution during the experimental phases.

5.1 Geometry and general characteristics of the test-house

As previously mentioned, the reference single-storey full-scale test-building is a prototype structure representative of a detached house, common in the region of Groningen. It was built directly on the shaking table at the EUCENTRE Lab. (see Figure 148). The structure has been adapted to the need imposed by the laboratory constraints and it is not expected to be exhaustive of all possible variations in geometry present in the actual building stock in Groningen area, but it was designed to combine several common features of detached houses pre 1940's (Arup 2016b).

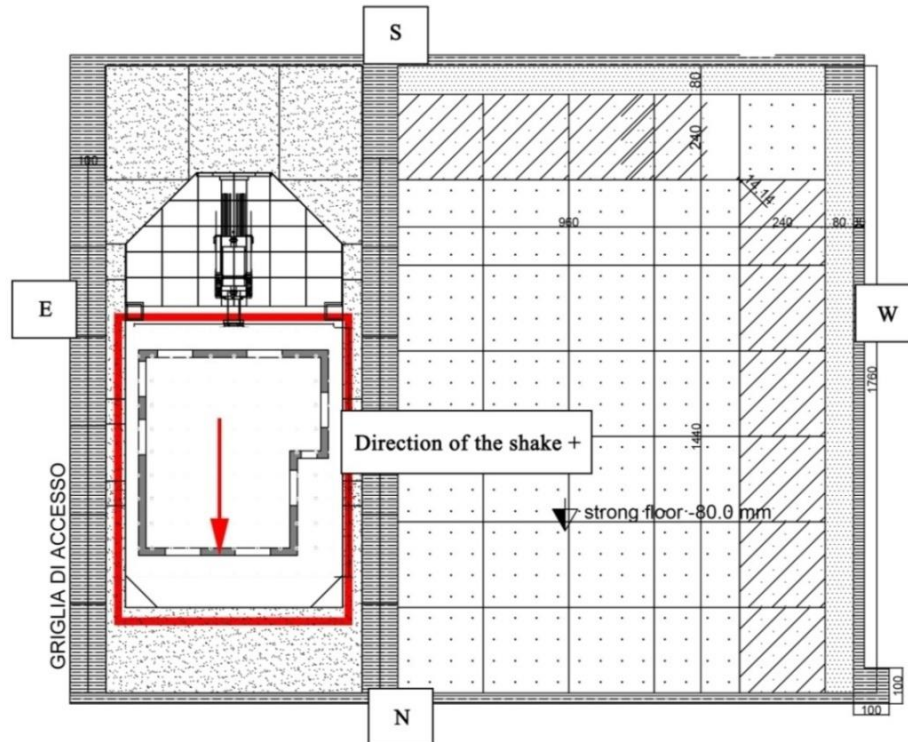


Figure 148. Plan view of the EUCENTRE Lab. and position of the shake-table and the walls of the test-house.

Horizontal irregularities and complexity in the roof structure are common features in the existing structures clustered in this typology, the reduced size of the building, due to the lab constraints without a reduction in scale determined a limitation in the variations. Furthermore, a simplified structure ensures a more effective post-processing of the test records and a consequent increased reliability of the comparison with the numerical models.

The prototype building has been constructed on rigid steel foundations; the walls are all founded on a 11 cm thick layer of concrete (Mapei Mapefill 50), contained in steel formworks, that run above the foundation grid, to which are bolted. The latter is made of HE 300B beams, reinforced approximately every 25 cm with welded steel plates, that are arranged perpendicularly to the longitudinal axis of the beams (see Figure 152). The specimen is composed by a first-floor timber diaphragm and a pitched timber roof finished with clay tiles. 208-mm-thick, double-wythe masonry walls support floor and roof framing members. The same walls extend above the first floor to form 208-mm-

thick gables at the front and back of the building. The front gable does not extend through the roof ridge, as a half-hipped roof is completing the façade.

The plan dimensions of the specimen are 5.33 m x 5.77 m, with a height of about 6.23m. The building plan is characterized by a re-entrant corner, causing a discontinuity in one of the longitudinal masonry walls, a configuration that is expected to magnify torsional effects under uniaxial seismic excitation, compared to a more regular layout (see Figure 154).

Figure 149 show examples of actual buildings similar to the test specimen with their typical details.



Figure 149. Buildings with construction details similar to the one adopted for the shake-table test specimen.

As mentioned above, the load-bearing structural system is provided by 208-mm-thick, double-wythe unreinforced masonry walls. The brickwork bond adopted is the one called Dutch Cross bond (see Figure 151), that is exactly like the most common English Cross bond except in the generating of the lap at the quoins. The English Cross bond is characterized by having alternating stretching and heading courses, with the headers centered over the midpoint of the stretchers. The stretching courses alternate between a course composed entirely of stretchers, and a course composed of stretchers half off-

set relative to the stretchers two courses above or below, by reason of a header placed just before the quoins at either end. In Dutch bond, instead, all quoins are three-quarter bats, placed in alternately stretching and heading orientation with successive courses, and no use whatever is made of queen closers.

The exact brick dimensions are 208 x 100 x 50 mm, while the mortar joints are approximately 10 mm thick.



Figure 150. Clay brick dimensions (in mm).

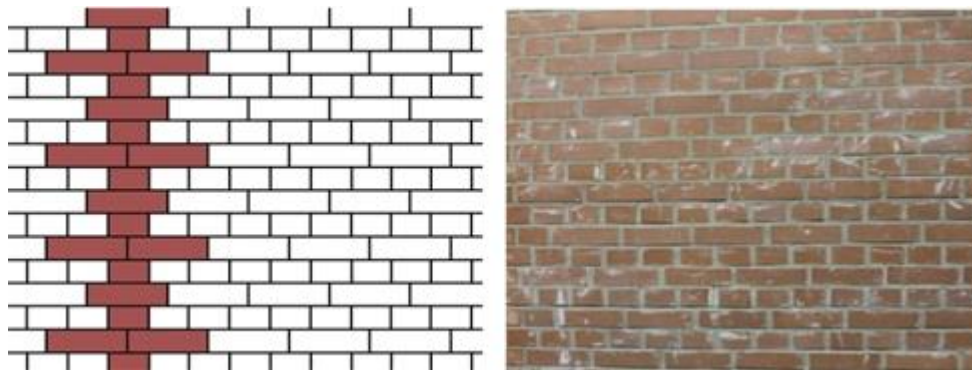


Figure 151. Dutch Cross bond scheme (left), brick bond pattern of the specimen walls (right).



Figure 152. Test-house perimeter walls during the construction works and detail of the foundation.

Regarding the mortar used for the construction of the walls, the following figures show its identification code and properties.



Figure 153. Type of mortar used and ID code.

Moreover, the percentage of water with respect to the weight of the mortar is indicated in following Table.

Table 26. Mortar for clay brick walls - content of water.

Water	Mortar	W/M
(lt=kg)	(kg) - one bag	(% weight)
3.7	25	14.80%

The following figure shows the geometry of the test-house (placed on the foundation) and its dimension (in cm).

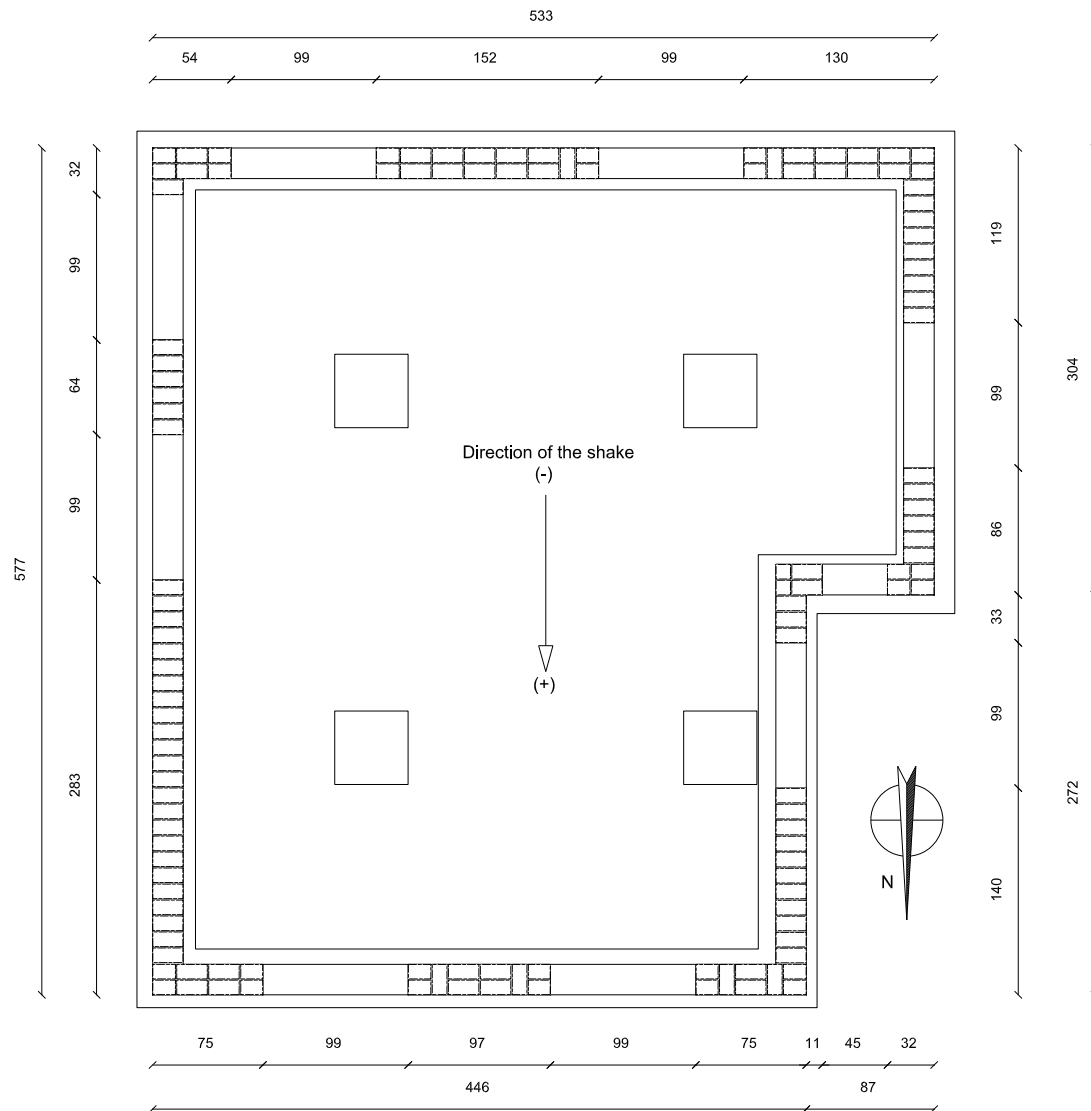


Figure 154. Plan view of the test-house, ground floor (in cm).

The following figures show a plan view of the walls of the building, placed on the foundation beams, with the first four bricks courses layout.

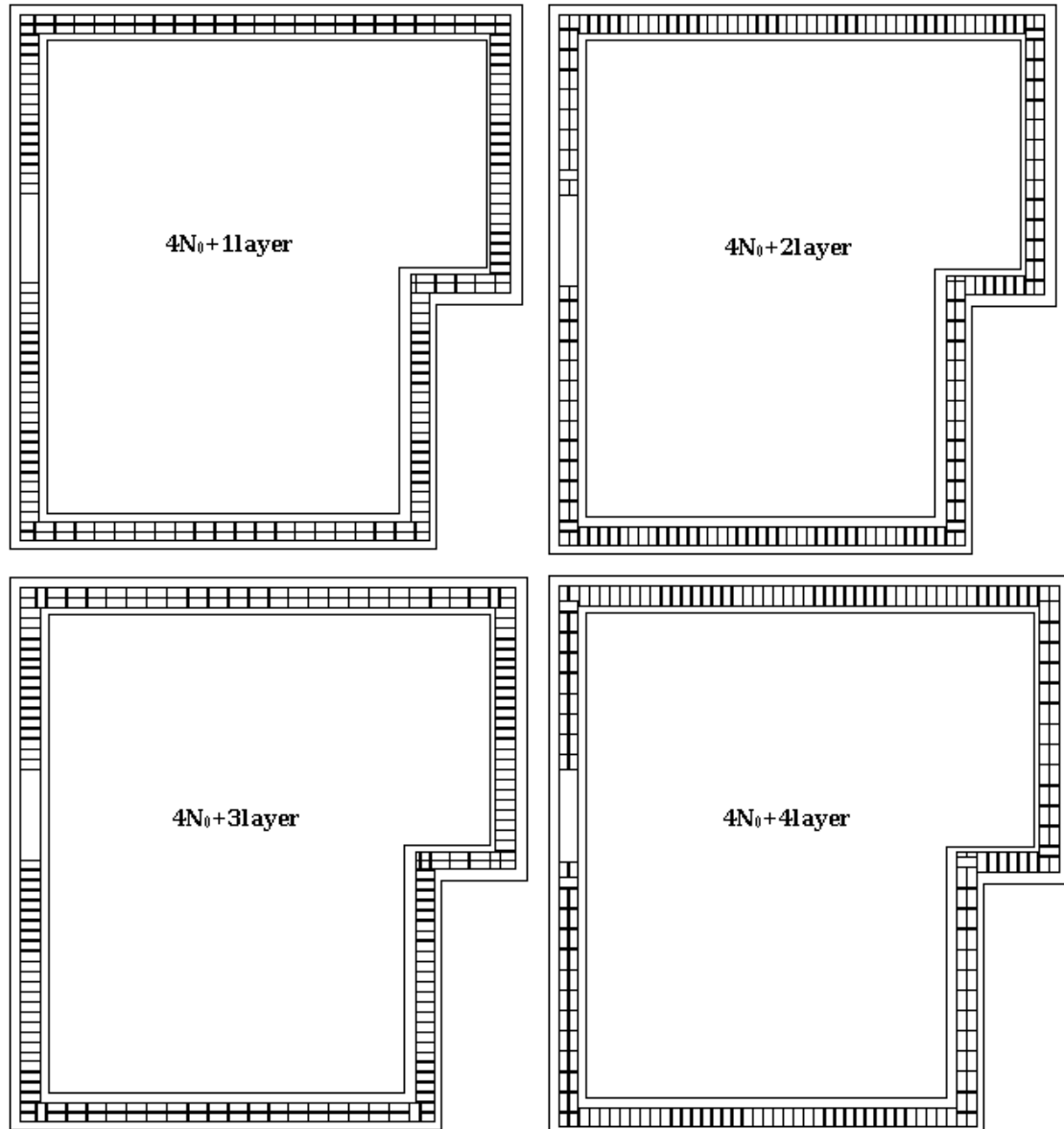


Figure 155. First four bricks courses layout.

The following figures show the elevation views of the test-house and its dimensions (in cm).

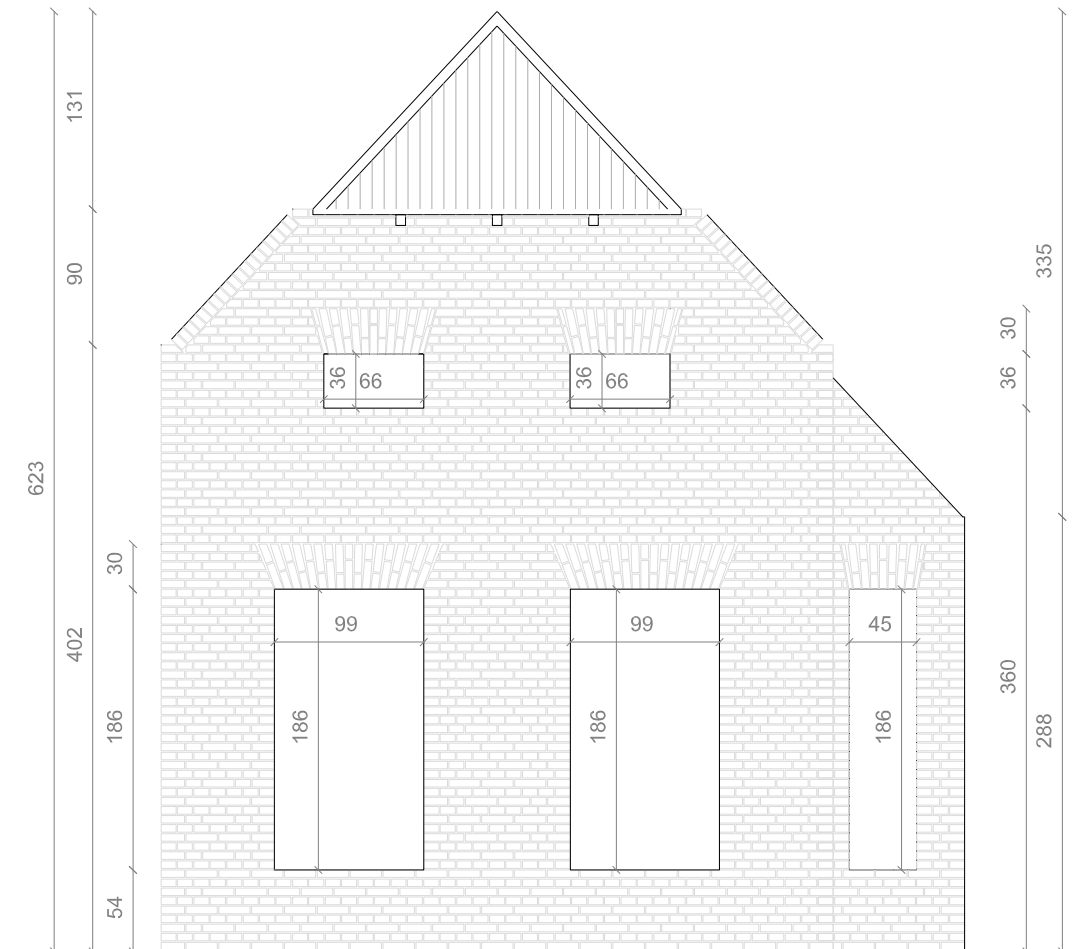


Figure 156. Elevation view of the test-house - North side (in cm).

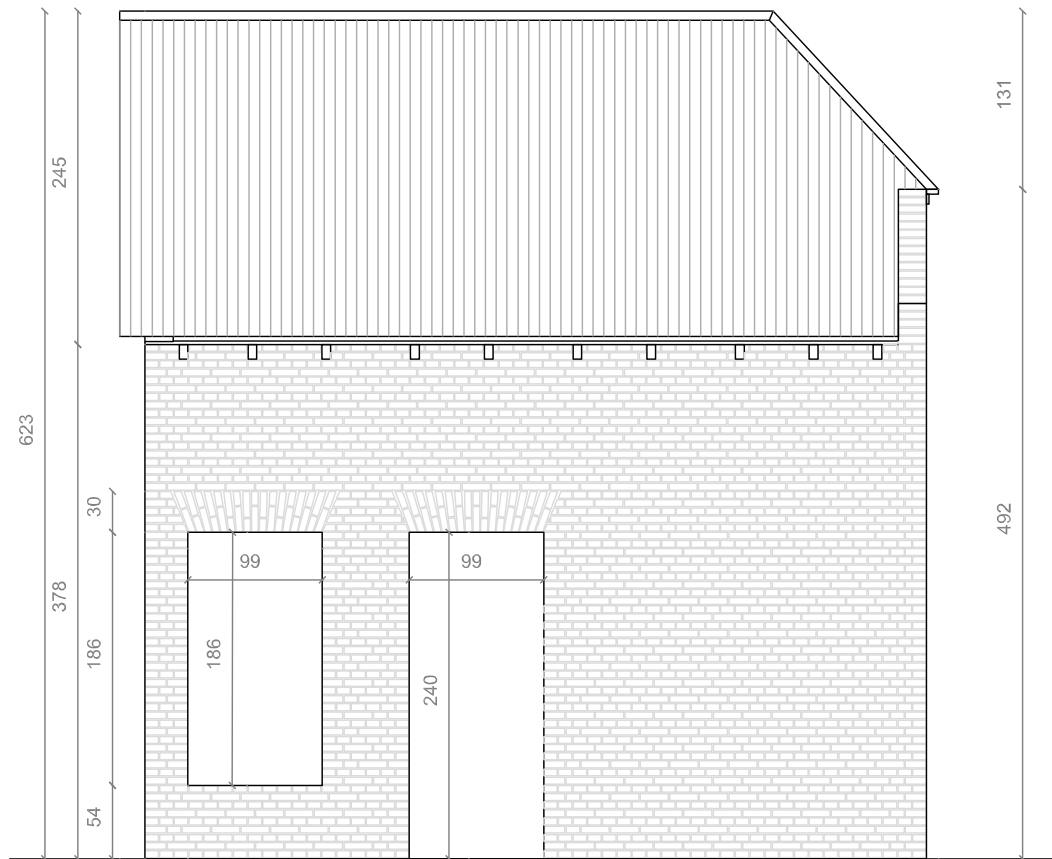


Figure 157. Elevation view of the test-house - East side (in cm).

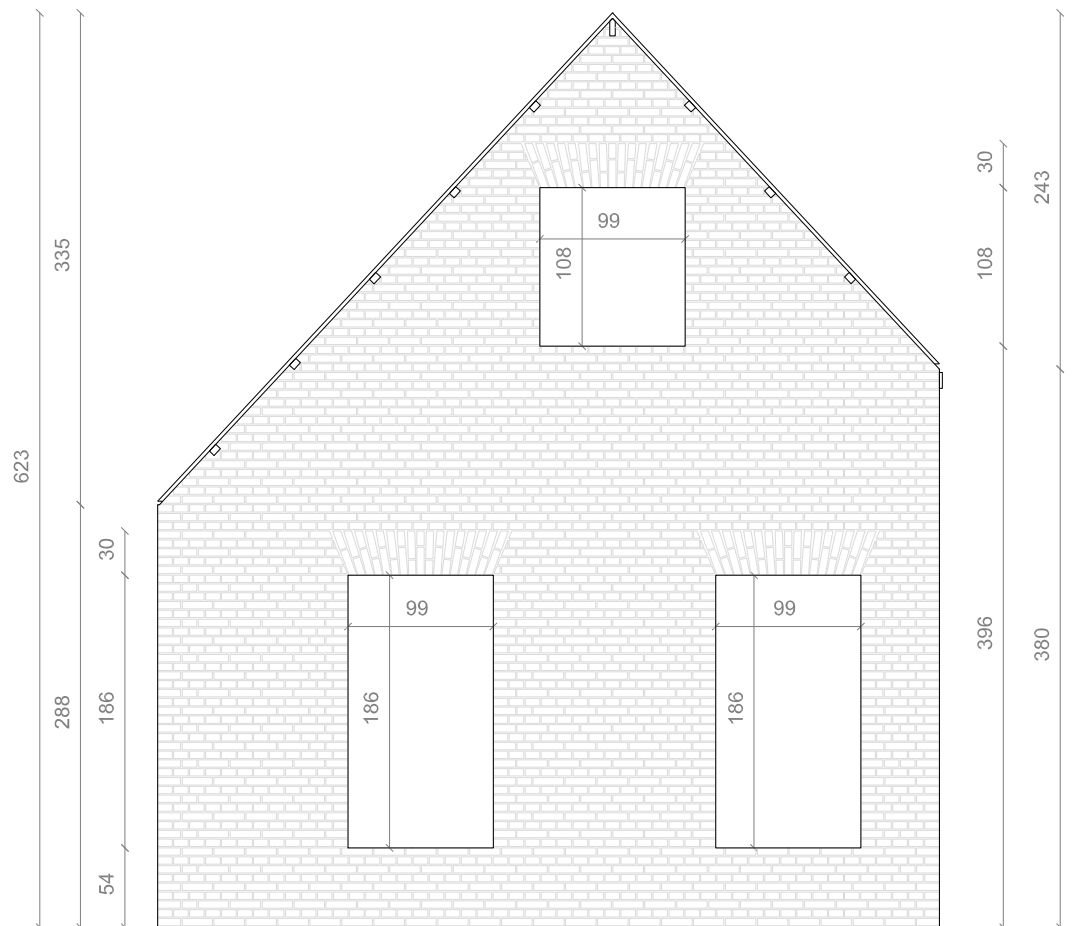


Figure 158. Elevation view of the test-house - South side (in cm).

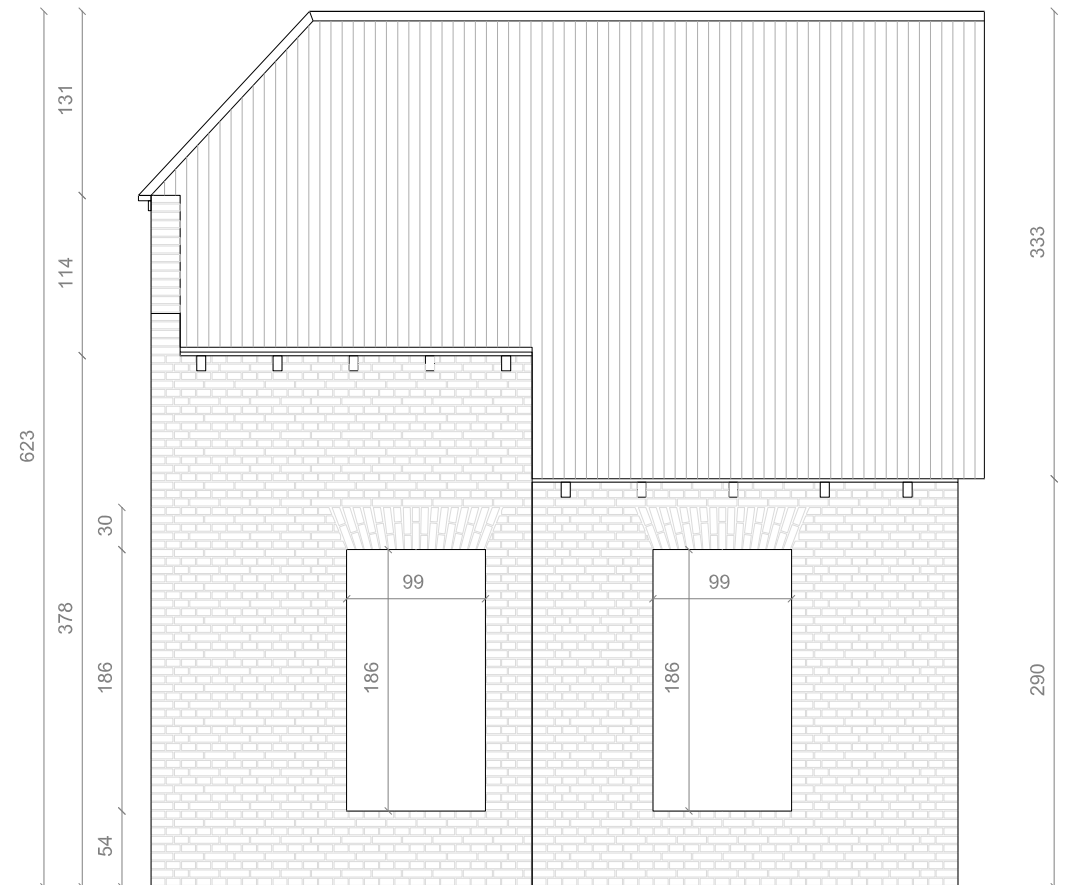


Figure 159. Elevation view of the test-house - West side (in cm).

Moreover, a rigid steel frame has been built, transported inside the EUCENTRE laboratory and fixed on the shaking table in order to serve two main goals: firstly to act as a protective support for the building specimen in the event of a potential partial or global collapse that may occur during the dynamic tests, a scenario that would create significant damages to the testing apparatus; secondly to establish reference points in order to directly measure the relative displacement of some portions of the house with respect to the shaking table surface, to which the structure is firmly bolted. The steel structure columns pass through the floor diaphragm. Four holes in the timber floorboards allow the independent deformation of the specimen with respect to the rigid steel frame.

The following pictures show the rigid steel structure of the first floor and of the roof during the construction phases of the specimen.



Figure 160. View of the steel structure of the first floor during the construction works.



Figure 161. View of the steel structure of the roof during the construction works.

5.1.1 Construction details

In the following sections, the details involved in the construction of the house are described in extent. These details concern the floor and its connections with the perimeter walls, the geometry and the placement of the lintels and the construction of the roof.

Characteristics of the floor diaphragm and detailing of the connections between the floor and the walls

The floor system is made up of 200 mm x 24 mm straight-edge timber (spruce) floorboards nailed perpendicular to ten 80 mm x 180 mm timber joists, that span continuously between the east and west URM walls. Floor joists are spaced at on average 600 mm centre to centre, combining to an overall width of 5.35 m. Floor joists

spacing has been adapted to the need imposed by the laboratory constraints and consequently made variable near the perimeter walls in order to provide a gap of 100 mm between the 350 mm x 350 mm steel plates (laying at the first floor height), welded on top of the columns of the internal steel structure (see Figure 160), and each of the two adjacent joists, in order to allow the independent deformation of the floor diaphragm with respect to the steel frame during the shaking. The position of the four holes in the floorboards, through which the columns of the steel frame go, is shown in the following Figures.

Figure 162 shows a 3D view of the first floor, while Figure 163 a plan view of the floor framing with indication of the floor joists spacing, refer to Figure 182 for a table with dimensions.

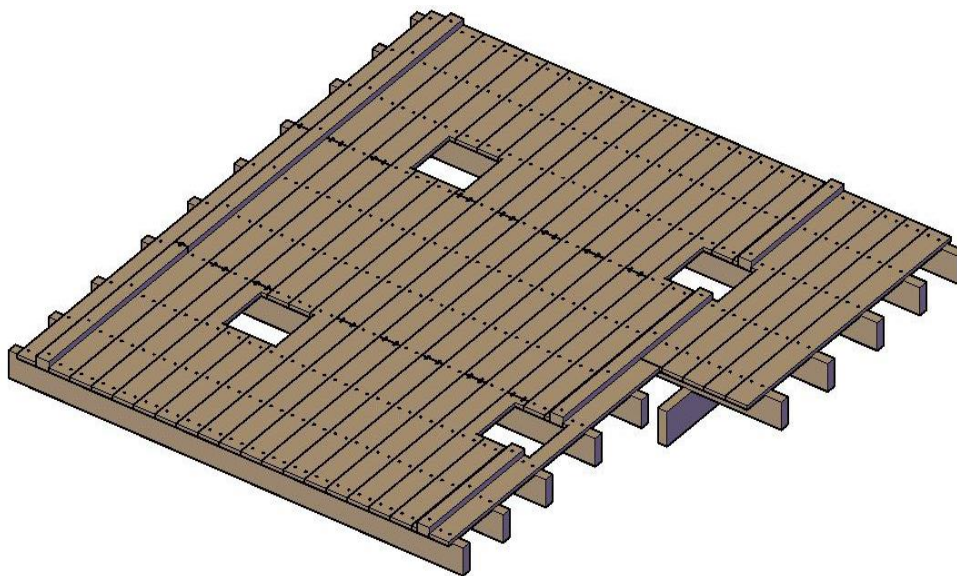


Figure 162. 3D view of the first floor.

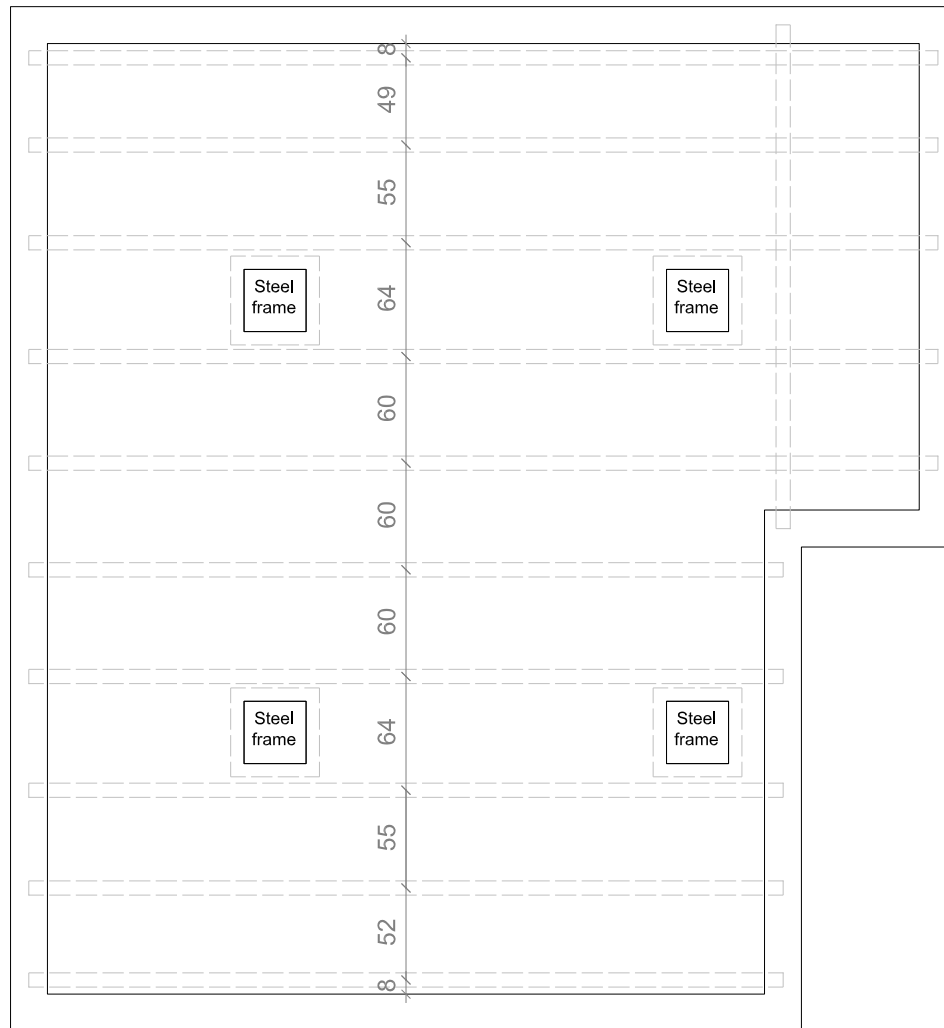


Figure 163. Plan view of the floor framing (dimensions in cm).



Figure 164. Floor joists placement (left), floorboards placement (right).

A 75 mm x 225 mm girder, placed as a support for the floor joists, spans between the south and the north re-entrant walls. This beam is inserted into the masonry thickness and only in the internal 100 mm thick masonry wythe. No connection between the floor girder and joists has been suggested.



Figure 165. Detail of the floor girder-to-south wall connection (left) and positioning of the floor girder (right).

The walls were built up to a height of 270 cm (brick course number 45), afterwards the joists were laid on the internal 10 cm thick wythe of the perimeter walls. Once transported to the EUCENTRE laboratory, the timber floor girder and joists were cut at each end with an inclination of 80°, as in the common Dutch construction practice. Next Figure shows a detail of the floor girder-to-masonry walls connection.

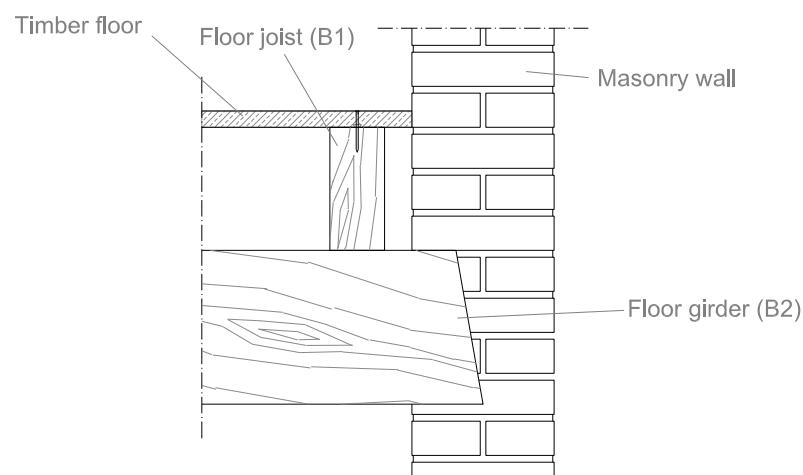


Figure 166. Detail of the floor girder-to-masonry wall connection.

35 square-edge straight floorboards of various lengths were nailed to the joists with two standard nails at each joist intersection. The floorboards layout and their plan dimensions are shown in Figure 167.

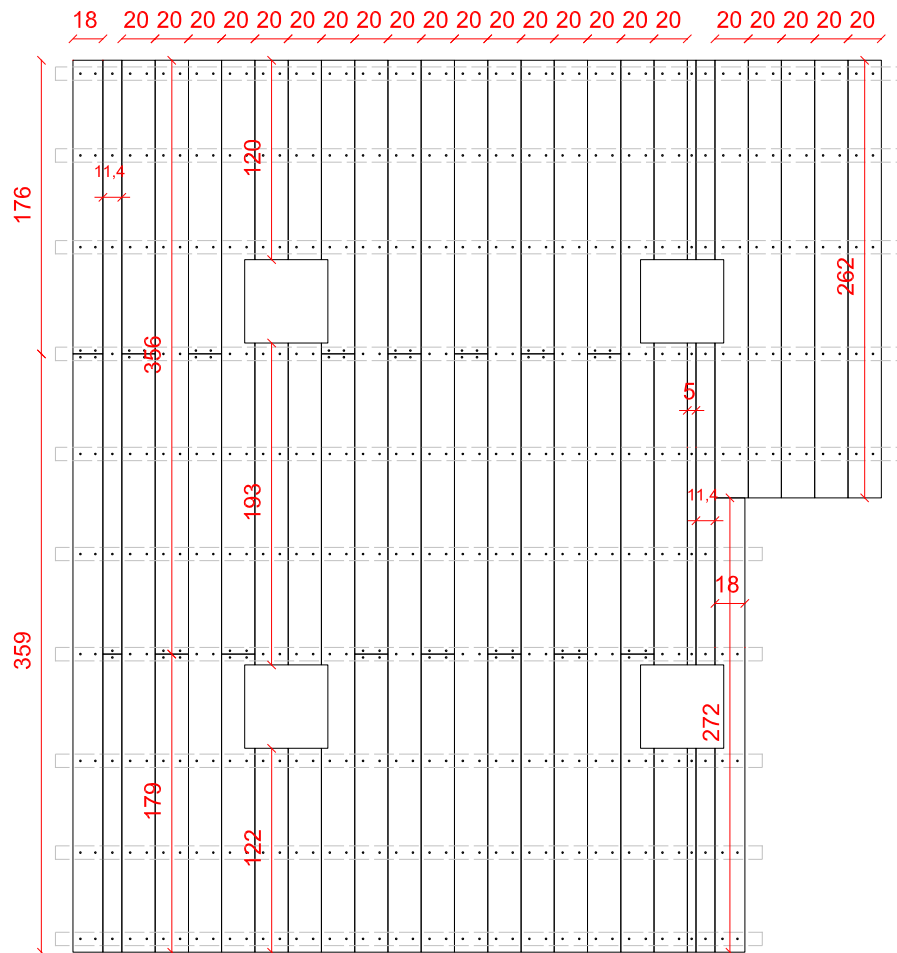


Figure 167. Floorboards layout (dimensions in cm).

The connections between the floor diaphragm and the walls of the Eastern and Western side consist of a series of 14 mm diameter L-shaped steel anchors (5 on each side) screwed laterally to the wood joists and embedded inside the masonry thickness (in the vertical mortar joint). This system is shown in detail in the following Figures. Figure 170 reports the L-shaped anchors dimensions. Figure 175 shows X1 anchors positions.



Figure 168. East wall-to-floor connections.

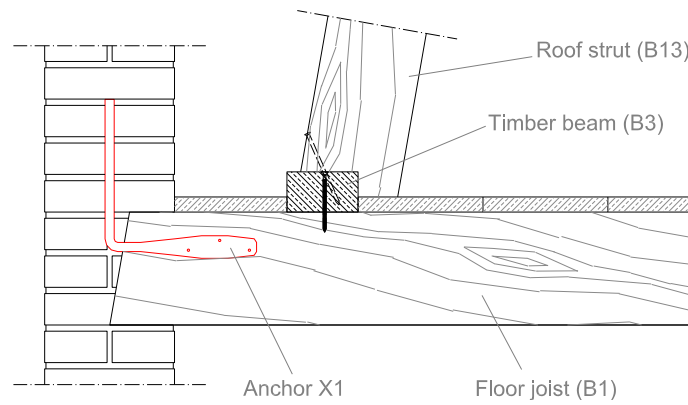


Figure 169. Detail of east wall-to-floor connection.

In particular, Figure 170 and Figure 171 show how the anchors, previously described, connect floor joists to the top of the west wall, that does not extend over the first floor level.

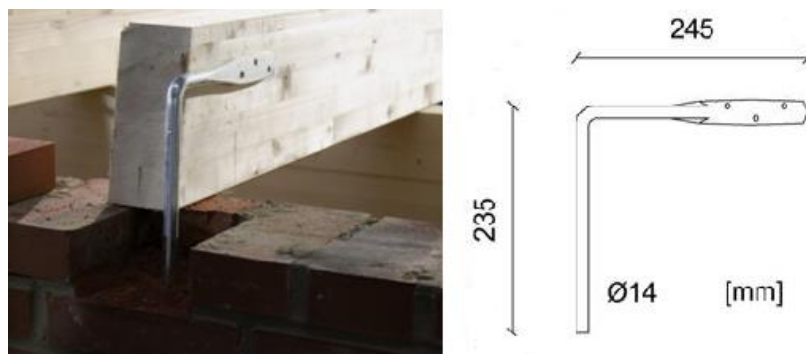


Figure 170. West wall-to-floor connection and dimensions of the anchor X1

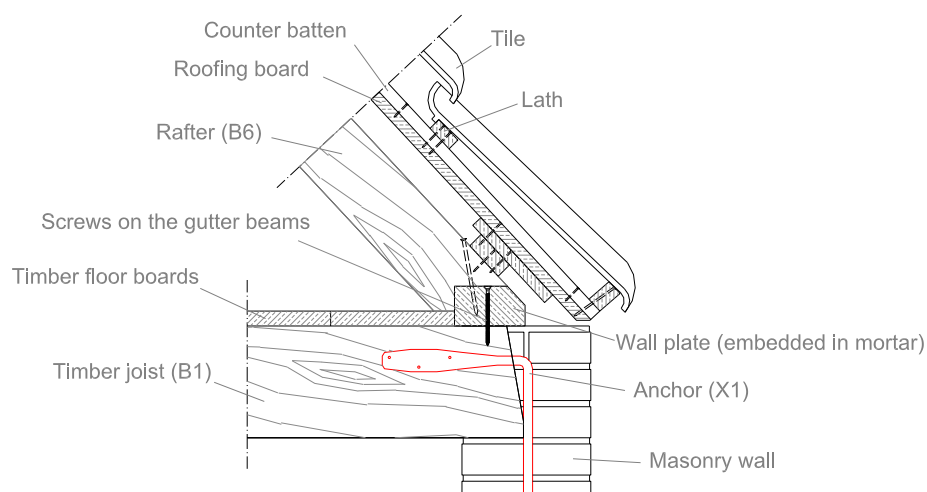


Figure 171. Detail of the west wall-to-floor connection.

Regarding the connections between the floor diaphragm and the walls of the northern and southern side, three 4 mm thick steel elements (that has a total length of 1.2 m and a width of 28 mm, tensile strenght of 47 kN) each side provide a restraint system against the occurrence of wall overturning mechanisms.

With reference to Figure 175, which shows this type of connections positions, anchor Y1 is S-shaped and runs below the timber floorboards. In accordance with the common Dutch construction practice, it was permanently active, since it usually represents the only floor-to-wall connection element in the direction perpendicular to the gables. It is embedded inside the masonry thickness and ties together the first two floor joists by means of screws. The two lateral anchors (Y2), instead, are U-shaped and run below the first two floor joists; they are embedded into the masonry on one side and were be left inactive during the test; they could be connected to the joists in case a wall overturning mechanism occurs, that may damage laboratory equipment.

The next figures show in detail the two different types of connections previously described.

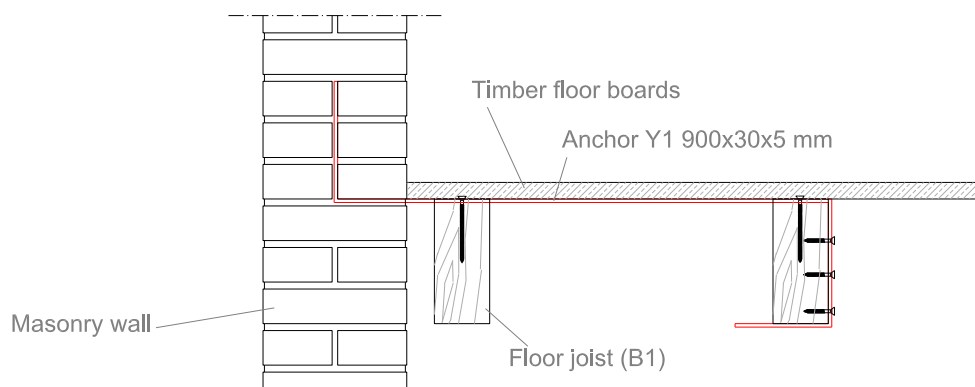


Figure 172. Detail of the floor-to-wall connection in the north-south direction - anchor Y1.

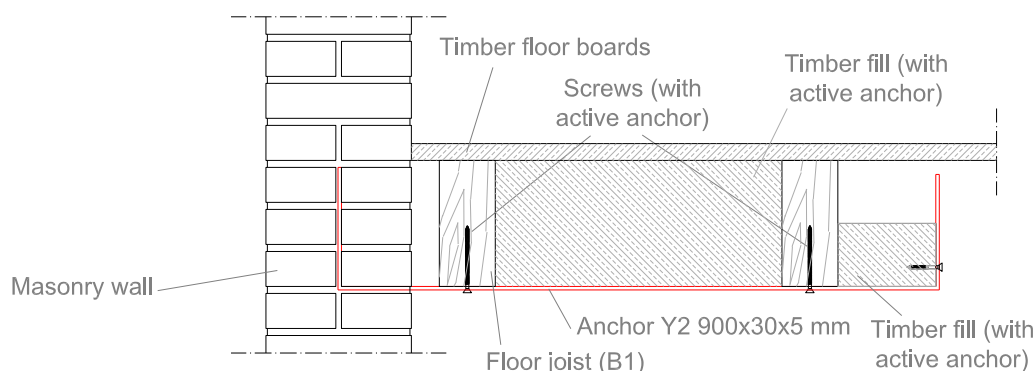


Figure 173. Detail of the floor-to-wall connection in the north-south direction - anchor Y2, when active.



Figure 174. Floor-to-wall connections in the north-south direction - anchor Y1 (left), 3D view of the anchor (right).

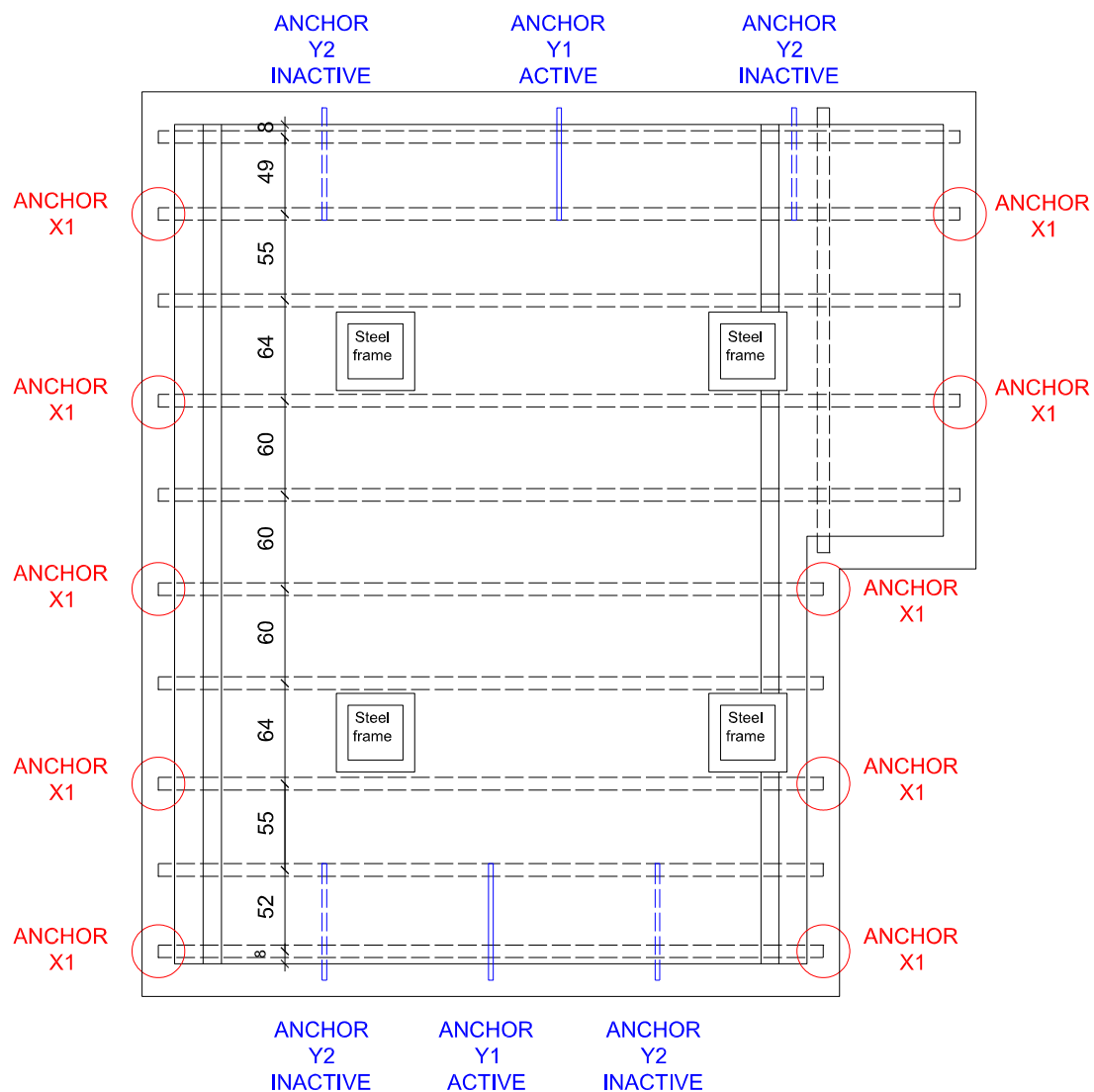


Figure 175. Floor-to-walls connections layout with indication of anchors that were left active or inactive during the test.

Detail of the lintels

The lintels in this type of houses are generally supported by a timber beam that is part of the timber window frame (see Figure 176).



Figure 176. Lintel as in the detached houses of the Groningen region.

Due to laboratory needs, the full window frame has not been included into the test specimen, but only the upper timber element has been integrated.

The test-house has in total 12 openings, 9 of which are located on the ground floor, while the others on the gables. A combined decorative and structural lintel is placed over each opening; this element is composed by a 100 mm x 50 mm timber beam, located on the internal 100 mm thick masonry wythe and a 300 mm high lowered brick tile vault. This masonry flat arch comprises of vertically placed cut bricks with stretchers facing outwards. The wood lintel is 200 mm longer than the window width and anchored into the masonry for 100 mm (half a brick) each side. Next figures show the system described.



Figure 177. Test specimen lintel (with timber temporary support).



Figure 178. Detail of the 100 mm x 50 mm timber beam.

Lintels have different geometrical properties, based on the openings width. Next figure shows the dimensions of the three different types of lintels built.

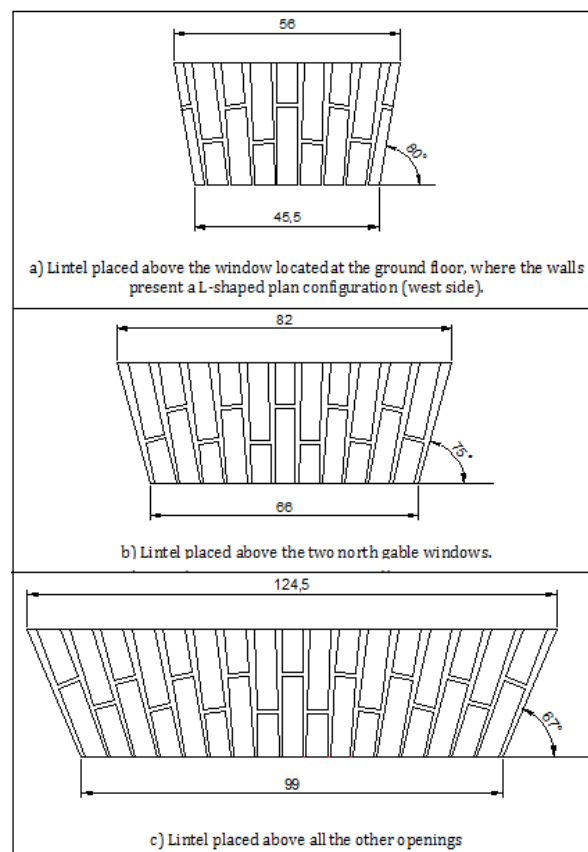


Figure 179. Details and dimensions of the test-house lintels (in cm).

Detail of the roof

An accurate analysis on detached houses in Groningen area, focusing on typical external roof shapes, types of roof internal structure, dimensions and additional information of actual Dutch building practice, has preceded the design process of the test-house. Moreover, after the main roof structure design was selected, the details of the connections have been gathered in a dedicated investigation (Arup 2016c). The study has reported that the majority (about 70%) of the detached houses in Groningen area have a pitched roof and, as a consequence, a triangular form of the façades. It is also to be noted that several combinations, in terms of external roof shape, has been encountered. In particular, from the observations on pitched roofs, it has been concluded that nearly half of detached houses pitched roofs is a combination of roof-end shapes.

Since the prototype building has the main goal to incorporate different common features of the building stock, in order to investigate on differences in the behavior of the building related to the variation in details, the roof external shape has been selected to be a combination of two different roof-end shapes: a one-end gable roof and a one-end half-hipped roof. As such, one façade has the detailing of a half-hipped rural building, while the other façade presented a solid gable wall. This was mainly decided in order to characterize the seismic response of different types of gables, which are expected to be the most vulnerable elements of the test-house.

The prototype building roof internal structure is made up of four timber roof trusses, that occur at irregular intervals (roof trusses exact spacing is reported in Figure 183) and are linked by longitudinal purlins. Figure 180 shows a view of the roof internal structure, during the construction works, while Figure 181 reports a scheme of the two types of roof trusses built with their dimensions.



Figure 180. Test-house roof framing.

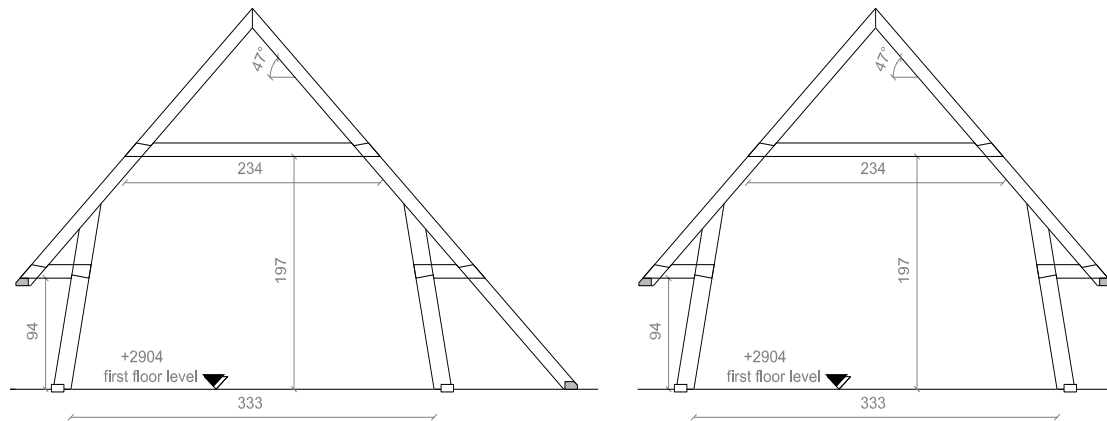


Figure 181. Roof truss spanning between the east and the protruding west walls (left), roof truss spanning between the east and the re-entrant west walls (right) (in cm).

The roof truss system is composed of two 64 mm x 114 mm rafters, nailed together at the top and to a 38 mm x 120 mm ridge beam. At the top end of the rafters, a small recess is made into which the ridge beam is placed. This beam is connected with diagonal nails to the rafters. Considering that a single ridge beam is too short to span across the masonry perimeter walls, it is made up of two timber beams, connected with a tongue and groove joint and nails. The detail of the rafters-to-ridge beam connection is shown in Figure 184 and in detail in Figure 198.

Figure 182 shows a 3D view of all timber beams constituting both the roof and the floor internal structure, followed by a list of each structural element dimensions. A plan view of the roof, instead, is shown in Figure 183 with indication of roof trusses relative spacing.

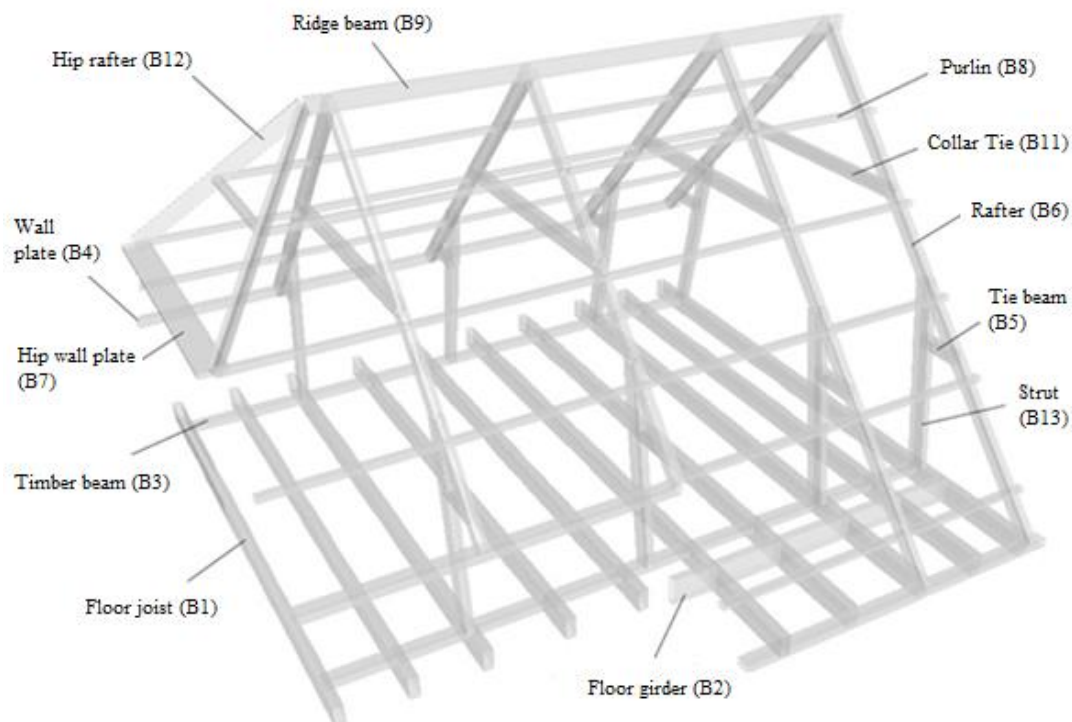


Figure 182. 3D view of the specimen timber internal structure.

TIMBER BEAMS SECTIONS:

- | | |
|-----------------------------------|---------------------------------|
| – B1 = floor joist 80 x 180 mm | – B8 = purlin 44 x 64 mm |
| – B2 = floor girder 75 x 225 mm | – B9 = ridge beam 38 x 120 mm |
| – B3 = timber beam 64 x 114 mm | – B10 = gutter beam 64 x 108 mm |
| – B4 = wall plate 64 x 114 mm | – B11 = collar tie 64 x 114 mm |
| – B5 = tie-beam 64 x 114 mm | – B12 = hip rafter 38 x 120 mm |
| – B6 = rafter 64 x 114 mm | – B13 = strut 64 x 160 mm |
| – B7 = hip wall plate 40 x 200 mm | |

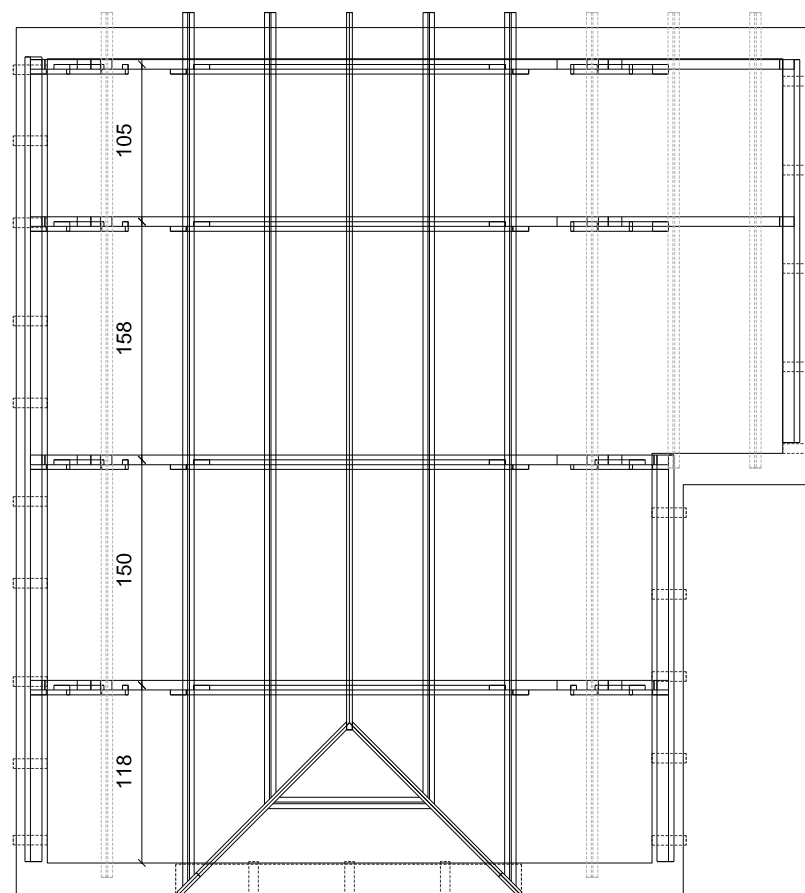


Figure 183. Plan view of the roof framing and roof trusses spacing (in cm).



Figure 184. Detail of the recess, on top end of the rafters, into which the ridge beam is placed (left) and of the rafters-to-ridge beam connection (right).

The two rafters are also connected by means of a 64 mm x 108 mm horizontal collar tie (B11), placed at 1.97 m from the floor extrados. In addition to these elements, a 64 mm x 160 mm strut beam (B13), in compression, on each side supports the rafter. The strut is connected to the floor by means of a 64 mm x 114 mm timber beam (to which it is nailed), laying between the diaphragm floorboards and nailed to the floor joists (see Figure 185). A timber tie-beam (B5) between the rafter and the strut guarantees transverse connection between the two elements. The member is mainly subjected to tensile forces and presents some cut-outs. The cut-out transfers the forces if the member comes in compression, while the taped shape guarantees transfer of tension between elements. A cut-out at the strut/tie-beam connection is provided to increase the connection strength. Some pictures of these elements and their connections are shown below, followed by detail drawings.



Figure 185. Detail of the rafters, struts and tie beams of the east (left) and of the west side (right).

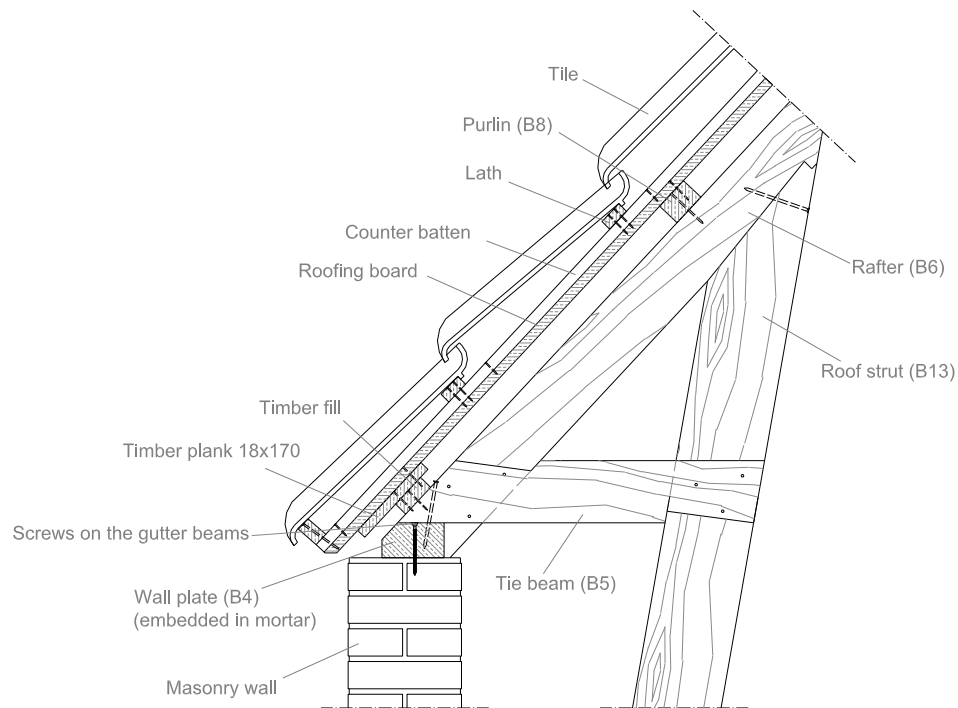


Figure 186. Rafter, tie-beam, strut and detail of the rafter-to-wall connection.

A common type connection between strut, rafter and tie-beam is the interlock connection. The tensile members in the truss are connected by 3 nails.



Figure 187. Typical interlocking connection.

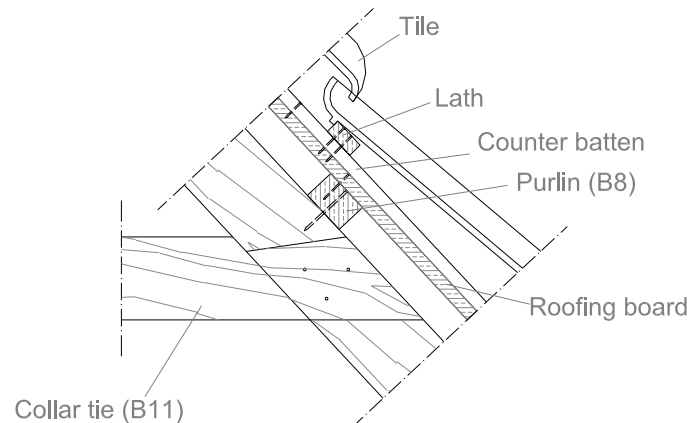


Figure 188. Detail of the collar tie-to-rafter interlocking connection.

The typical rafter-to-wall connection is shown in Figure 189 and depicted in detail in Figure 186. A wall plate (B4), embedded in mortar, is placed on top of longitudinal walls. The rafters are nailed to this plate and have a heel connection, designed for compression-only. The wall plate is screwed to a series of timber beams (64 mm x 108 mm, 10 on each side), which are recessed into the masonry and located laterally to the timber joists in order to support the roof gutter in actual structures (not present in this specimen) (gutter beams are shown in Figure 189 and sketched in Figure 195).



Figure 189. Detail of the rafter-to-west wall connection at the floor level (left), and of the rafter-to-east wall connection (right) by mean of a wall plate nailed to gutter beams.

At the south roof-end purlins (B8), connected to rafters with two nails, go through the masonry thickness and protrude of 10 cm from the gable (see Figure 190). Two planks were added on the exterior of the masonry, forming a L-shaped configuration, as shown in Figure 191.



Figure 190. South roof-end purlins.

Regarding the roof-to-north wall connection, the elements that are directly connected to the wall are the hip wall plate (B7) and, near the inclined portions of the wall, the purlins (B8). The hip wall plate is nailed to three gutter beams, which are located on top of the north gable and are embedded into the masonry thickness. The wall plate was not embedded in mortar.

The connection between the hip wall plate, the hip rafter and the adjacent purlin is provided by nails. A detail of this system is represented in Figure 192.



Figure 191. Plank on the exterior of the south gable.



Figure 192. Connection between the hip wall plate, the hip rafter and the adjacent purlin.

Above the roof internal structure, 18 mm x 200 mm timber roofing boards are placed perpendicularly to the purlins (see Figure 193). At the top of the east and west walls, these boards are nailed into a timber plank, that is in turn connected to the rafters by mean of two standard nails and a timber fill. The detail of this connection is shown in Figure 186, Figure 194 and Figure 195.

The roofing boards are connected to one another via tongue and groove; in particular, two nails guarantee the connection to the purlins at each intersection.

Regarding the north roof-end, the roofing boards are nailed into a 40 x 210 mm timber plank (shown in Figure 196). The planks are connected to the hip rafters (B12) by means of nails.

A grid made of laths and counter battens is nailed above the roofing boards in order to act as a support for the clay tiles. This system is shown in Figure 188 and Figure 197. Tiles are nailed to the laths. In Figure 198 a detail of the roof ridge beam is depicted.



Figure 193. Roofing boards placement.

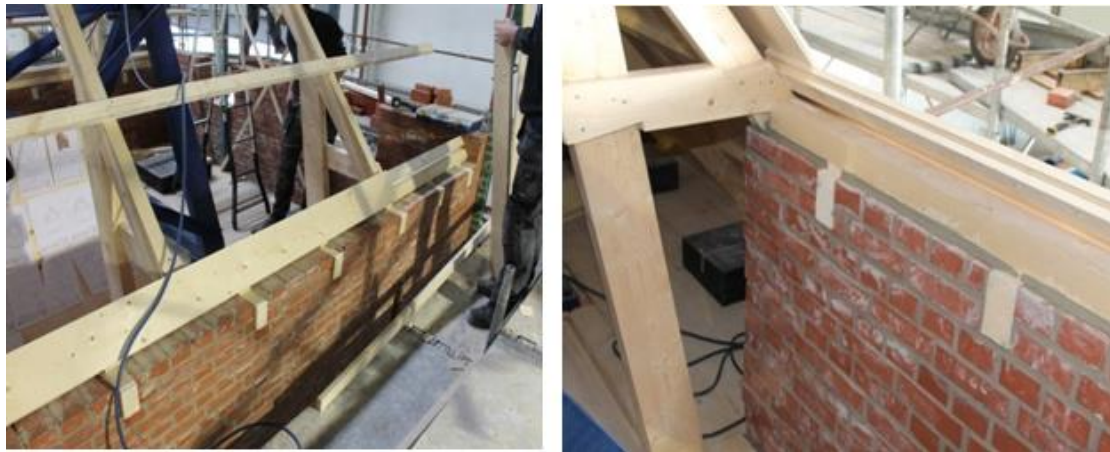


Figure 194. Timber plank at the top of the east wall (left) and west re-entrant wall (right).

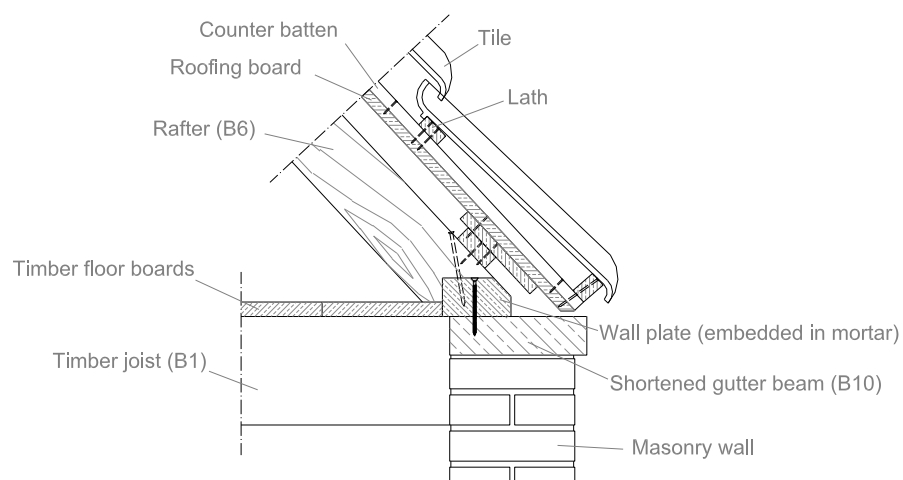


Figure 195. Detail of the connection between the rafter and top of the west wall, that does not extend over the floor level.



Figure 196. Timber plank to which roofing boards are nailed (north roof-end).

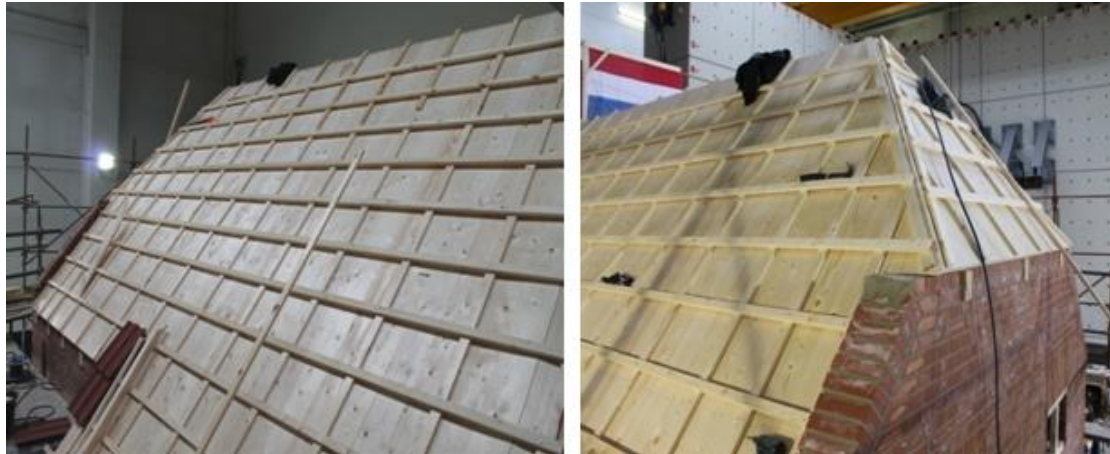


Figure 197. Laths and counter battens grid.

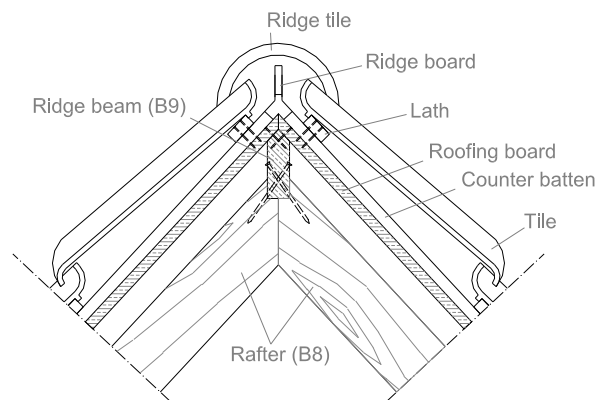


Figure 198. Detail of the rafters-to-ridge beam connection.

Regarding the roof-to-north wall connection, four threaded bars M12 are applied on the hip wall plate through long holes and left inactive; they would have been activated in case sliding of the wall plate or overturning of the gable. Next Figures show a detail of this connection system.



Figure 199. Threaded bars M12

During the construction of the specimen, the same L-shaped anchors, used to connect the floor diaphragm to the east and west walls, have been screwed laterally to the roof purlins and embedded into the masonry thickness of both gables (inside the mortar joint). Before the test, these elements were disabled removing the screws, and reactivated during the test if a premature out-of-plane failure would have been triggered. The following Figure shows these types of anchors and their position.



Figure 200. Details of one of the L-shaped anchor of the south gable (left) and of the south-end ridge beam (right) during the construction works.



Figure 201. Anchors of the north gable (left) and of the south gable (right).

Figure 202 shows a scheme specifying the position of both the types of gable-to-roof connections previously described and which of them is left inactive or activated before the dynamic test.

In addition, a restraint system made of steel rods anchoring the roof truss structures to the internal steel frame would be adopted in order to avoid a collapse mechanism of the roof, which could result in a severe damage to test equipment. This chains system allows the horizontal out-of-plane displacement of gables till significant displacement (≈ 15 cm) levels without any force resistance contribution (see Figure 203, Figure 204 and Figure 205).

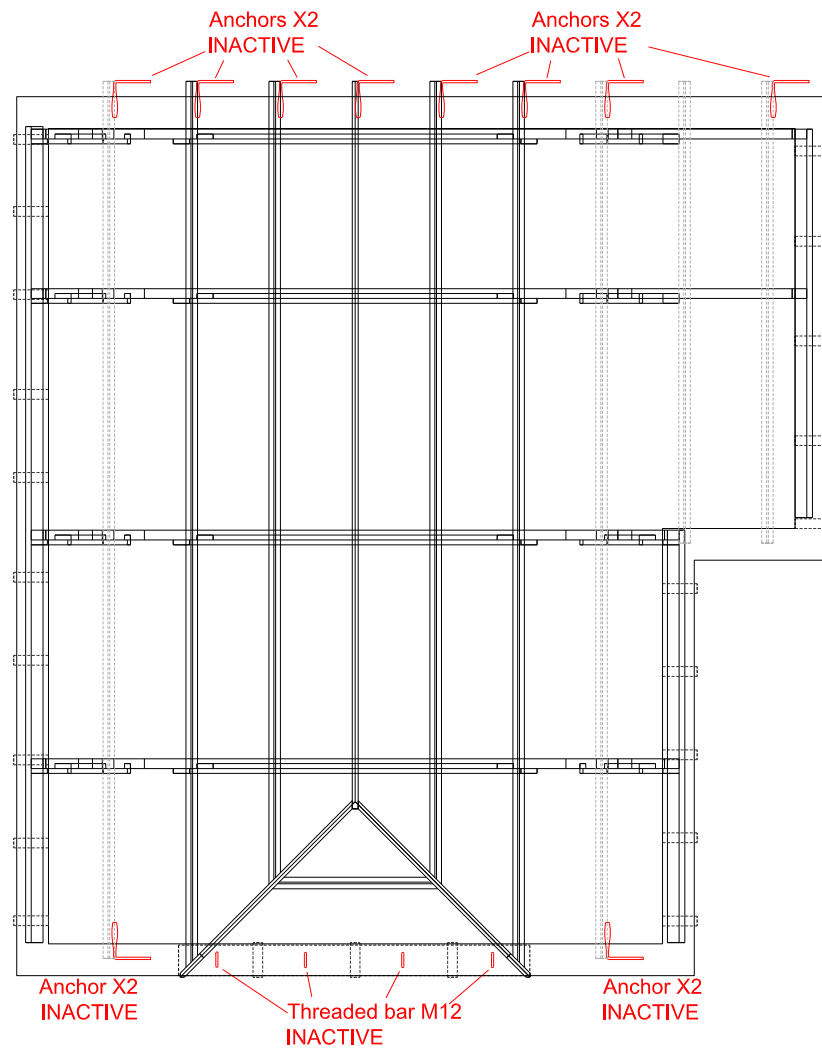


Figure 202. Roof-to-gables connections layout with indication of anchors that were be left active or inactive during the test.



Figure 203. Hip wall plate safety system (left), roof safety system (right).



Figure 204. Rods anchoring the rigid steel structure to the roof truss of the South side.



Figure 205. Rods anchoring the rigid steel structure to the roof truss of the North side.

Mass of the test specimen

On the first floor a total mass of 1.31 t was placed by mean of 8 laminated rubber blocks representing an additional load; six of these blocks are 600 mm x 700 mm and weigh 200 kg each, the others are 350 mm x 400 mm and weigh 55 kg each. Their exact position is shown in Figure 206, followed by a table, that summarizes the masses contributing to the total mass of the test-house (highlighted in red).

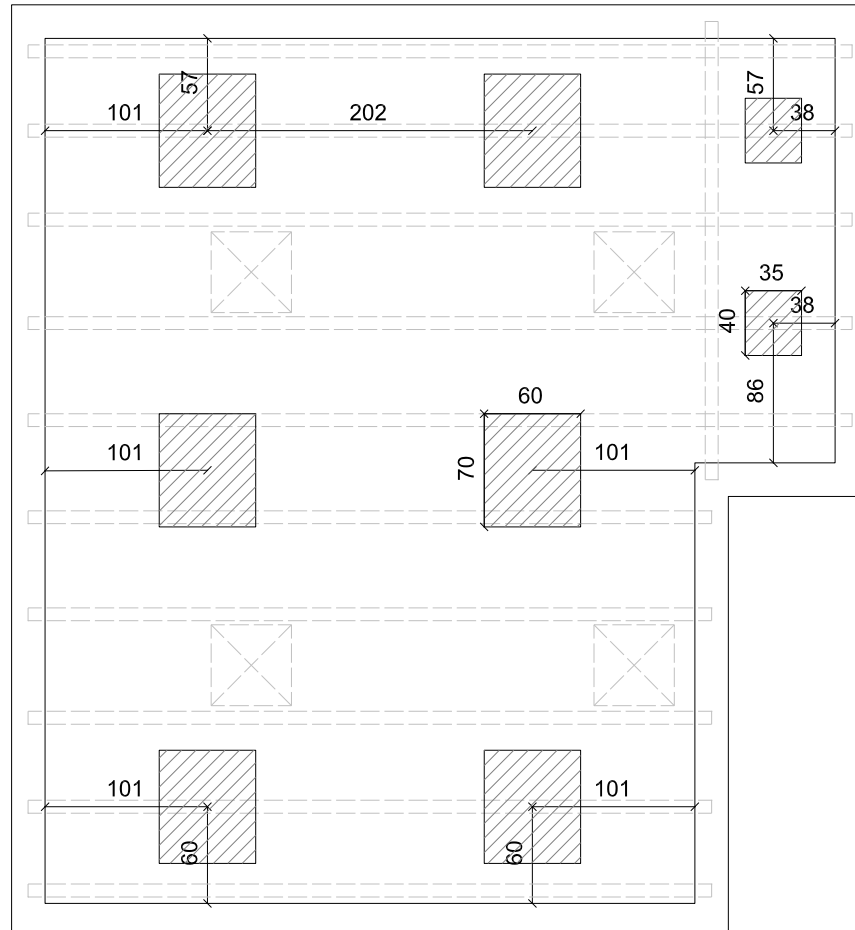


Figure 206. Additional masses position and dimensions (in cm).

Table 27. List of masses constituting the total mass of the specimen.

<i>Masses [t]</i>	
Mass of the masonry	28.96
Mass of the tiles	1.33
Mass of the roof structure	0.54
Total mass of the roof	1.87
Mass of the floor diaphragm	0.47
Floor blocks masses	1.31
Total mass of the floor	1.78
Total mass of the specimen	32.61

Some pictures of the building specimen at the end of the construction works are reported below.



Figure 207. Test-house at the end of the construction works. North-west corner.



Figure 208. Test-house at the end of the construction works. South-east corner.



Figure 209. Test-house at the end of the construction works. North-east corner.



Figure 210. Test-house at the end of the construction works. South-west corner.

5.2 Instrumentation and acquisition system

The sequence of dynamic tests consisted in subjecting the base of the specimen to seismic excitations in the north-south direction.

In order to monitor the structural response under different levels of input motion, several instruments were placed in the building. The location and typology of the instrumentation was determined based on the identification of the position of the critical zones and on the physical quantity to be recorded. The entire set-up instrumentation consisted in wire potentiometers, LVDTs (measuring the relative displacements), accelerometers and a 3D optical acquisition system.

A high number of passive target markers were placed on the external surface of the north, south and west walls in order to allow high-definition cameras to monitor the in-plane and out-of-plane displacements of defined points of the walls. Detailed info about the instrumentation and instructions regarding the use of the recordings are reported in Appendix-A.

5.2.1 Identification of the position of the displacement transducers

The instrumentation consisted of a set of wire potentiometers and LVDTs, which guaranteed a measurement of the displacements of some portions of the building. The wire potentiometers have been installed to monitor the in-plane and out-of-plane displacement of the walls, the in-plane displacements of the floor diaphragm and the displacements of the roof. The traditional potentiometer, instead, monitored relative displacements in the X and Y direction.

All wire potentiometers installed on the masonry, on the floor and on the roof are listed in Table 28 and shown in the following Figures.

Table 28. Wire potentiometers: identification and location.

Sensor ID	Location	Measured parameter
1	south wall, mid-height	displacement in the X direction
2	north wall, mid-height	displacement in the X direction
3	top of the south gable	displacement in the X direction
4	top of the north gable	displacement in the X direction
5	roof ridge, mid-span	displacement in the X direction
6	floor diaphragm, along the east wall	displacement in the X direction
7	floor diaphragm, along the west wall	displacement in the X direction
8	floor diaphragm, along the south-east/north-west diagonal	displacement along the diagonal
9	floor diaphragm, along the south-west/north-east diagonal	displacement along the diagonal
42	east façade window, between pier and bottom spandrel	displacement in the Z direction
43	east wall, wide pier, 1 st -floor height	displacement in the X direction
44	east wall, wide pier	displacement in the Z direction
45	east wall, wide pier	displacement in the Z direction
46	east wall, wide pier	displacement along the diagonal
47	east wall, wide pier	displacement along the diagonal
48	east wall, wide pier	displacement in the X direction
49	east wall, wide pier	displacement in the Z direction
50	east wall, wide pier	displacement in the Z direction
51	east wall, wide pier	displacement along the diagonal
52	east wall, wide pier	displacement along the diagonal
57	east wall, wide pier	displacement in the X direction

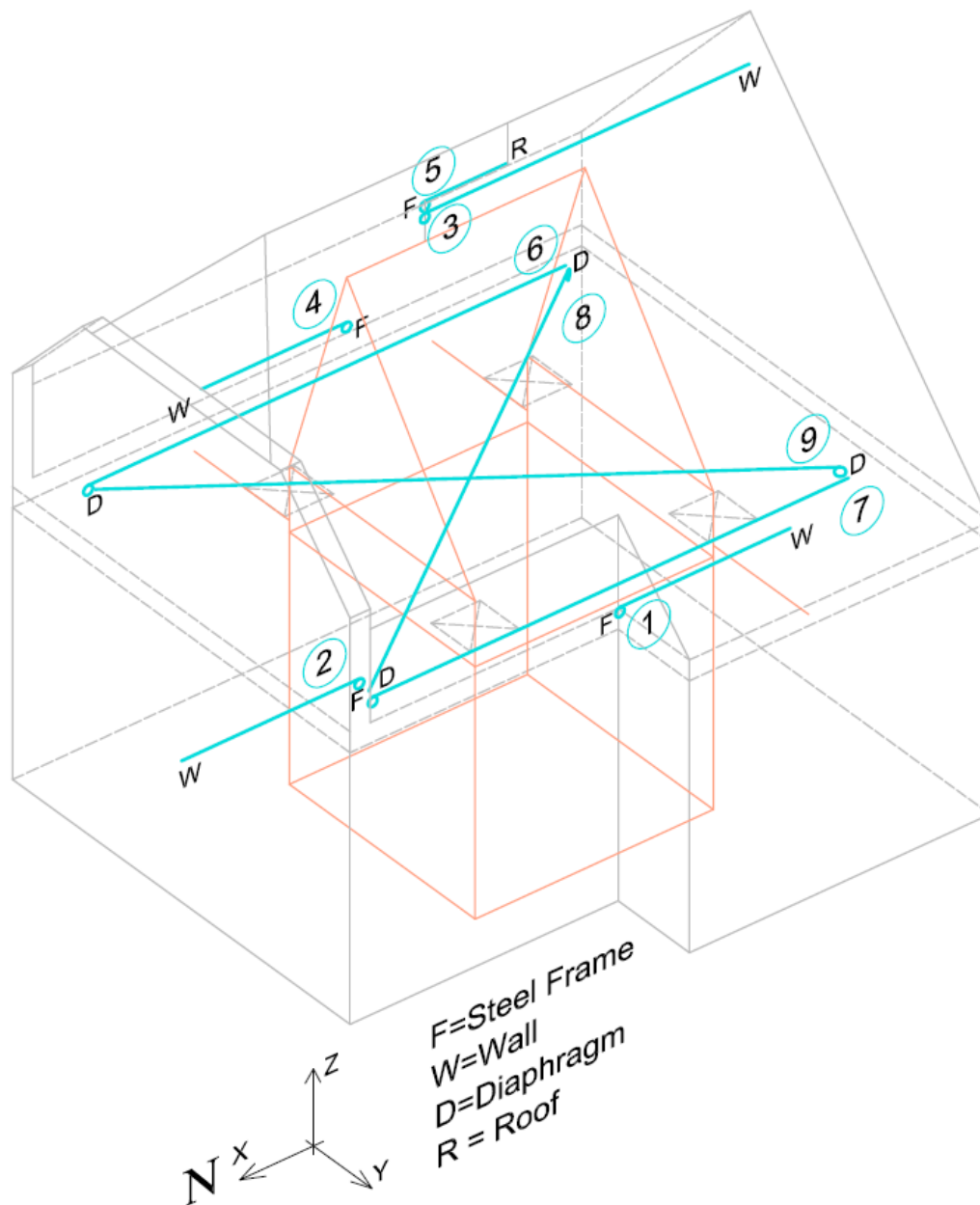


Figure 211. 3D view of all wire potentiometers installed in the building.

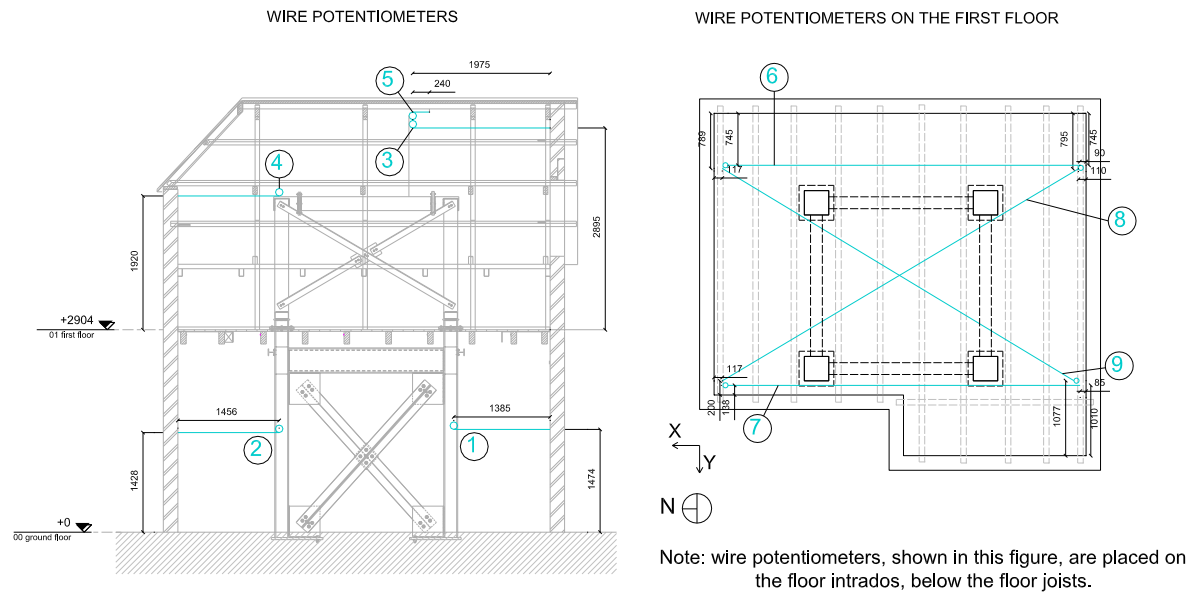


Figure 212. Identification of the position of wire potentiometers installed inside the building (left) and on the floor intrados (right).

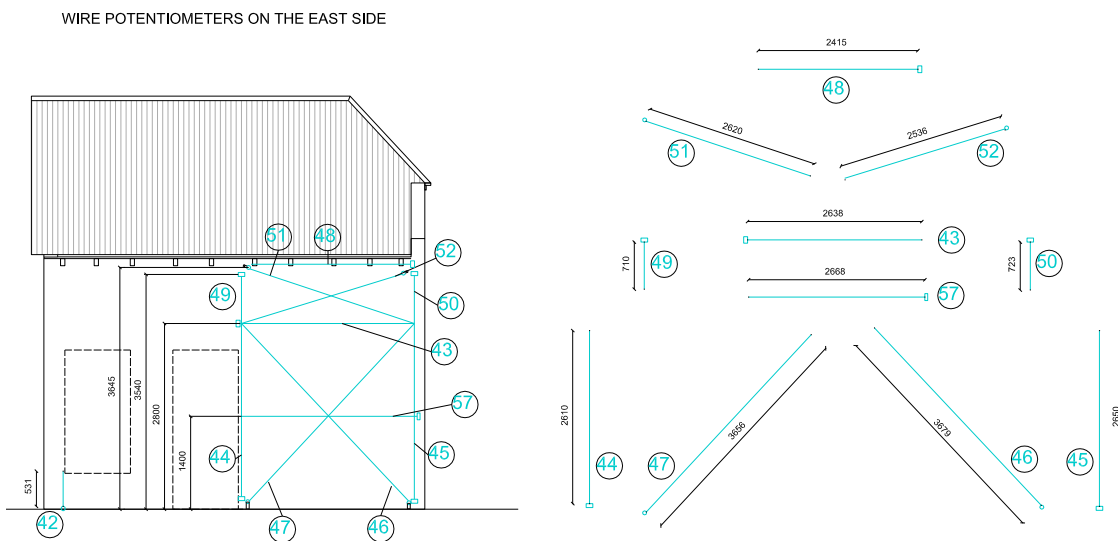


Figure 213. Identification of the position of wire potentiometers installed on the surface of the east walls (left) and indication of their length (right).

All LVDTs installed on the masonry, on the floor and on the roof are listed in Table 29 and shown in the following Figures.

Table 29. LVDTs: identification and location.

Sensor ID	Location	Measured parameter
10	floor diaphragm/steel frame, free end of south-west cantilever beam	displacement in the X direction
11	floor diaphragm/steel frame, free end of south-west cantilever beam	displacement in the Y direction
12	floor diaphragm/steel frame, close to south-west column	displacement in the X direction
13	floor diaphragm/steel frame, south side, aligned with LVDT #18	displacement in the X direction
14	floor diaphragm/steel frame, free end of south-east cantilever beam	displacement in the X direction
15	floor diaphragm/steel frame, free end of south-east cantilever beam	displacement in the Y direction
16	floor diaphragm/steel frame, free end of north-east cantilever beam	displacement in the X direction
17	floor diaphragm/steel frame, free end of north-east cantilever beam	displacement in the Y direction
18	floor diaphragm/steel frame, north side, at mid-span of the north wall	displacement in the X direction
19	floor diaphragm/steel frame, free end of north-west cantilever beam	displacement in the X direction
20	floor diaphragm/steel frame, free end of north-west cantilever beam	displacement in the Y direction
21	floor diaphragm, close to south-west corner/south wall	displacement in the X direction
22	floor diaphragm, at mid-span of the south wall/south wall	displacement in the X direction
23	floor diaphragm, close to south-east corner/south wall	displacement in the X direction
24	floor diaphragm, close to north-east corner/north wall	displacement in the X direction
25	floor diaphragm, at mid-span of the north wall/north wall	displacement in the X direction
26	floor diaphragm, close to north-west corner/north wall	displacement in the X direction
27	top of the east wall/steel frame	displacement in the Y direction
28	top of the east wall/steel frame	displacement in the X direction
29	top of the east wall/steel frame	displacement in the Y direction
30	top of the east wall/steel frame	displacement in the X direction
31	top of the west wall/steel frame	displacement in the Y direction
32	top of the west wall/steel frame	displacement in the X direction
33	top of the south gable/south roof truss	displacement in the X direction
34	north gable/roof purlin (east side)	displacement in the X direction
35	north gable/roof purlin (west side)	displacement in the X direction



36	foundation/east wall	displacement in the X direction
37	east-side foundation/shaking table	displacement in the X direction
38	foundation/west wall	displacement in the X direction
39	west-side foundation/shake-table	displacement in the X direction
40	shake-table/laboratory floor	displacement in the X direction
41	top of the north gable/hip wall plate	displacement in the X direction
53	floor diaphragm, close to west protruding corner/ wall	displacement in the X direction
54	top of east wall/wall plate	displacement in the X direction
55	south gable/roof truss collar tie	displacement in the X direction
56	south gable/roof truss collar tie	displacement in the X direction
58	roof truss/steel frame	displacement in the X direction

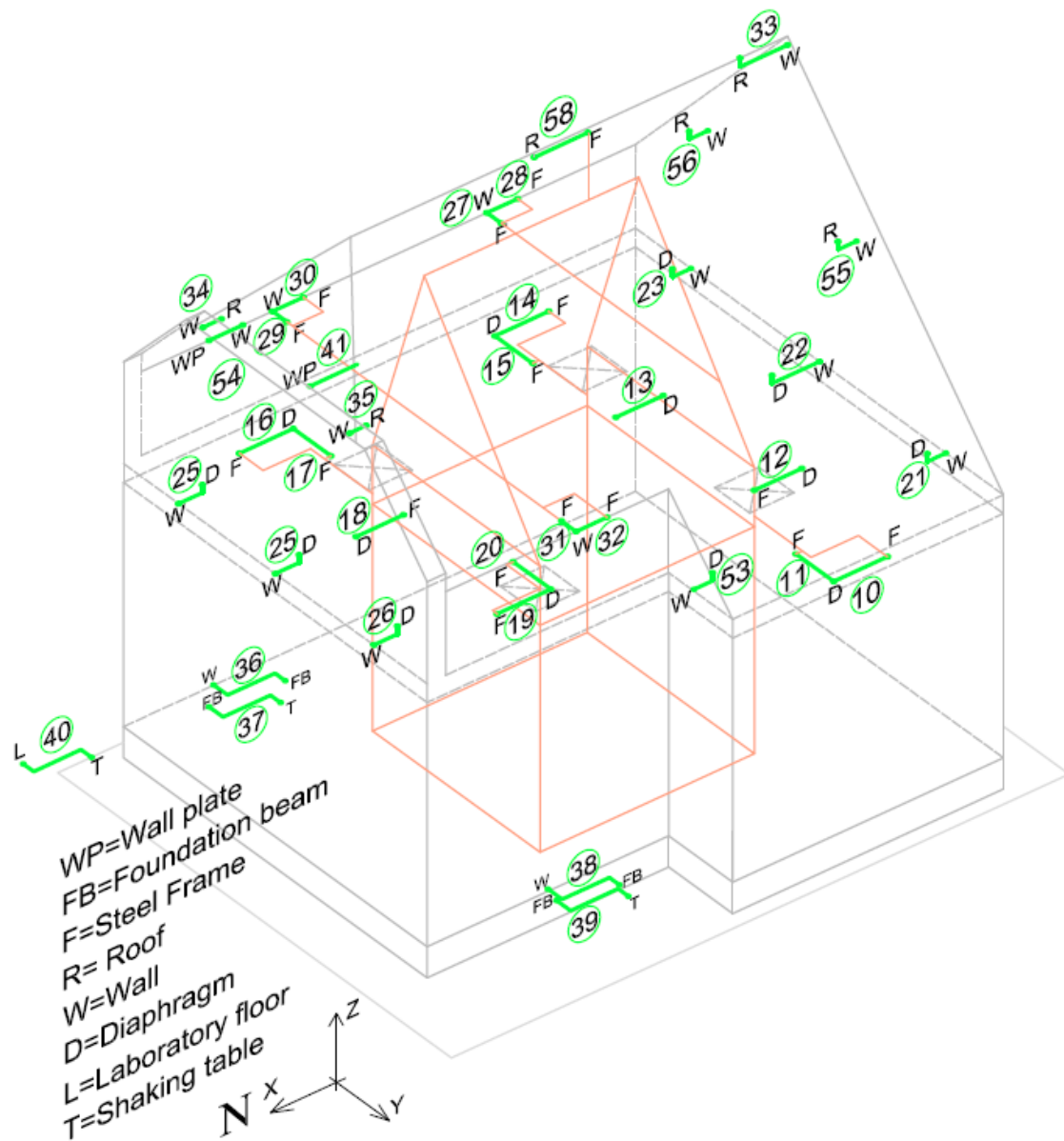


Figure 214. 3D view of all LVDTs installed in the building.

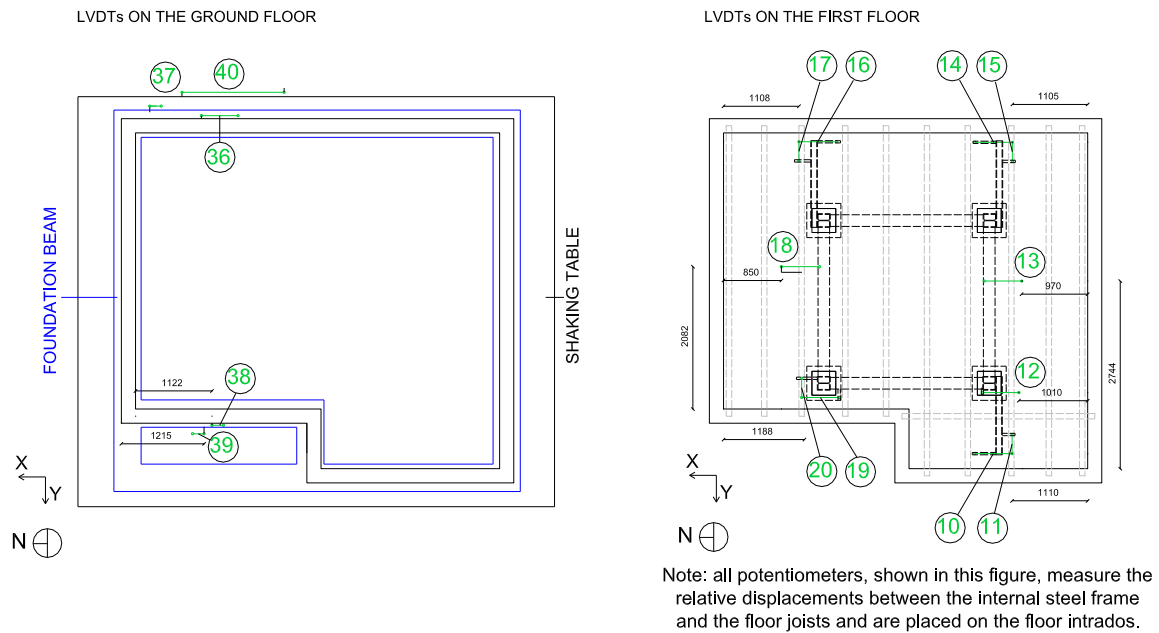


Figure 215. Identification of the position of LVDTs installed at the foundation level (left) and on the floor intrados (right).

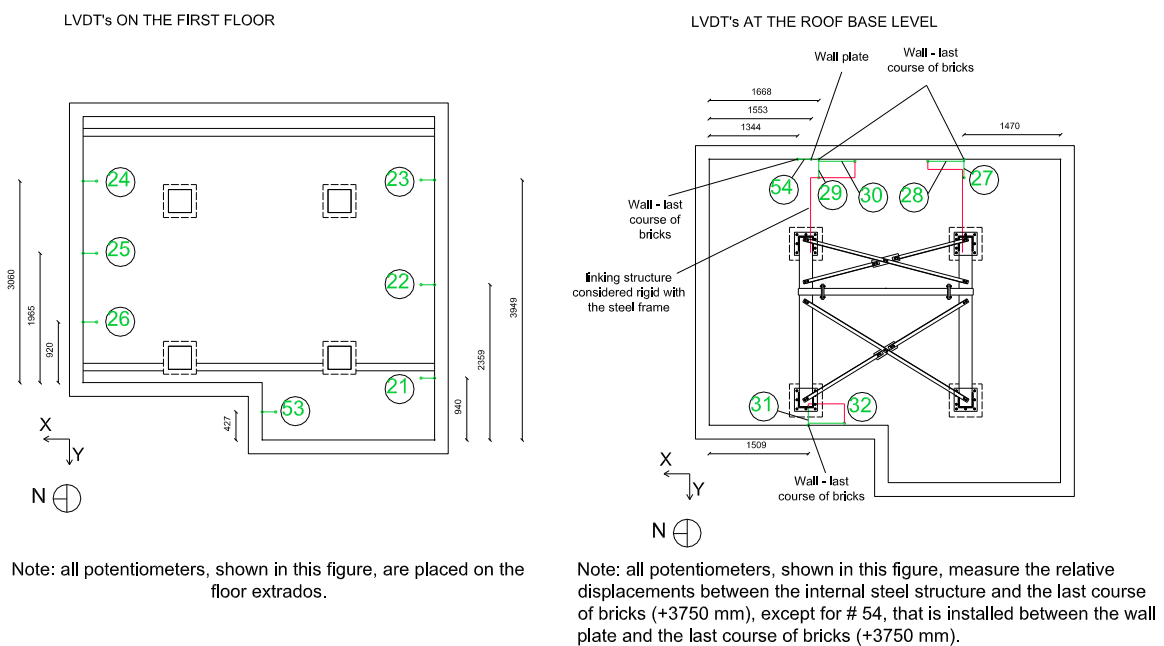


Figure 216. Identification of the position of LVDTs installed on the floor extrados.

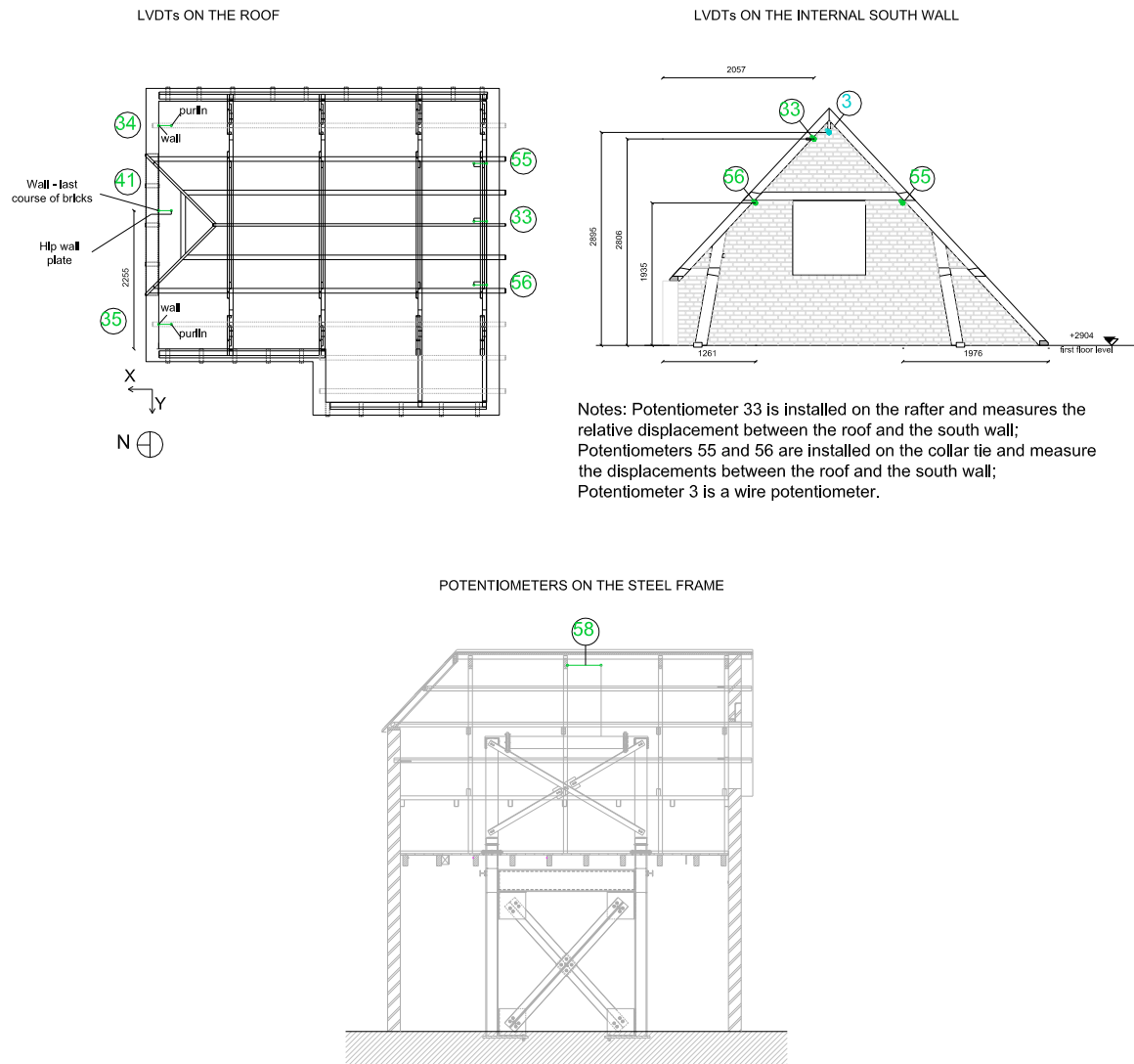


Figure 217. Identification of the position of LVDs installed on the roof structure.

5.2.2 Identification of the position of the accelerometers

The accelerometers guaranteed an accurate measurement of the following quantities:

- Acceleration of the shake table;
- Walls accelerations along their height;
- Acceleration at 1st and roof levels;
- Acceleration of the internal steel structure.

All the acceleration histories are absolute accelerations. Following Table lists the accelerometers installed on the walls, on the floor diaphragm, on the roof and on the rigid steel structure, specifying their position and ID number.

Table 30. Accelerometers: identification and location.

Sensor ID	Location	Measured parameter
1	floor diaphragm, south-west corner	accelerations in the X, Y, and Z directions
2	floor diaphragm, south-east corner	accelerations in the X, Y, and Z directions
3	floor diaphragm, north-east corner	accelerations in the X, Y, and Z directions
4	floor diaphragm, north-west corner	accelerations in the X, Y, and Z directions
5	floor diaphragm, close to reentrant corner	accelerations in the X direction
6	floor diaphragm, close to center of mass	accelerations in the X, Y, and Z directions
7	roof ridge, south end	accelerations in the X, Y, and Z directions
8	roof ridge, north end	accelerations in the X, Y, and Z directions
9	roof ridge, midspan	accelerations in the Z direction
10	south wall, 1 st -floor height, close to south-west corner	accelerations in the X direction
11	south wall, 1 st -floor height, mid-span	accelerations in the X direction
12	south wall, 1 st -floor height, close to south-east corner	accelerations in the X direction
13	north wall, 1 st -floor height, close to north-east corner	accelerations in the X direction
14	north wall, 1 st -floor height, mid-span	accelerations in the X direction
15	north wall, 1 st -floor height, close to north-west corner	accelerations in the X direction
16	floor diaphragm, at mid-span of the south wall	accelerations in the X direction
17	floor diaphragm, at mid-span of the north wall	accelerations in the X direction
18	ground-story south wall, centered on the pier between windows	accelerations in the X direction
19	south gable, close to north-west corner, aligned with accelerometer # 37	accelerations in the X direction
20	south gable, top	accelerations in the X direction
21	ground-story north wall, centered on the pier between windows	accelerations in the X direction
22	north gable, centered between windows	accelerations in the X direction
23	north gable, top	accelerations in the X direction
24	foundation, west side	accelerations in the X direction
25	foundation, east side	accelerations in the X direction
26	north gable, at windows mid-height, close to north-east corner	accelerations in the X direction
27	steel frame, 1 st -floor height, on the north-west column	accelerations in the X direction



28	steel frame, ridge-beam height, north end	accelerations in the X direction
29	east façade, between door and window, 1 st -floor height	accelerations in the X direction
30	east façade, wide pier, 1 st -floor height	accelerations in the X direction
31	west façade, protruding corner, 1 st -floor height	accelerations in the X direction
32	west reentrant wall, 1 st -floor height, close to reentrant corner	accelerations in the X direction
33	roof, south-west corner	accelerations in the X direction
34	roof, south-east corner	accelerations in the X direction
35	roof, north-east corner	accelerations in the X direction
36	roof, north-west corner	accelerations in the X direction
37	south gable, roof base height, close to south-east corner	accelerations in the X direction

Next Figures show the exact position of the accelerometers and the reference system assumed for all the accelerograms.

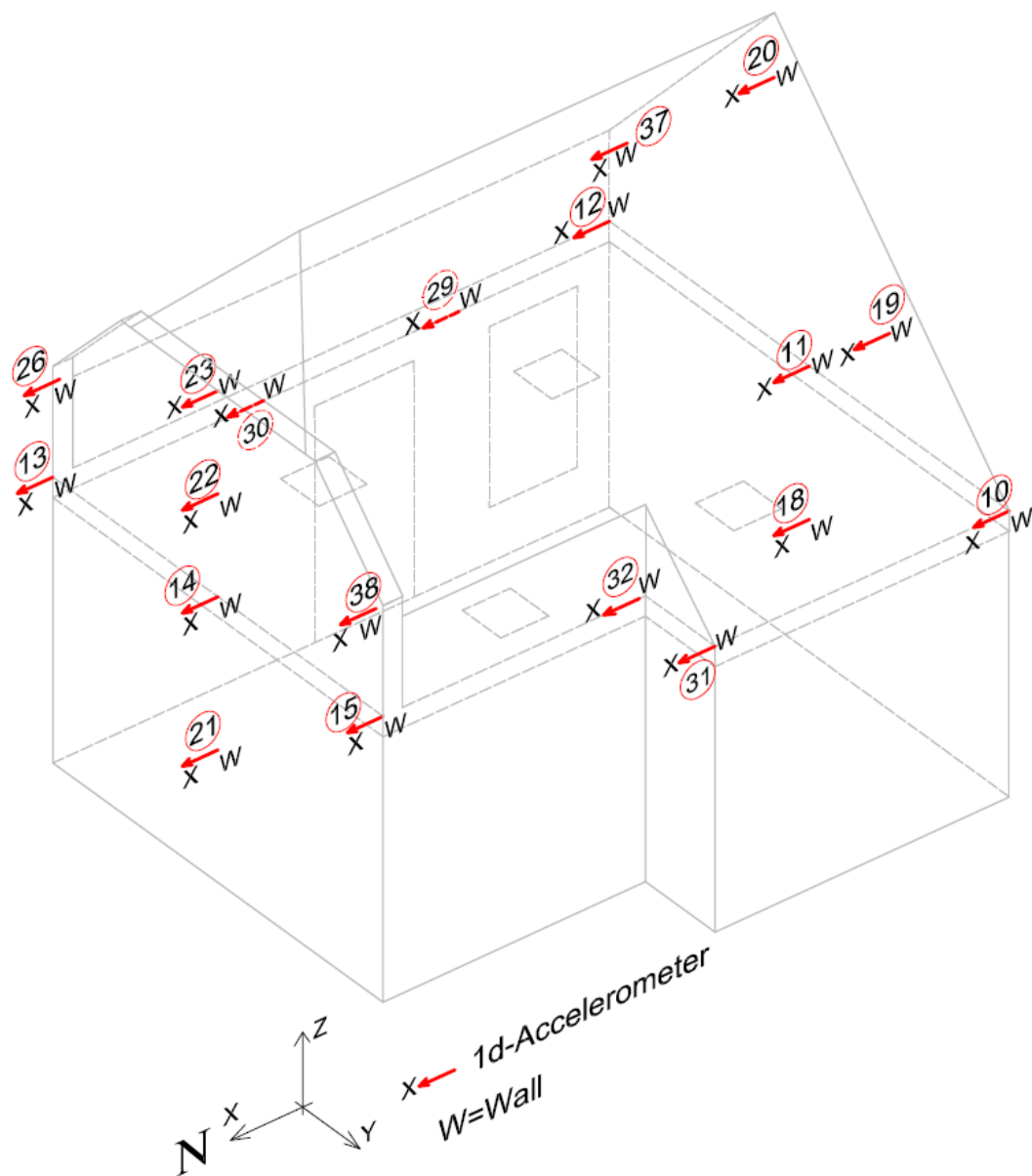


Figure 218. 3D view of accelerometers installed on masonry walls.

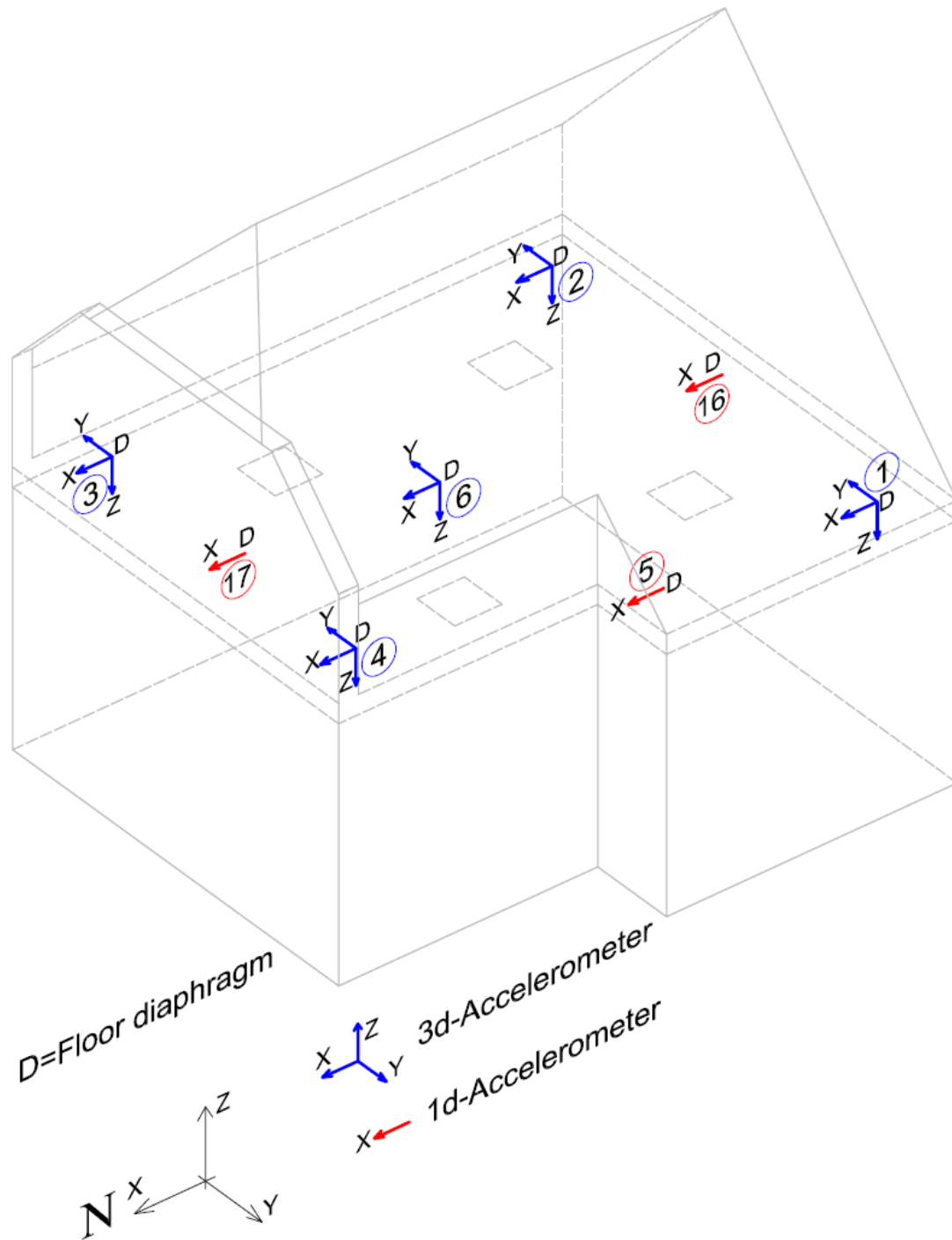


Figure 219. 3D view of accelerometers installed on the floor diaphragm intrados.

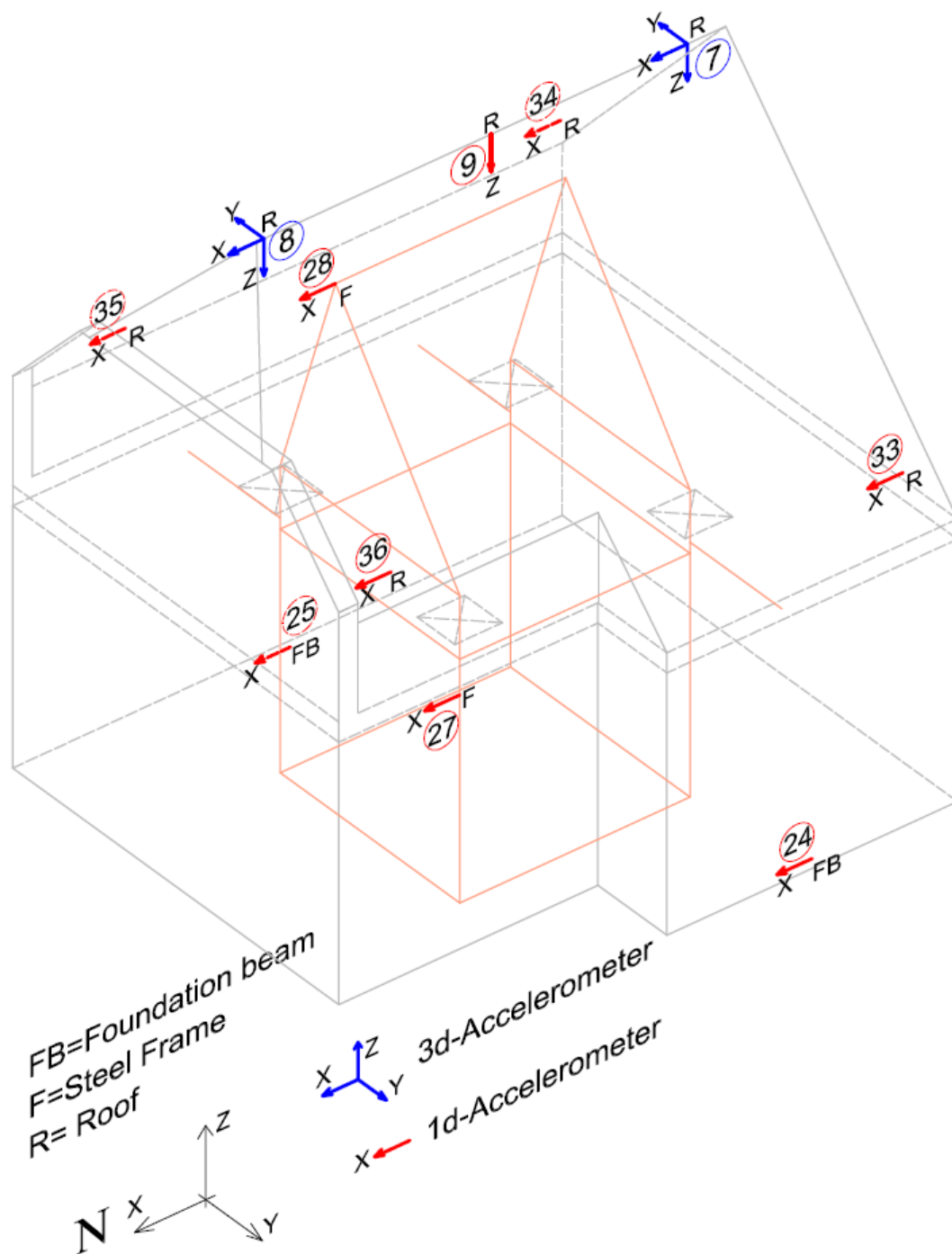
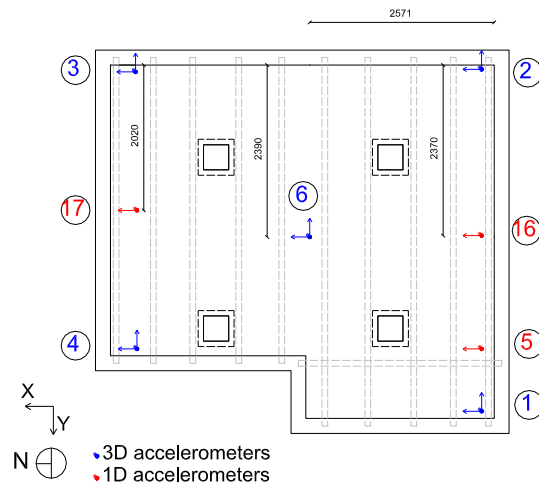


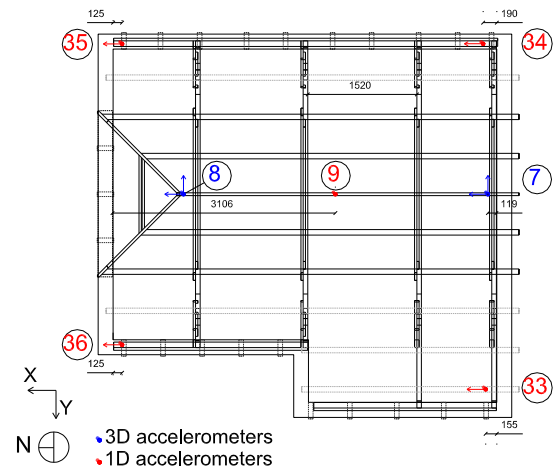
Figure 220. 3D view of accelerometers installed on the foundation beams, the steel structure and the roof.

ACCELEROMETERS ON THE FIRST FLOOR



Note: accelerometers, shown in this figure, are placed on the floor intrados, below the floorboards.

ACCELEROMETERS ON THE ROOF



Note: accelerometers, shown in this figure, are placed on the roof intrados.

Figure 221. Identification of the position of accelerometers installed on the floor diaphragm (left) and on the roof structure (right).

ACCELEROMETERS ON THE STEEL FRAME

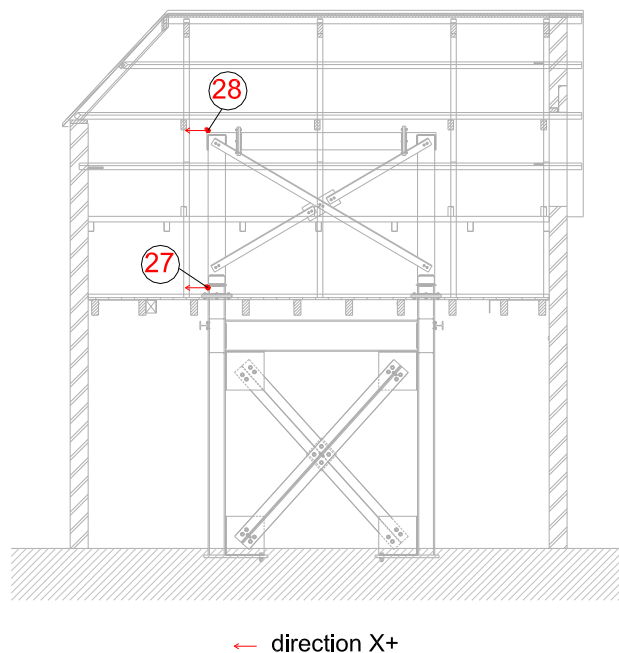


Figure 222. Identification of the position of accelerometers installed on the internal steel structure.

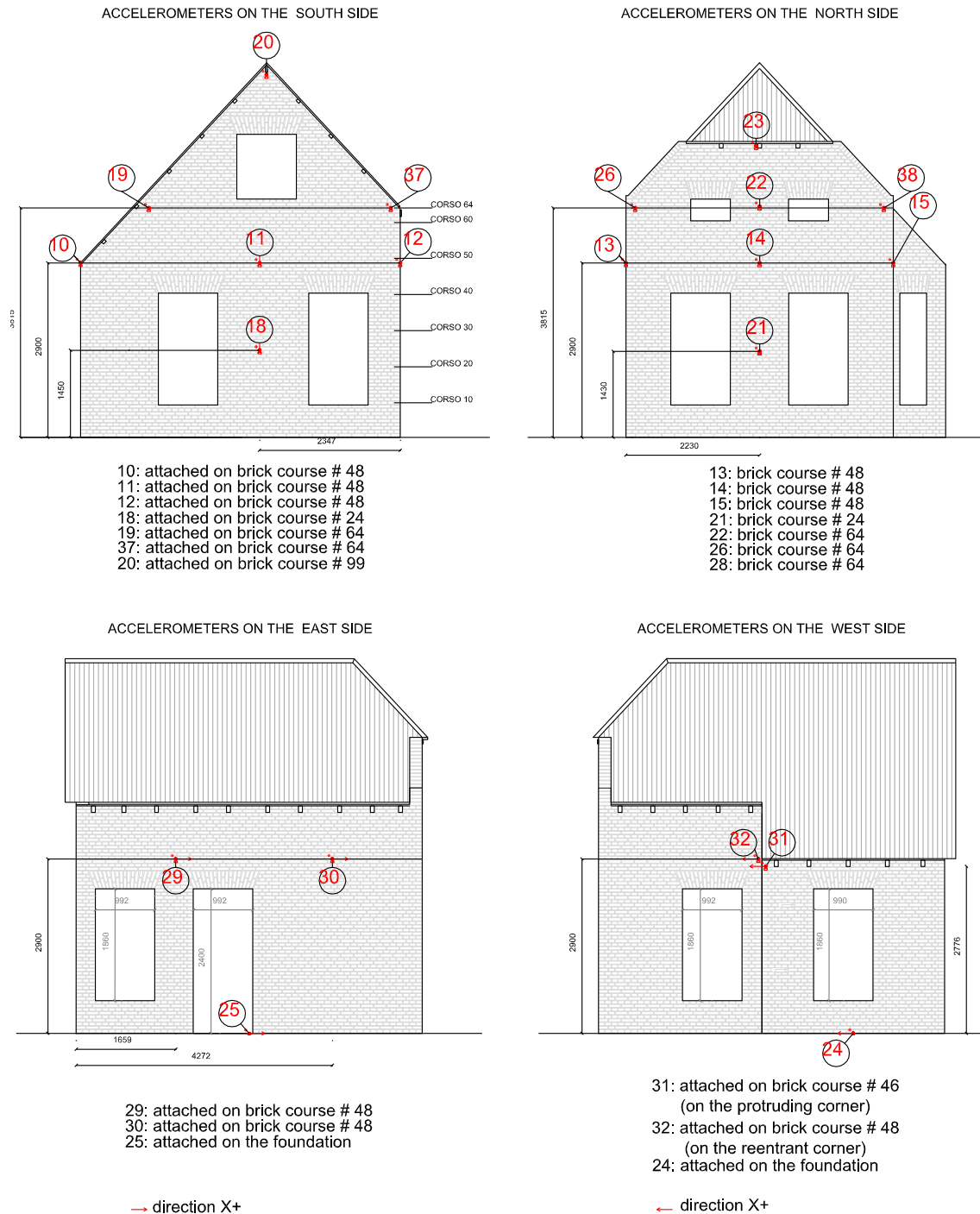


Figure 223. Identification of the position of accelerometers placed on the surface of the perimeter walls.

5.3 Test procedure

The building specimen was subjected to incremental dynamic test runs, namely a series of table motions of increasing intensity. The table input motions consist of accelerograms aiming to reproduce a realistic simulation of possible ground motions in the study area, corresponding to scenarios with different return periods. A detailed study on the seismic hazard characteristics (Appendix B-2) identified 2 main scenarios. Two records were chosen from those scenarios in order to maximize the outcome of the test, taking into account the final goal: i.e. to develop a solid experimental reference for the development of fragility models for URM buildings in the Groningen area.

5.3.1 Dynamic Input Typologies

During the test the specimen was subjected to 3 different typologies of motion: a random white noise (RNDM) (for table calibration and structural identification purposes) and two types of earthquake signals (SC1, SC2), each associated to a different scenario.

Table 31. Summary selected report.

Input	PGA [g]	Waveform name	5-75% significant duration [s]
RNDM	0.036	RNDM_035	180
RNDM	0.051	RNDM_050	180
SC1	-0.096	00201L	0.39
SC2	0.155	01703L	1.73

Figure 224 and Figure 225 show the acceleration time history of the SC1 and SC2 respectively. Figure 226 plots a comparison between the three acceleration response spectra.

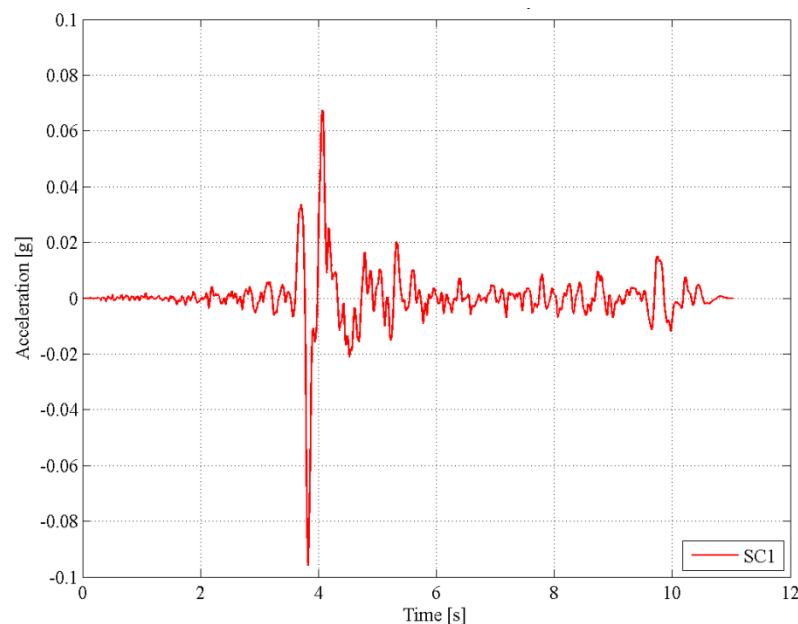


Figure 224. Acceleration time history of SC1 input.

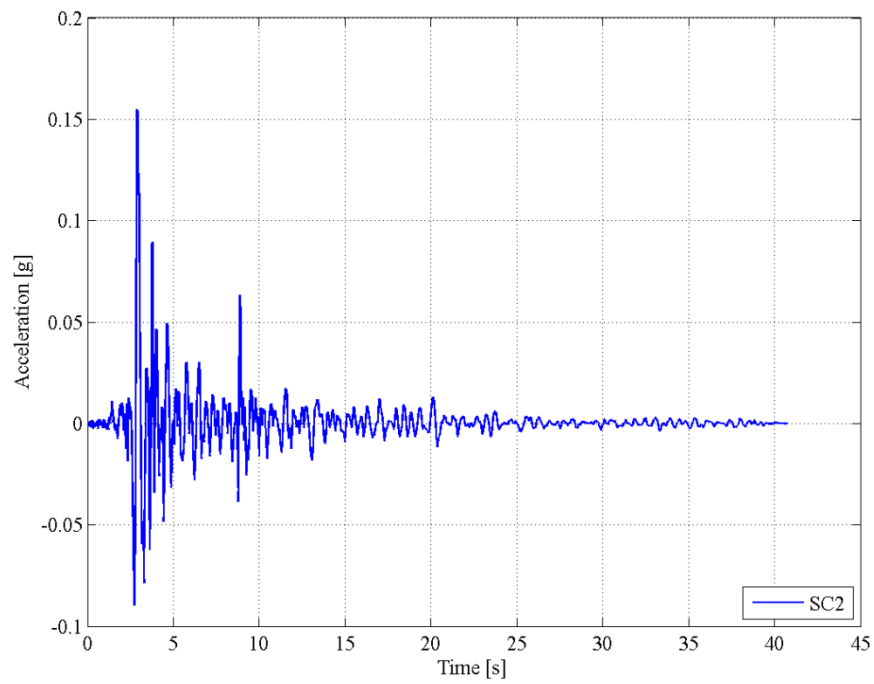


Figure 225. Acceleration time history of SC2 input.

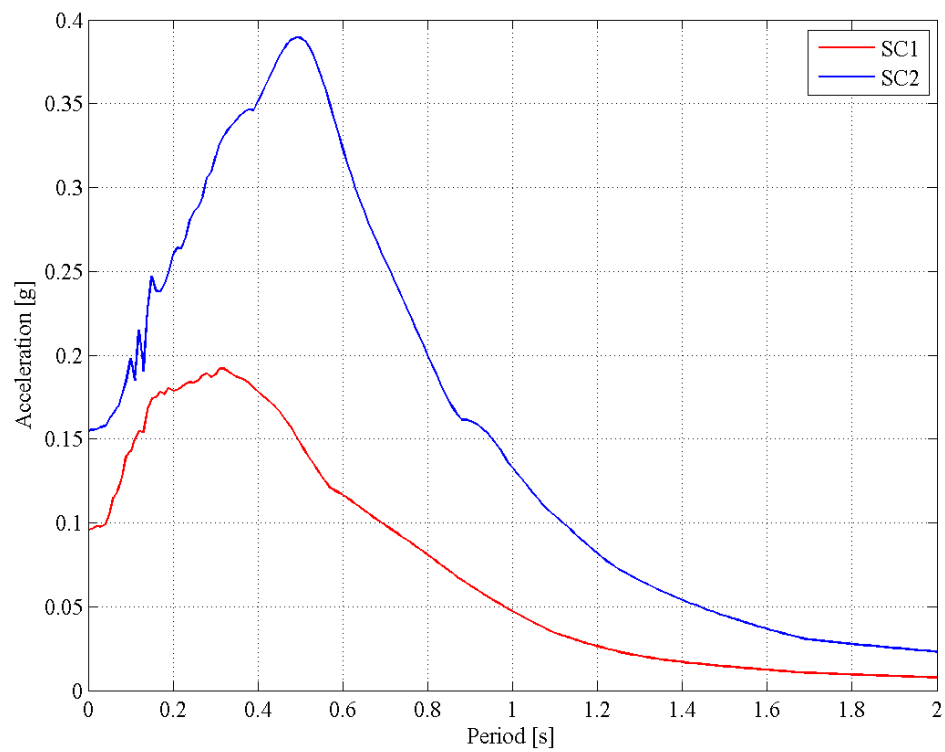


Figure 226. Acceleration response spectra comparison.



5.3.2 Applied Dynamic Input Sequence

Table 32 presents the testing sequence applied on the building specimen specifying the input typology, the test name and the PGA recorded. The bolt lines represent the incremental tests described in next sections.

Table 32. Summary testing sequence.

Date	Test #	Test Input	Test Name	Nominal PGA [g]	Recorded PGA [g]
12/04/2016	1	RNDM	01_RNDM_035	0.036	-
	2	RNDM	02_RNDM_035	0.036	-
	3	SC1	25%_SC1_024	0.024	0.026
	4	SC1	50%_SC1_047	0.048	0.053
	5	SC1	50%_SC1_047	0.048	0.050
	6	RNDM	06_RNDM_035	0.036	-
	7	SC1	100%_SC1_094	0.096	0.098
	8	RNDM	08_RNDM_035	0.036	-
	9	SC1	150%_SC1_141	0.144	0.149
	10	RNDM	10_RNDM_050	0.051	-
	11	SC2	50%_SC2_076	0.077	0.097
	12	SC2	50%_SC2_076	0.077	0.083
	13	SC2	50%_SC2_076	0.077	0.080
	14	RNDM	14_RNDM_035	0.036	-
	15	SC2	100%_SC2_152	0.155	0.140
13/04/2016	16	RNDM	16_RNDM_050	0.051	-
	17	SC2	50%_SC2_076	0.077	0.094
	18	SC2	50%_SC2_076	0.077	0.084
	19	SC2	50%_SC2_076	0.077	0.080
	20	SC2	150%_SC2_228	0.232	0.227
	21	RNDM	21_RNDM_050	0.051	-
	22	SC2	200%_SC2_304	0.310	0.293
	23	RNDM	23_RNDM_050	0.051	-
	24	SC2	250%_SC2_380	0.387	0.392
	25	RNDM	25_RNDM_050	0.051	-
15/04/2016	26	RNDM	26_RNDM_050	0.051	-
	27	SC2	100%_SC2_152	0.155	0.187
	28	SC2	100%_SC2_152	0.155	0.172
	29	SC2	100%_SC2_152	0.155	0.164
	30	SC2	300%_SC2_456	0.465	0.500
	31	RNDM	31_RNDM_050	0.051	-
	32	SC2	130%_SC2_200	0.204	0.249
	33	SC2	130%_SC2_200	0.204	0.223
	34	SC2	130%_SC2_200	0.204	0.216
	35	RNDM	35_RNDM_050	0.051	-
	36	SC2	400%_SC2_608	0.620	0.679
18/04/2016	37	RNDM	37_RNDM_050 *	0.051	-
	38	SC2	100%_SC2_152 *	0.155	0.208
	39	SC2	100%_SC2_152 *	0.155	0.167
	40	SC2	100%_SC2_152 *	0.155	0.161
	41	SC2	200%_SC2_304 *	0.310	0.283
	42	RNDM	42_RNDM_050 *	0.051	-

* Performed with a reduced amount of displacement transducers.

5.4 Test results

5.4.1 Damage pattern evolution

At the end of every stage of the shaking table testing, detailed surveys of the crack patterns developed were carried out for the detection of every possible sign of damage having affected the structure. As emerged from those surveys, the tested building behaviour was mainly governed by the out-of-plane response of the gables, although diffuse damage was observed with activation of both in-plane and out-of-plane failure mechanisms involving all of the facades of the building. Videos of the tests could be found at the following web link:

<https://www.youtube.com/playlist?list=PLRDMVFxhFvQmBoI9vm9giiPk9ymGks87P>

The following section reports the building damage evolution throughout the sequence of increasing intensity ground motions, in terms of the cracks developed in the body of the masonry and the occurrence of local damage mechanisms.

Observed damage after Test #9, 150% of SC1

At the end of testing under the first scenario's seismic excitations (scaled up to 0.144g – 150% of SC1), the building did not experience any severe eye-catching damage. The most remarkable observation regards the development of minor cracks of the paint along the perimeter of the various timber elements that are in contact with the masonry, first noticed just after testing at 100% of SC1. In particular, opening of cracks around the gutter beams, as well as along the interface between the wall plates and the top of the walls was due to the coexistence of the two different materials at these locations. The width of these cracks was negligible, as also illustrated in Figure 227.



Figure 227. Formation of cracks around gutter beams (left), and along the interface wall plates-masonry (right).

Observed damage after Test #15, 100% of SC2

During the test performed at 100% of SC2 (nominal PGA of 0.155g), the structure did not suffer any further damage, since the level of shaking intensity was similar to what the structure had already been subjected in previous tests.

Observed damage after Test #20, 150% of SC2

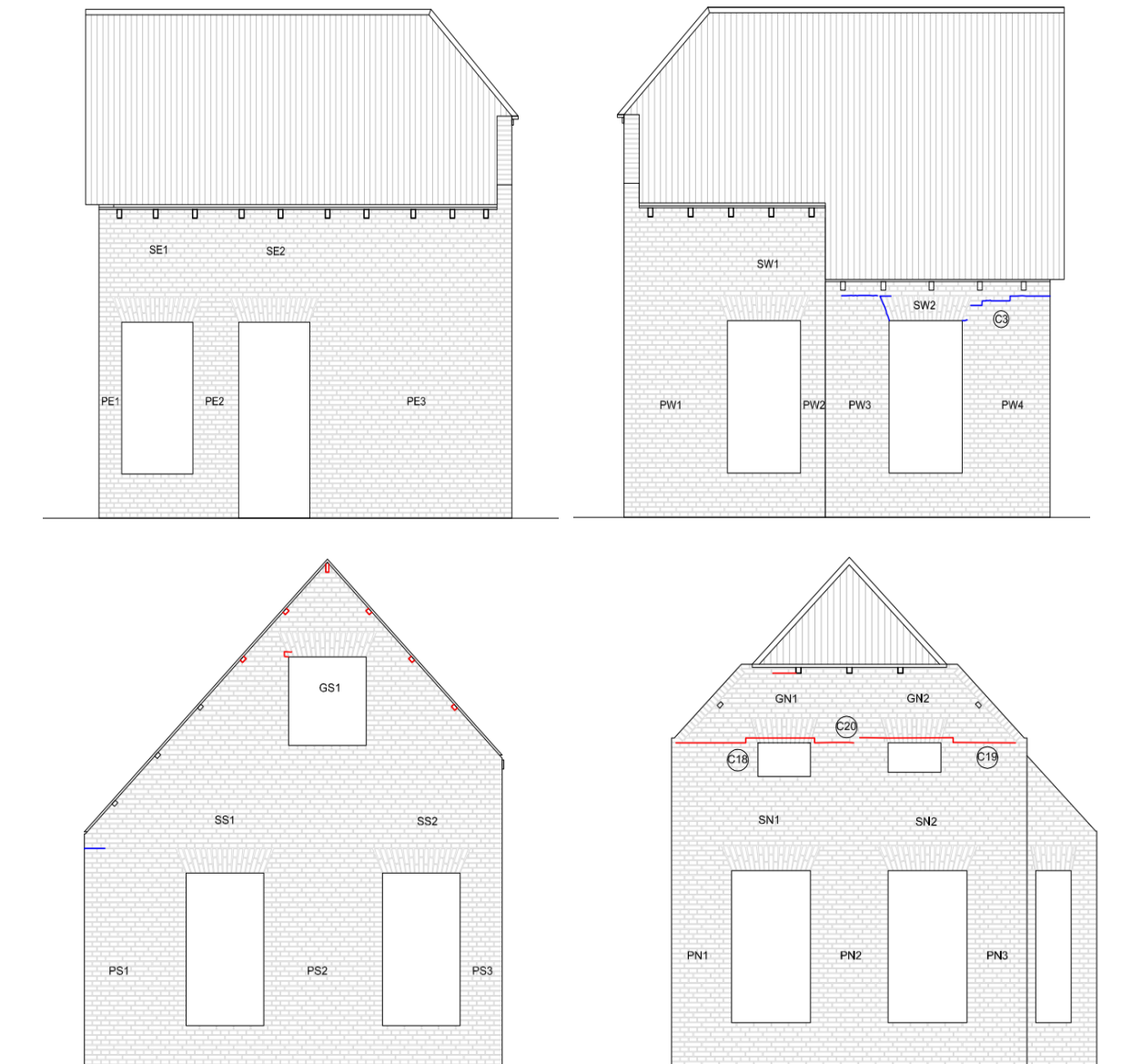


Figure 228. Illustration of the crack pattern observed after testing at 150% of SC2.

During shaking under nominal PGA of 0.232g (150% of SC2), the specimen suffered only slight damage identified mainly in the Northern gable, where the formation of a few horizontal cracks was observed just above the openings, at the level of the lintels, depicted in Figure 228 (cracks C18, C19 and C20). Originating from the interface between the timber lintels and the masonry, these cracks were extending along the whole length of the gable, whereas their width was infinitesimal. They were associated with the tendency of the gable (regions denoted as GN1 and GN2) to develop an out-

of-plane overturning mechanism, as evidenced in Figure 229, where their thickness was intentionally stressed in black for better clarity.



Figure 229. Horizontal cracks above the North gable opening.

Observed damage after Test #22, 200% of SC2

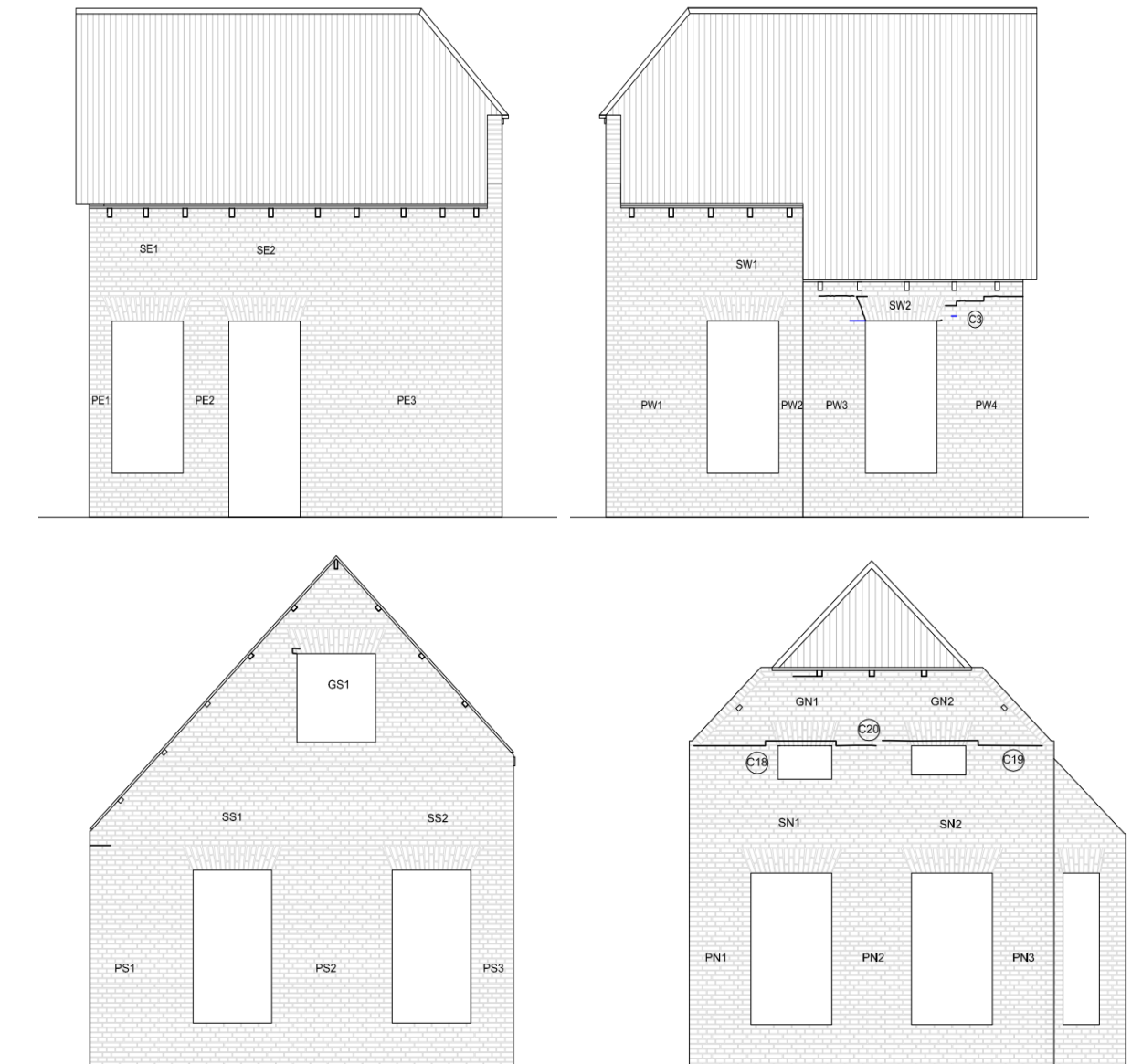


Figure 230. Illustration of the crack pattern observed after testing at 200% of SC2.

At the end of testing the building under 200% of SC2 (nominal PGA of 0.310g), no additional damage was noticed. The only cracks observed regard some minor cracks with zero residual width that became apparent in the spandrel of the West façade, SW2, as shown in Figure 231 in which the width of the cracks was emphasized in black for better clarity. They are identified in the vicinity of the intersection with the South wall, just a few rows of bricks below the floor level and they are associated with the forces transmitted from the timber joists that are present there.

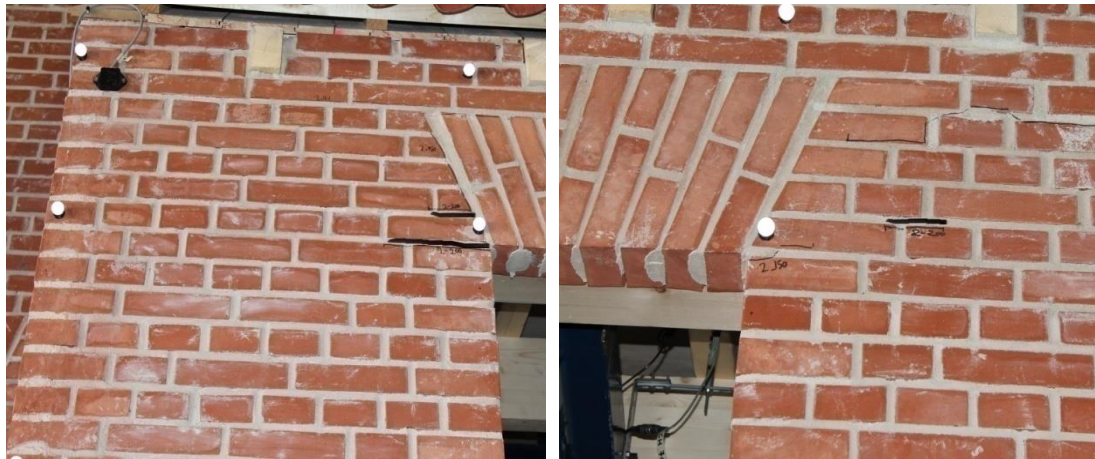


Figure 231. Cracks in the spandrel of the West façade.

Observed damage after Test #24, 250% of SC2

As a result of the detailed damage survey conducted immediately after the shaking table test at nominal PGA of 0.387g (250% of SC2), the formation of new cracks was detected in different parts of the building. On the Southern façade, horizontal cracks were developed a few centimetres above the floor level, associated with the onset of the development of an out-of-plane overturning mechanism of the gable (denoted as GS1 in Figure 232). This crack appeared at about the mid-distance between the top of the West and the top of the East façade. As depicted in Figure 232, the crack extends along most of the gable's length, but its width is imperceptibly small. Also in this case, for the sake of clarity the width of the crack was stressed in black, shown in Figure 233.

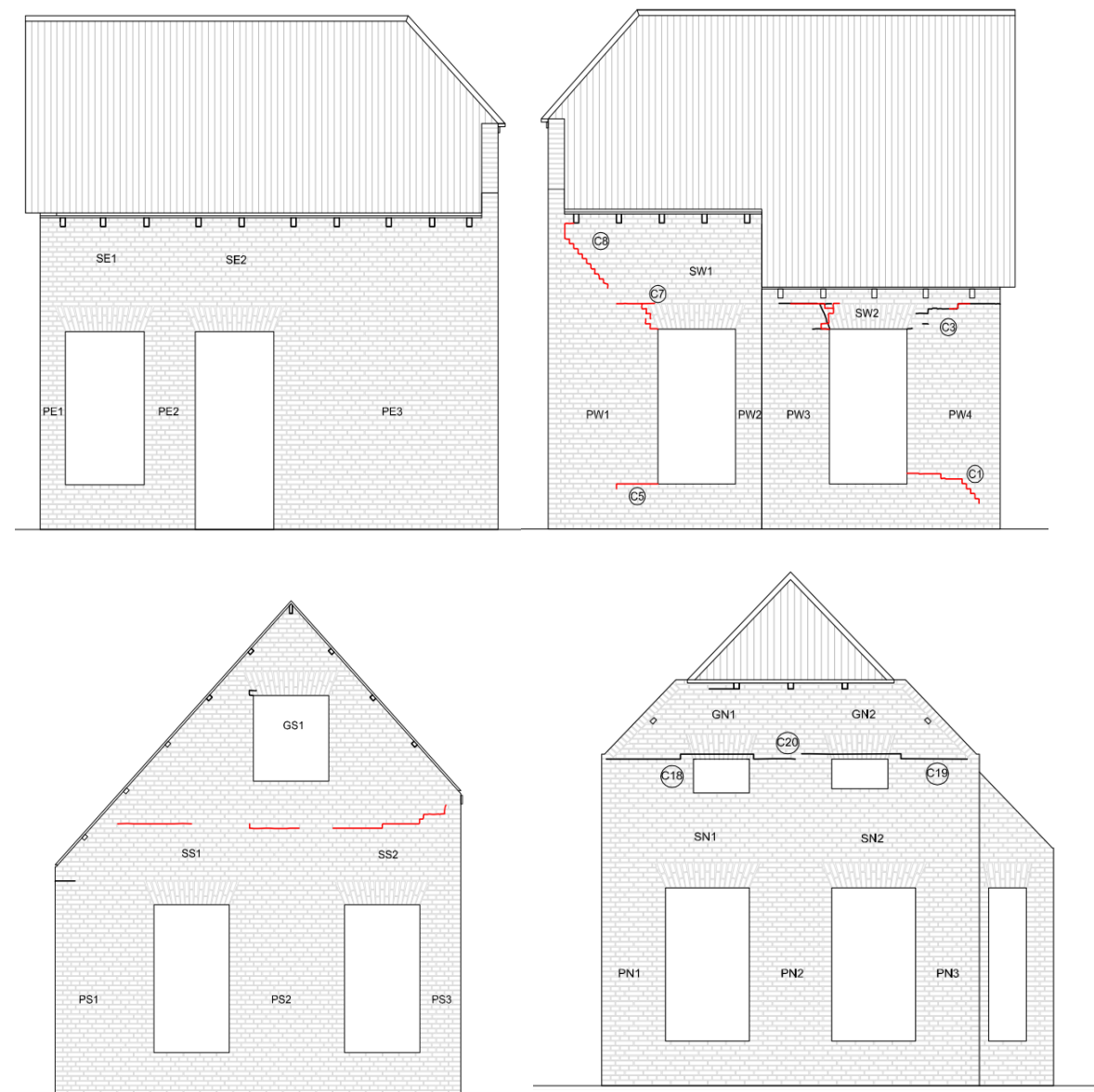


Figure 232. Illustration of the crack pattern observed after testing at 250% of SC2.



Figure 233. Horizontal crack identified in the gable of the South façade.

Other cracks were identified at the bottom of the squat piers PW1 and PW4 of the West façade, as depicted in Figure 232 and Figure 234 (cracks C1 and C5). The formation of these cracks was associated mainly to the piers' rocking behaviour, and their width was completely closed after the end of the test. On the other hand, piers PW2 and PW3, being slender enough, accommodated the deformations without cracking at this phase of testing.



Figure 234. Cracks due to flexure on the base of the piers PW1 (left), and PW4 (right) of the West façade.

Observed damage after Test #30, 300% of SC2

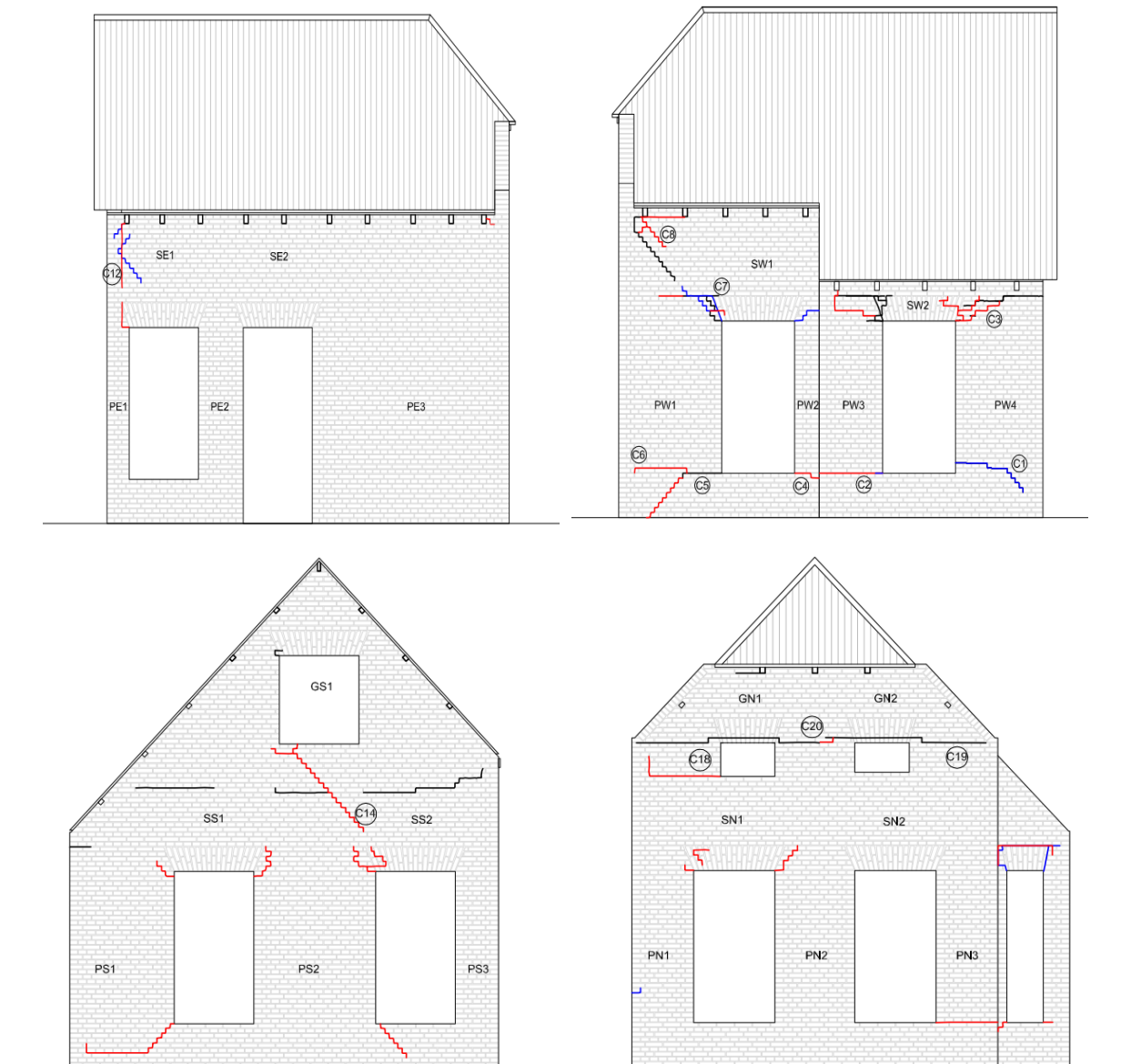


Figure 235. Illustration of the crack pattern observed after testing at 300% of SC2.

Because of the relatively high level of acceleration imposed, the specimen experienced visible damage during the test at nominal PGA of 0.465g (300% of SC2). In particular, the test performed at this level has been characterized by a global response of the structure, as evidenced by the formation of many new cracks, as well as the elongation of pre-existing cracks, documented on every one of the specimen walls (both longitudinal and transversal).

On the Southern gable, the diagonal stair-stepped crack, C14, with an angle of 45°, as shown in Figure 235 and Figure 236 (left), constitutes a clear indication of the development of an out-of-plane mechanism with unequal displacements of its eastern and western parts. Due to good interlocking between the intersecting walls, also part of the Eastern façade participates in the out-of-plane response of the South gable, resulting in the formation of the cracks C12, depicted in Figure 235 and Figure 236 (right). In

Figure 235, cracks observed only on the internal side of the walls are illustrated in red, whereas the ones that are visible on the external side are depicted in blue. All those cracks that have opened during previous tests are illustrated in black.



Figure 236. Cracks associated with the development of an out-of-plane mechanism of the gable of the Southern façade: diagonal crack at an angle of 45° (left), and cracks at the intersection with the top of the eastern façade (right).

On the Northern side, horizontal cracks have also been created just below the level of the openings of the gable (crack C18), as well as at the intersection with the longitudinal walls (crack C8 on the Western side), as shown in Figure 237. The diagonal crack C8 presents for the first time during the testing procedure a residual width of 1.0 mm.



Figure 237. Cracks associated with the development of an out-of-plane mechanism of the gable of the Northern façade.

In-plane failure modes occurred in the masonry piers of the longitudinal walls, with prevailing flexural-rocking behaviour exhibited in the West façade, as illustrated by the damage patterns observed in Figure 235. At this level of testing, all piers of the West side have exceeded the cracking limit state, and horizontal cracks indicative of such

behaviour have been developed along both top and bottom of these piers, with a width that was practically zero after the end of oscillations (Figure 238 and Figure 239).

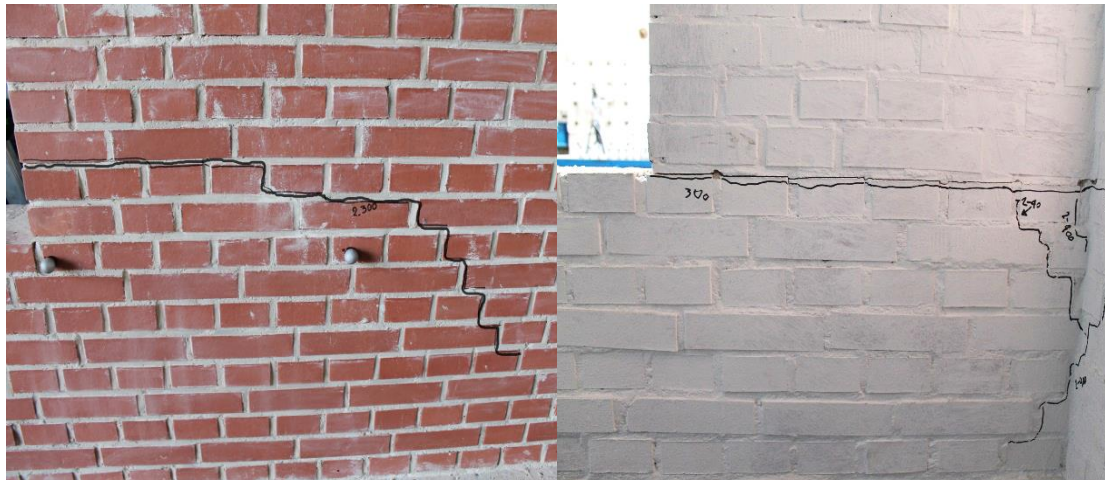


Figure 238. Cracks at the base of the piers of the West façade, indicative of their rocking behaviour: pier PW4 (left), and pier PW3 (right).



Figure 239. Cracks at the base of the piers of the West façade, indicative of their rocking behaviour: pier PW2 (left), and pier PW1 (right).

Observed damage after Test #36, 400% of SC2

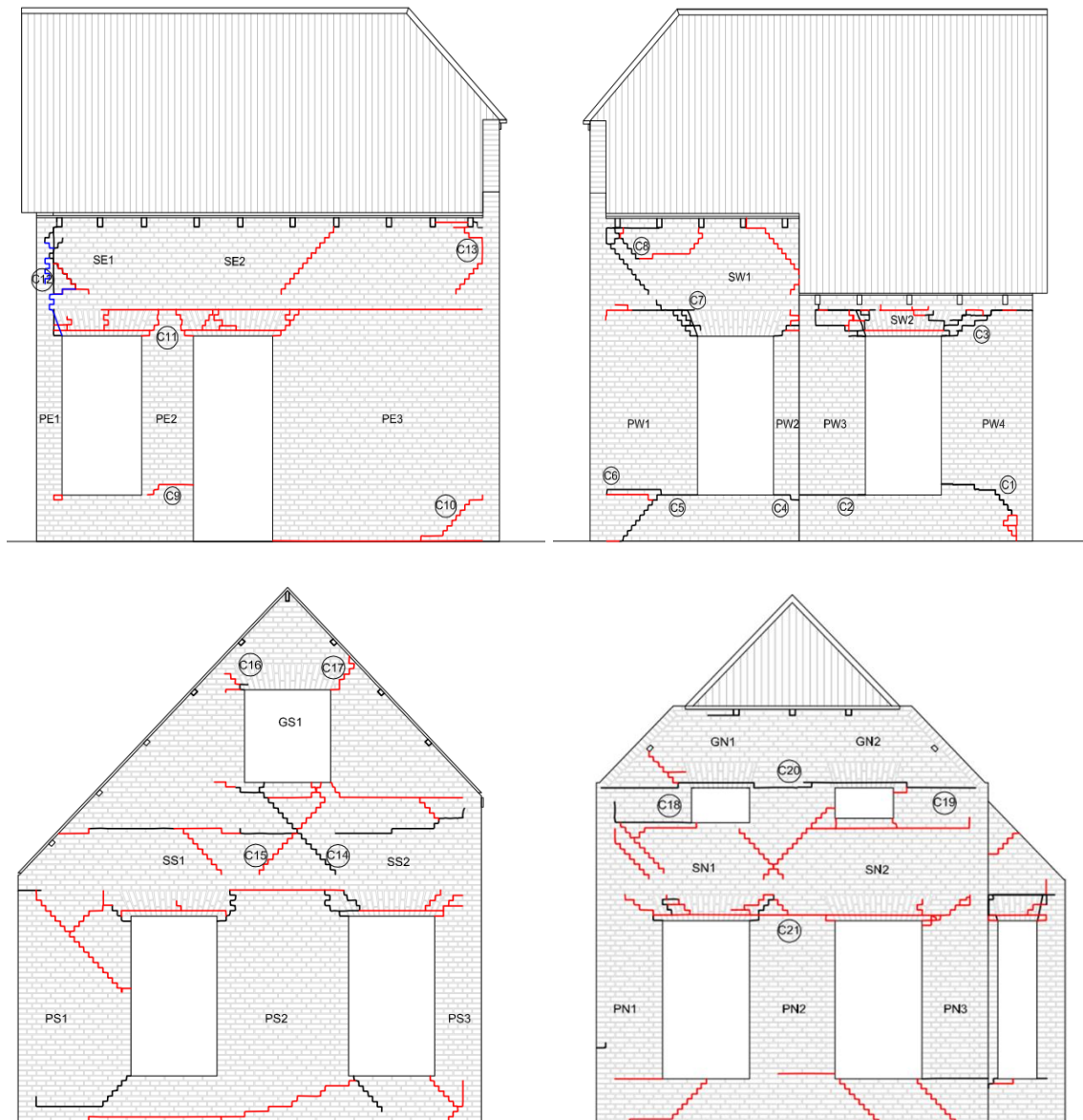


Figure 240. Illustration of the crack pattern observed after testing at 400% of SC2.

The near-collapse limit state of the structure was reached at a nominal PGA of 0.620g (400% of SC2), where the building experienced a substantial level of damage. Detailed survey of the building was conducted revealing extensive damage in the gable of the Northern façade. The activation of the out-of-plane failure mechanism involving the upper part of the gable resulted in its intensive rocking as a rigid body. Sliding was also observed at this location after failure through the horizontal cracks C20, just below the lintels level, as illustrated in Figure 240. These cracks now exhibit considerably high residuals of the order of 2 cm, while in some cases de-cohesion of masonry blocks also occurred, as shown in Figure 241.

The tendency of the North façade to overturn has been clearly manifested by the crack patterns observed both in this façade and at its intersection with the longitudinal ones.

In particular, the diagonal cracks originating from the corners of the openings of this wall, in combination with the cracks C8 and C13, located at the top of the East and West façades, respectively, indicate the development of an out-of-plane mechanism involving the entire façade, as well as parts of the longitudinal walls. The good interlocking between the intersecting walls led to observation of the flange effect and the formation of a C shape. To a great extent, most of the deformations were absorbed by the cracks C8 and C13 that exhibit residual width of 4 mm and 8 mm, respectively.



Figure 241. Failure through horizontal cracks in the gable of the Northern facade.

On the Southern side, the out-of-plane overturning mechanism has also been fully activated after the formation of an X-shaped crack pattern below the opening, C14 and C15, as well as the diagonal cracks originating from the upper corners of the window, C16 and C17, shown in Figure 242. The maximum residual width of these diagonal cracks was 2 mm. The development of this type of crack pattern is mainly attributed to the presence of the steel tie at the floor level that, having been well-embedded in the body of masonry, does not allow excessively high deflections of the wall at this location. During the demolition phase of the building, on the North side, the tensile cracking of the brick on which the steel tie was connected is clear evidence of the high stresses that were developed at this location.



Figure 242. Cracks of X-shape associated with the development of an out-of-plane mechanism of the gable of the Southern façade.



Figure 243. Failure of the brick on which the steel tie was connected.

The piers of the West façade exhibited a pure flexural failure mechanism, where the pre-existing cracks that had opened during preceding tests were enlarged and extended along the base and the top. These cracks present residual widths between 1.5 mm and 3 mm.

Regarding the damage noticed in the long pier of the Eastern wall, perceptible cracks were developed only during this last test. More specifically, this squat pier, developed a flexural mechanism of rocking for motion toward the South direction, with subsequent sliding all along its base of approximately 0.2 mm. A stair-stepped cracking pattern through the mortar joints, with opening no greater than 0.2 mm (crack C10), shown in Figure 244, pointed out the importance of the flange participation of the transversal North wall in the long pier's in-plane response.

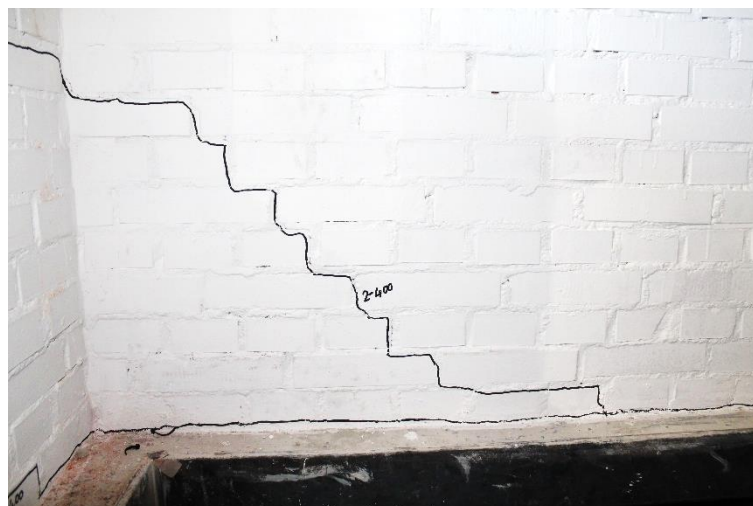


Figure 244. Stair-stepped and horizontal crack at the base of the long pier of the East façade.

Regarding the condition of the supports of the timber roof structure after testing, no sliding between the wall plates and the longitudinal walls has been observed, as shown in Figure 245. Furthermore, no sliding or differential displacements have been noted at the supports of the roof on the two gables, although the steel ties that existed at these locations were not screwed on the timber beams (see Figure 246). This is due to the well-restrained connections of the roof timber beams and the gable walls, realized as illustrated in Figure 247.



Figure 245. Roof supports at South-East corner (left), and East wall (right).

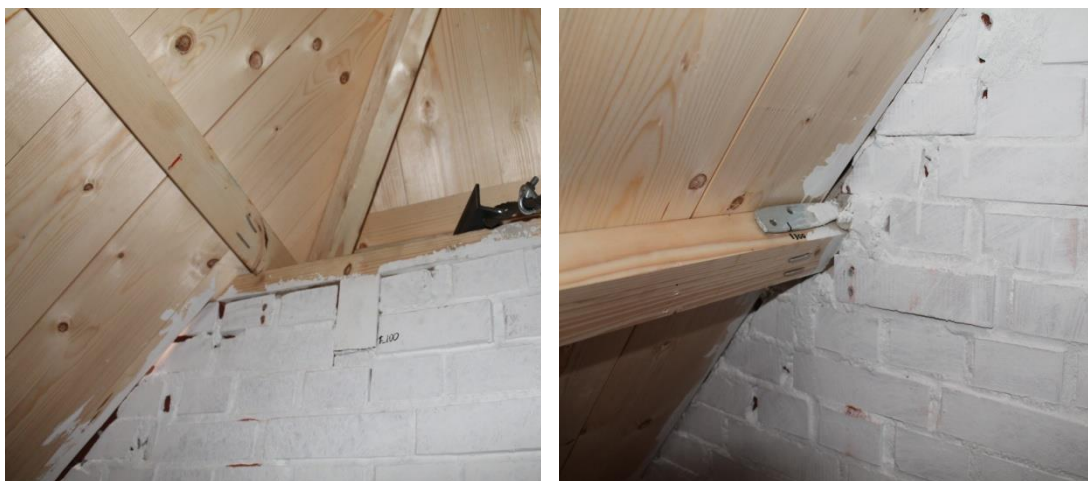


Figure 246. Roof supports in the Northern gable (left), and Southern gable (right).



Figure 247. Well-restrained connection of the roof timber beams to the South gable wall.

Cracks are identified at the locations where the timber sheathing of the roof was in contact with the gable walls, shown in Figure 248 and Figure 249. Cracks along these areas are due to pounding of the roof on the masonry gable walls. Furthermore, significant damage was also reported at the supports of the timber rafters of the roof on the wall-plates due to failure of the nailed connections, as shown in Figure 250 and Figure 251.



Figure 248. Cracks due to pounding of the roof on the North gable (left), and South gable (right).



Figure 249. Crack due to pounding of the roof on the small parapet of the West façade.



Figure 250. Damage observed at the nailed connections of the timber roof.

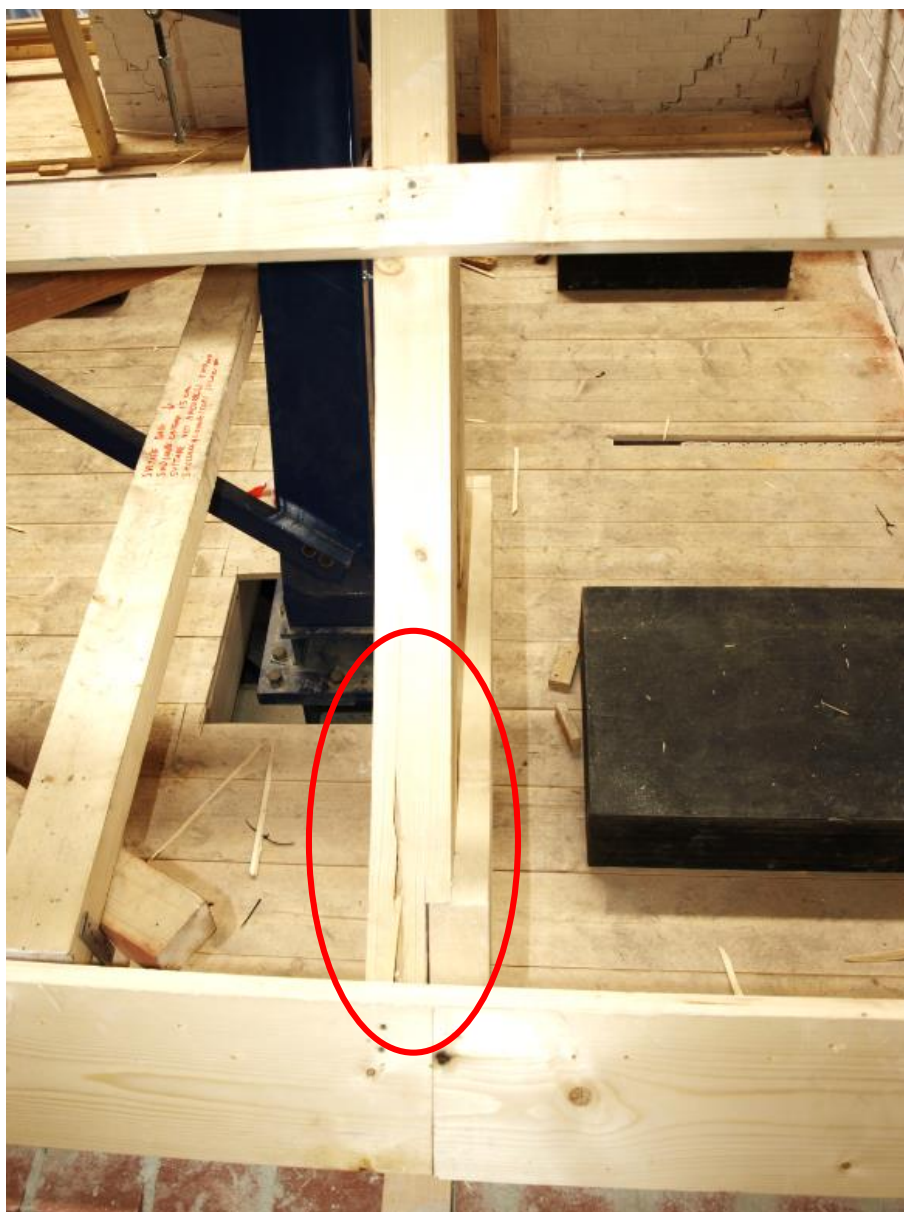


Figure 251. Damage identified at one of the nailed connections between the timber elements of the roof.

Finally, during the tests performed as simulation of potential aftershocks experienced by the damaged building (200% of SC2 – nominal PGA of 0.310g), development of new cracks or significant enlargement of the cracks that opened during the preceding tests was not observed. However, even though these input motions were substantially modest in comparison to the strong motions having already been applied, widening of cracks was reported in many cases. The following table (Table 33) summarizes the width of the most significant cracks observed during the sequence of the incremental shaking table testing. At the end of the testing procedure, the lintel of the southern opening on the West façade was completely damaged, as shown in Figure 252.



Figure 252. Near Collapse status of the lintel of the southern opening on the West façade.



The following table summarizes the damage induced in the full-scale specimen, in terms of cracks and their corresponding widths observed in the masonry pier.

Table 33. Residual width of the most remarkable cracks observed after the end of every test.

Residual width of cracks [mm]					
Façade	Crack name	Test			
		#24, 250% of SC2	#30, 300% of SC2	#36, 400% of SC2	Aftershock, 200% of SC2
West	C1	/	/	3.0	3.5
West	C2	/	/	1.5	2.0
West	C3	/	/	10.0	5.0
West	C4	/	/	2.0	2.0
West	C5	/	/	1.5	1.5
West	C6	/	/	1.5	1.5
West	C7	/	/	4.0	5.0
West	C8	/	1.0	4.0	10.0
East	C9	/	/	0.1	0.1
East	C10	/	/	0.2	0.2
East	C11	/	/	1.0	1.0
East	C12	/	/	8.0	10.0
East	C13	/	0.8	1.5	2.0
South	C14	/	/	2.0	2.0
South	C15	/	/	1.5	2.0
South	C16	/	/	0.4	0.6
South	C17	/	/	0.4	0.6
North	C18	/	/	1.5	1.5
North	C19	/	/	1.0	2.0
North	C20	/	/	20.0	25.0
North	C21	/	/	10.0	10.0

5.4.2 Test #3_25%_SC1_024

Figure 253 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

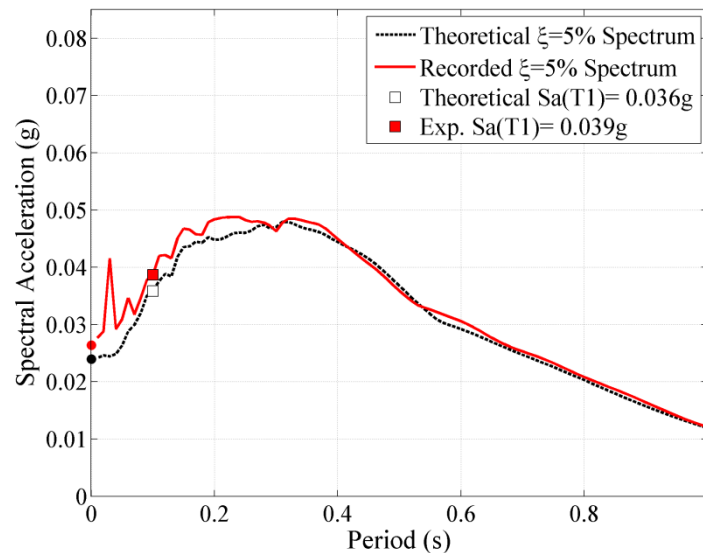


Figure 253. Test#3_25%_SC1_024: Theoretical-Experimental Spectral Comparison.

At this stage the specimen behaved essentially as a rigid body. The displacements were generally lower than the wire potentiometer sensitivity. Figure 254 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer.

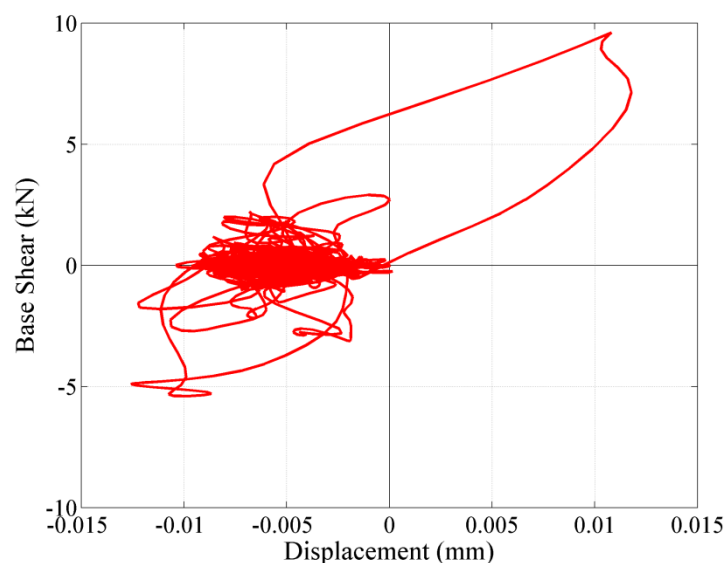


Figure 254. Test #3_25%_SC1_024: Base Shear vs 1st Floor Displacement.

5.4.3 Test #5_50%_SC1_047

Figure 255 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

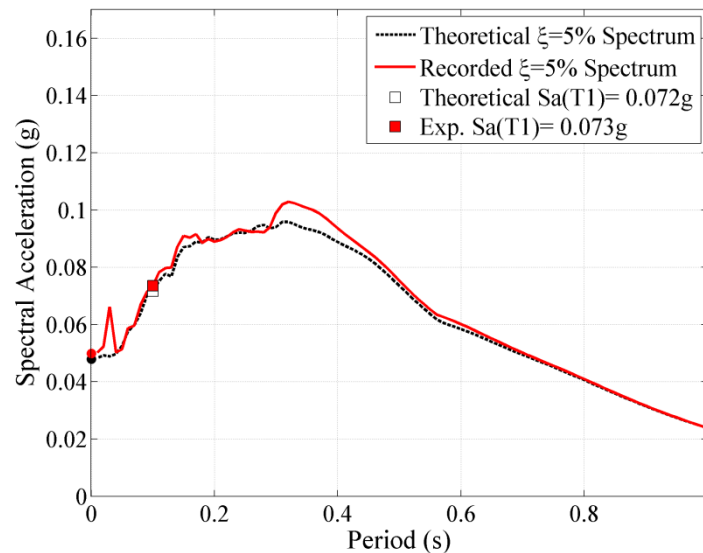


Figure 255. Test#5_50%_EQ1_047: Theoretical-Experimental Spectral Comparison.

Figure 256 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

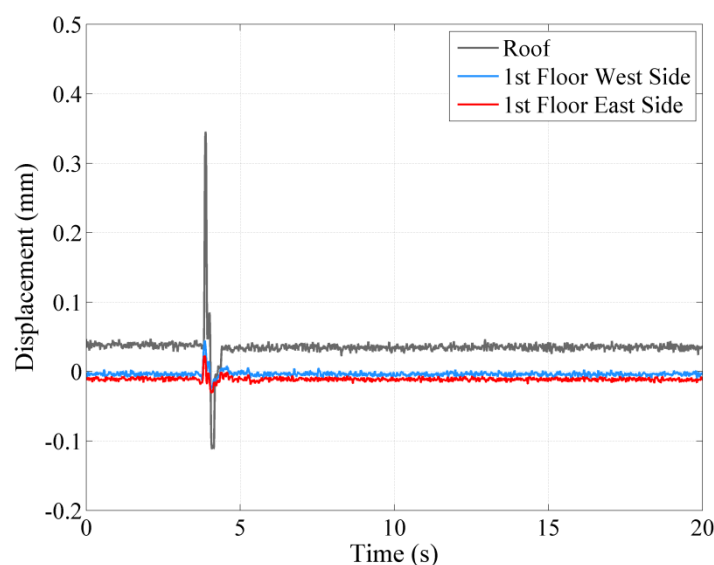


Figure 256. Test #5_50%_SC1_047: Displacement Histories.

Figure 257a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 257b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (3.15 m).

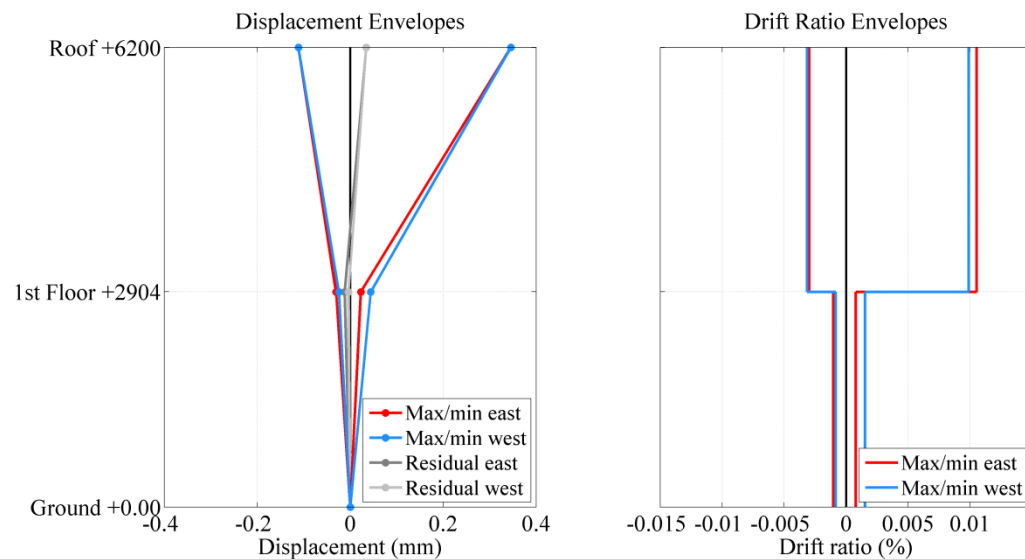


Figure 257. Test #5_50%_SC1_047: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 258 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

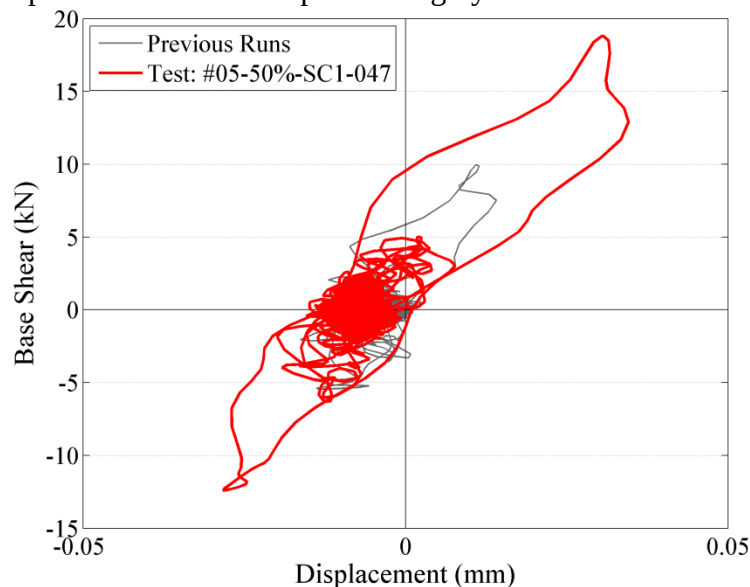


Figure 258. Test #5_50%_SC1_047: Base Shear vs 1st Floor Average Displacement.

Figure 259 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The forces calculated from the accelerometers on each side have been assigned to the

corresponding wall. The forces calculated from the accelerometers located in the middle of the building have been split equally between the two walls. The hysteretic response from previous tests is also plotted in grey.

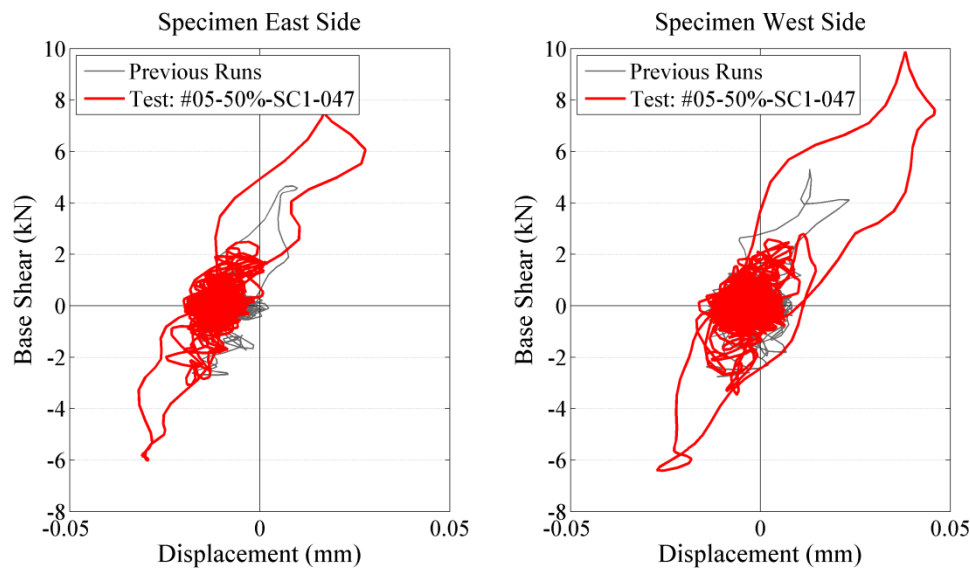


Figure 259. Test #5_50%_SC1_047: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 260 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The displacements of the 4 corner points were monitored by two orthogonal potentiometers each in the longitudinal and transverse directions. The potentiometers assigned to the SE, NE, NW, and SW corners were respectively: #14 and #15 (longitudinal and transverse direction), #16 and #17, #19 and #20, #10 and #11. The displacements along the floor span were monitored only by longitudinal potentiometers: #12 and #13 along the South side, and #18 along the North Side. The trajectories of the corner points were calculated accounting for the rotation of the potentiometers. Knowing these trajectories, the procedure proposed by Seible *et al.* (1991) was implemented in order to compute the floor shear deformation history. The floor deflected shape is magnified by the scaling factor indicated on the figure.

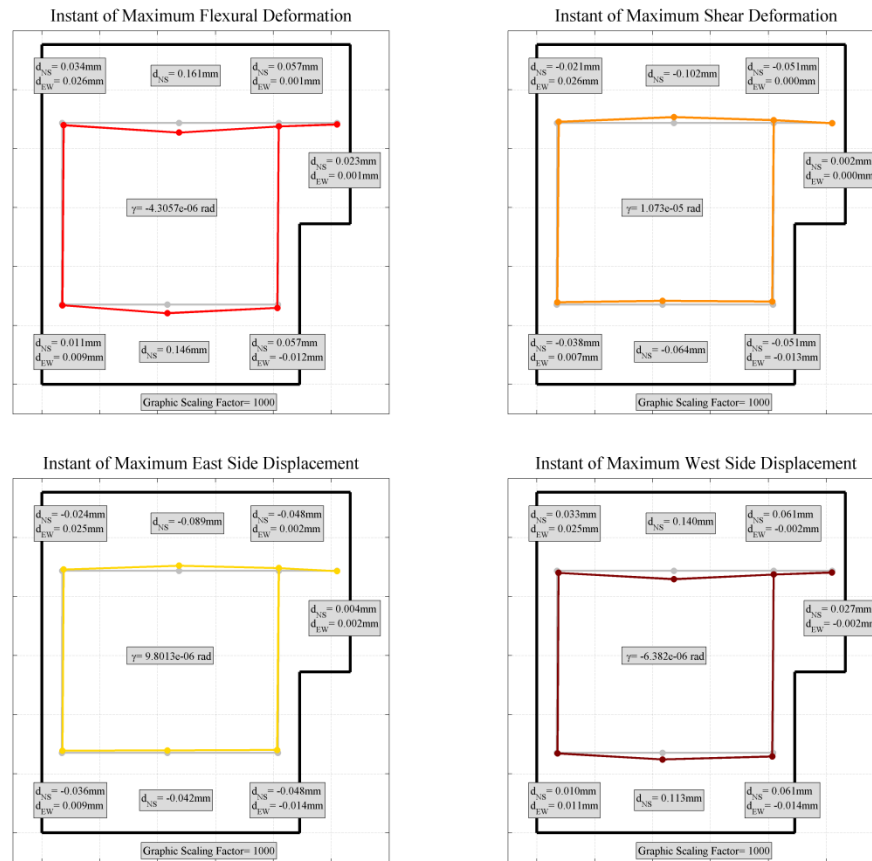


Figure 260. Test #5_50%_SC1_047: Floor deflected shapes at four test instants.

Figure 261 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The horizontal OOP displacements of the North and South walls at the floor level were computed by summing the floor average displacement with the wall-to-floor relative displacements, measured by potentiometer #22 in the South wall and potentiometer #25 in the North wall. All other horizontal OOP displacements were measured by wire potentiometers. The deflected shape is magnified by the scaling factor indicated on the figure.



Figure 261. Test #5_50%_SC1_047: Building Longitudinal Cross Section Deformed Shape.

5.4.4 Test #7_100%_SC1_094

Figure 262 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

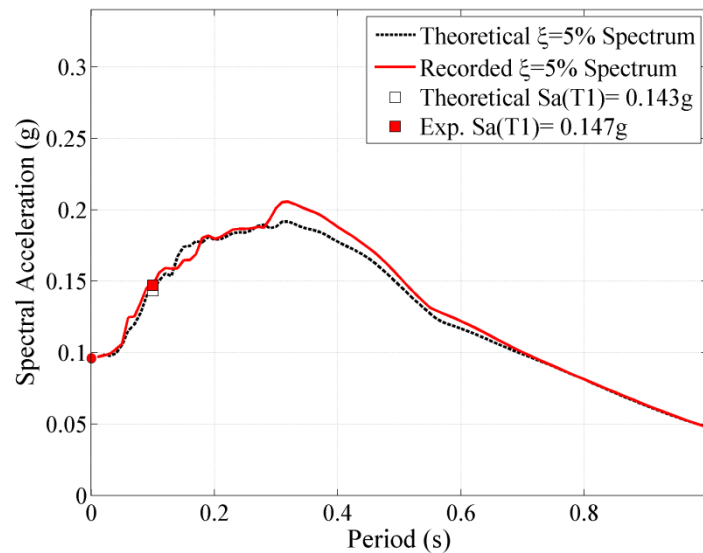


Figure 262. Test #7_100%_SC1_094: Theoretical-Experimental Spectral Comparison.

Figure 263 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

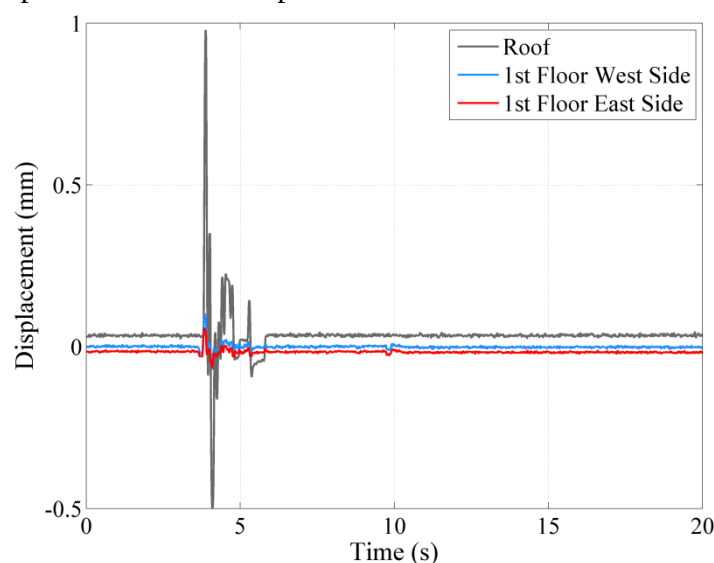


Figure 263. Test #7_100%_SC1_094: Displacement Histories.

Figure 264a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 264b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

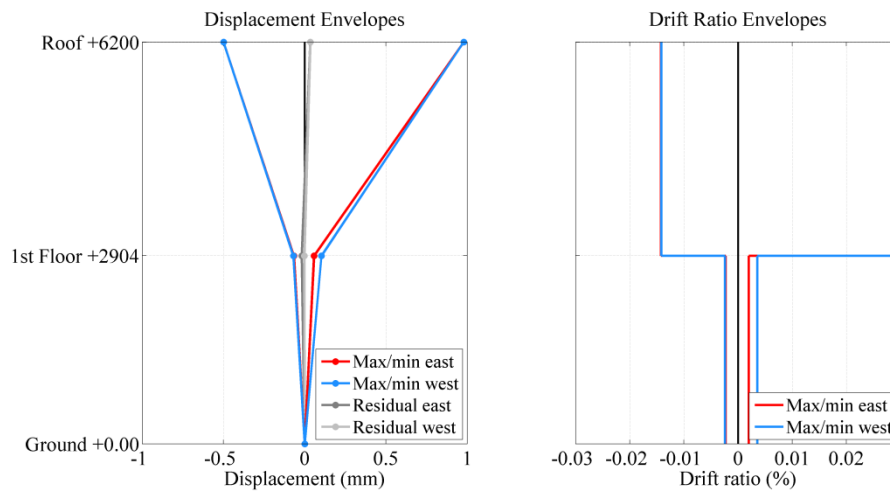


Figure 264. Test #7_100%_SC1_094: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 265 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

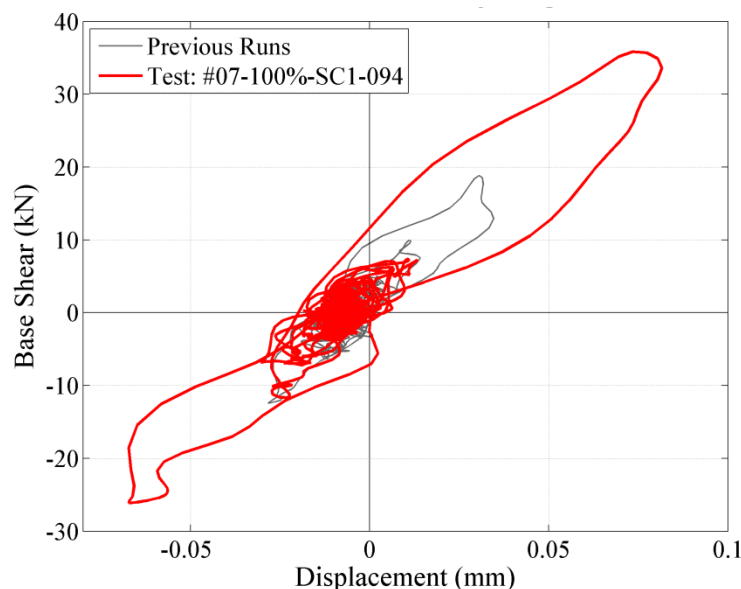


Figure 265. Test #7_100%_SC1_094: Base Shear vs 1st Floor Average Displacement.

Figure 266 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

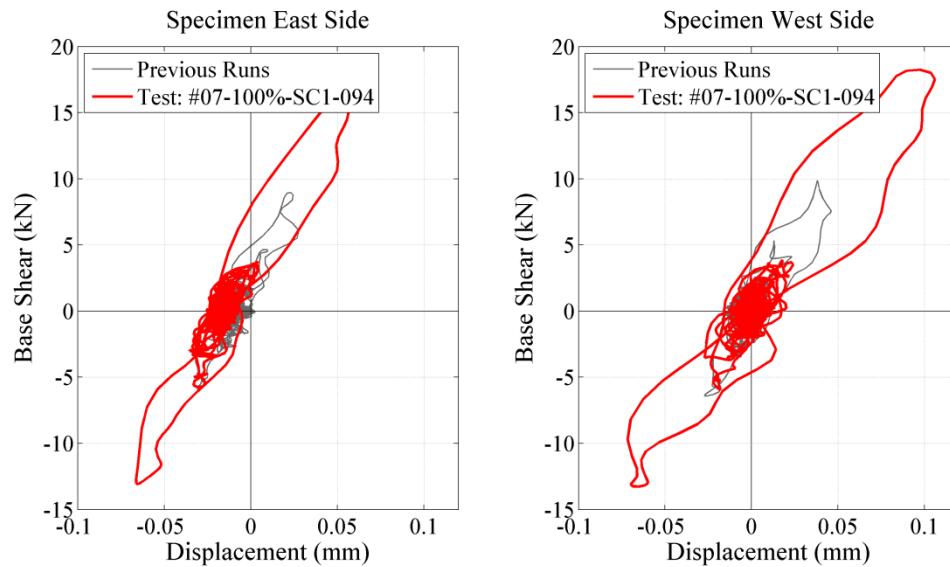


Figure 266. Test #7_100%_SC1_094: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 267 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

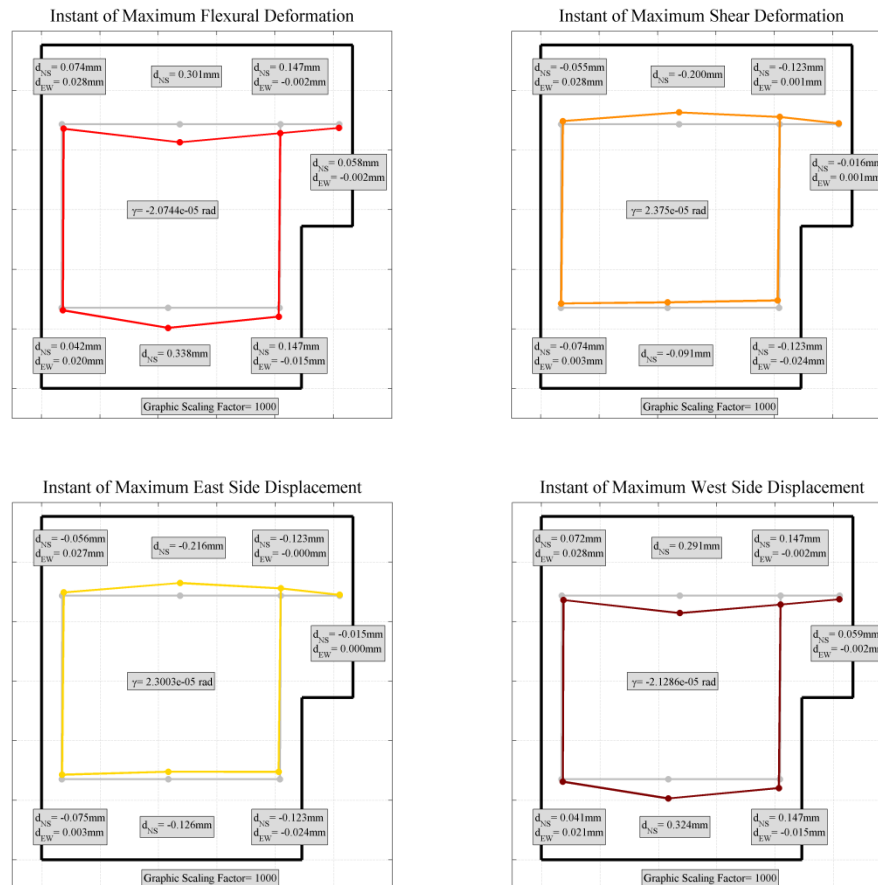


Figure 267. Test #7_100%_SC1_094: Floor deflected shapes at four test instants.

Figure 268 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

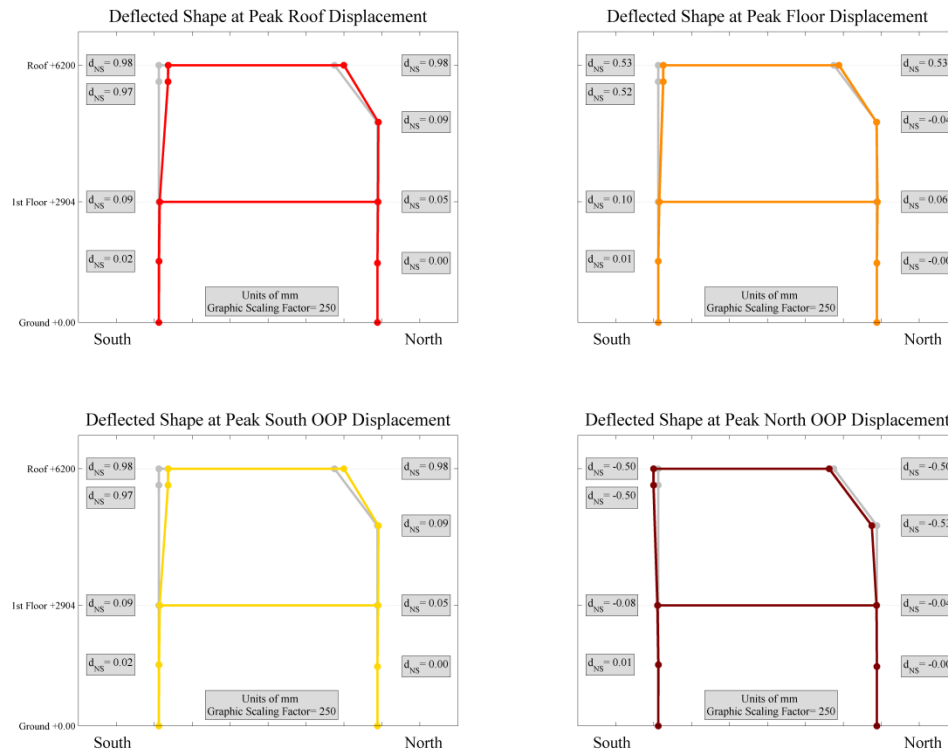


Figure 268. Test #7_100%_SC1_094: Longitudinal Cross Section Deformed Shape

5.4.5 Test #9_150%_SC1_141

Figure 269 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

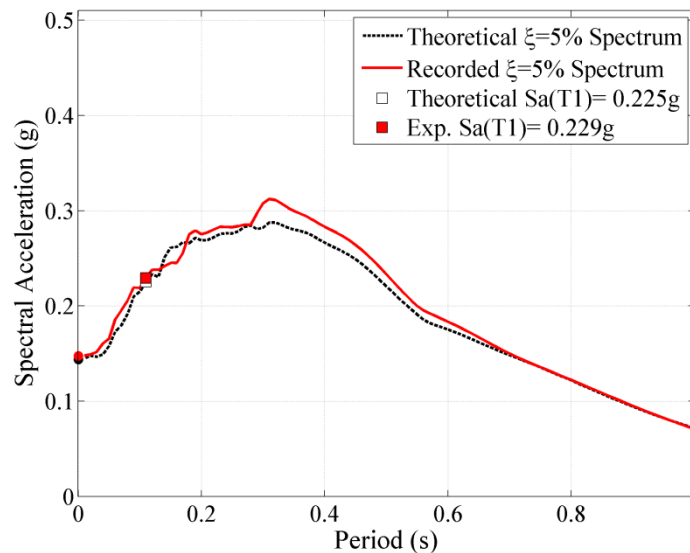


Figure 269. Test #9_150%_SC1_141: Theoretical-Experimental Spectral Comparison.

Figure 270 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

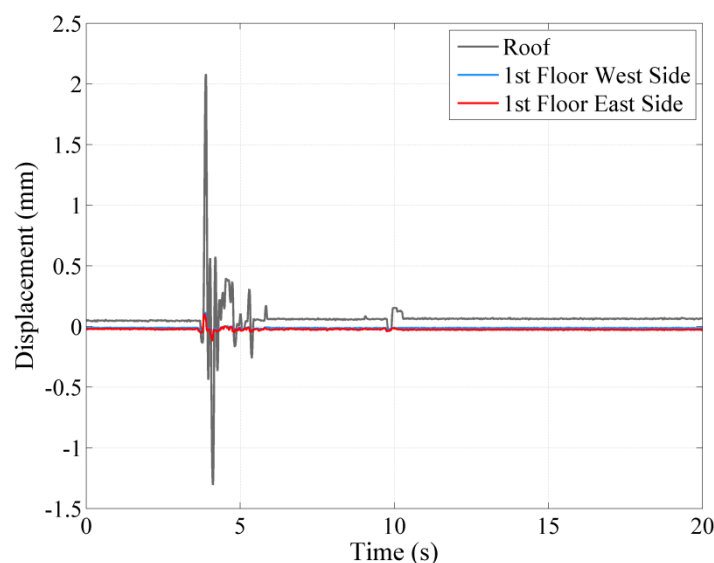


Figure 270. Test #9_150%_SC1_141: Displacement Histories.

Figure 271a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 271b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

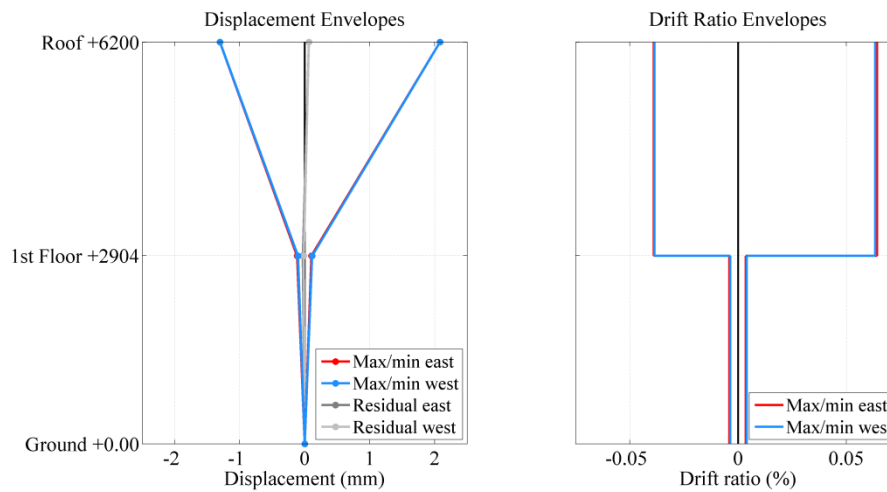


Figure 271. Test #9_150%_SC1_141: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 272 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

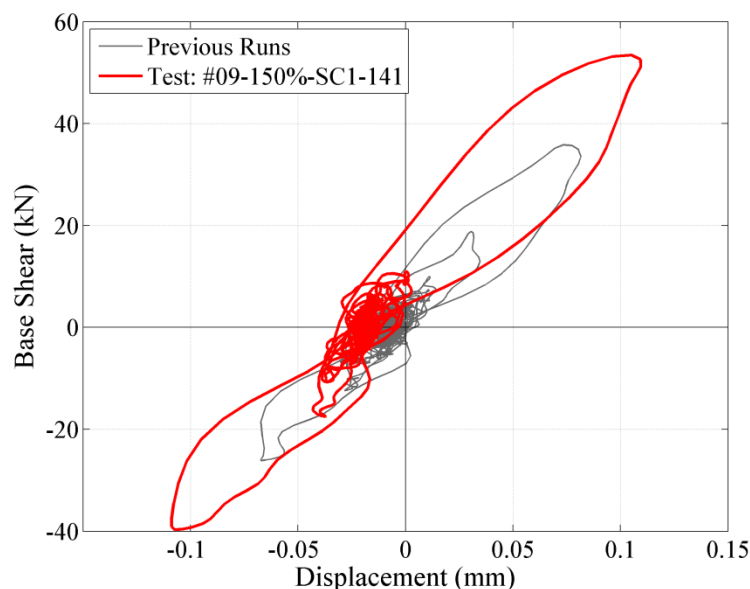


Figure 272. Test #9_150%_SC1_141: Base Shear vs 1st Floor Average Displacement.

Figure 273 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

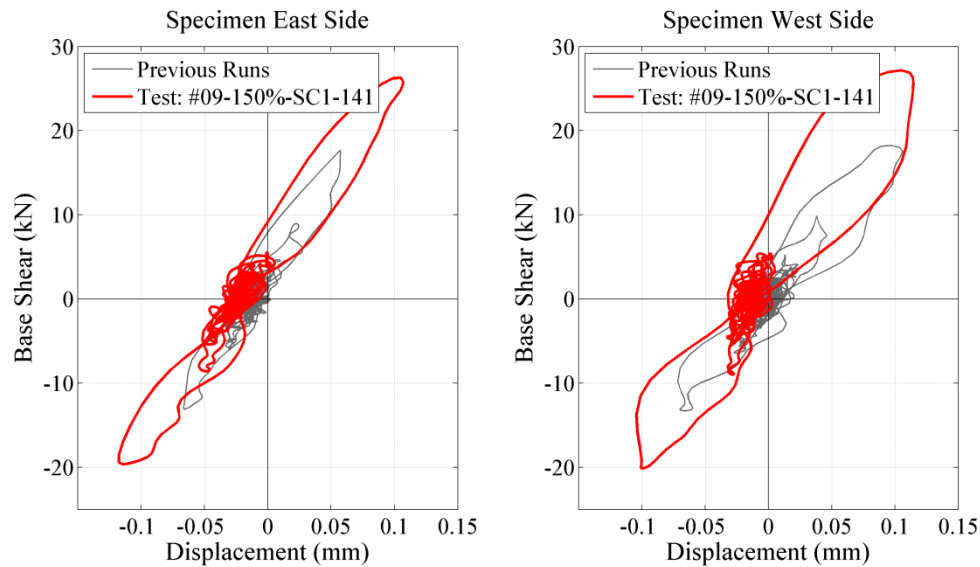


Figure 273. Test #9_150%_SC1_141: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 274 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

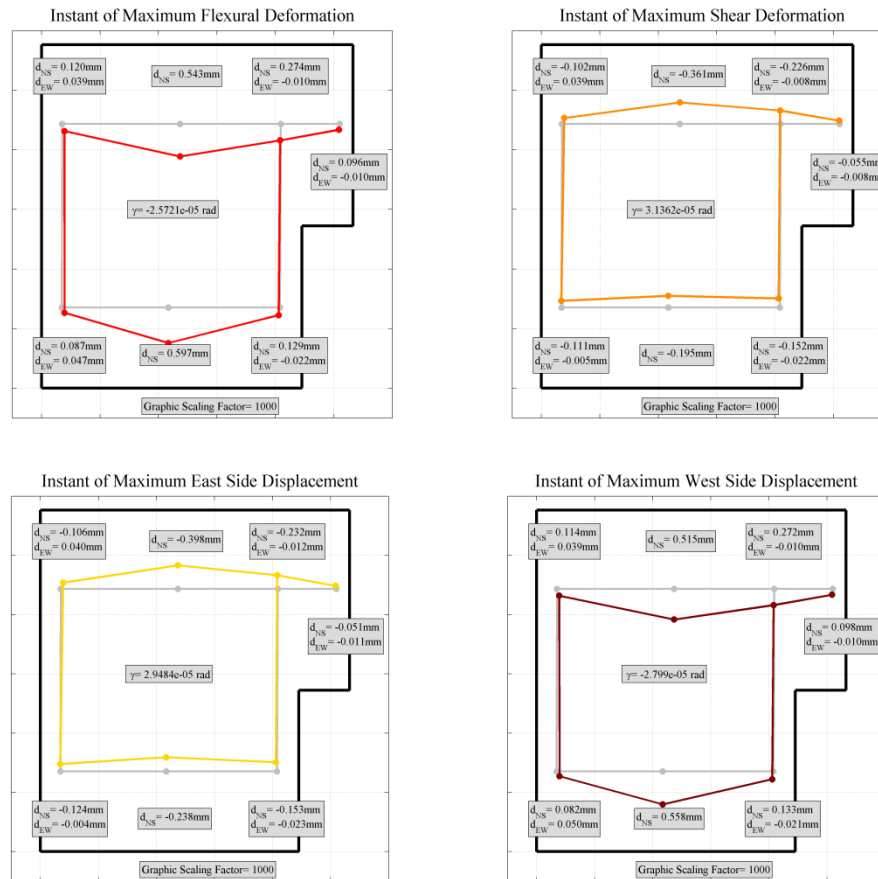


Figure 274. Test #9_150%_SC1_141: Floor deflected shapes at four test instants.

Figure 275 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.



Figure 275. Test #9_150%_SC1_141: Longitudinal Cross Section Deformed Shape

5.4.6 Test #13_50%_SC2_076

Figure 276 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

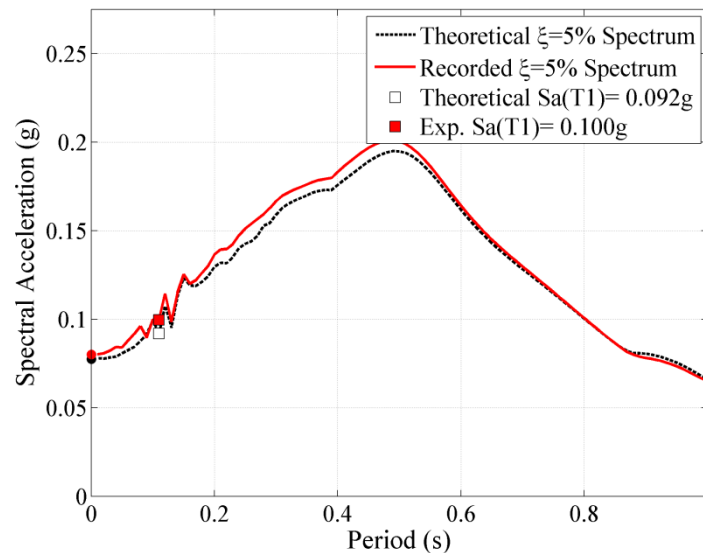


Figure 276. Test #13_50%_SC2_076: Theoretical-Experimental Spectral Comparison.

Figure 277 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

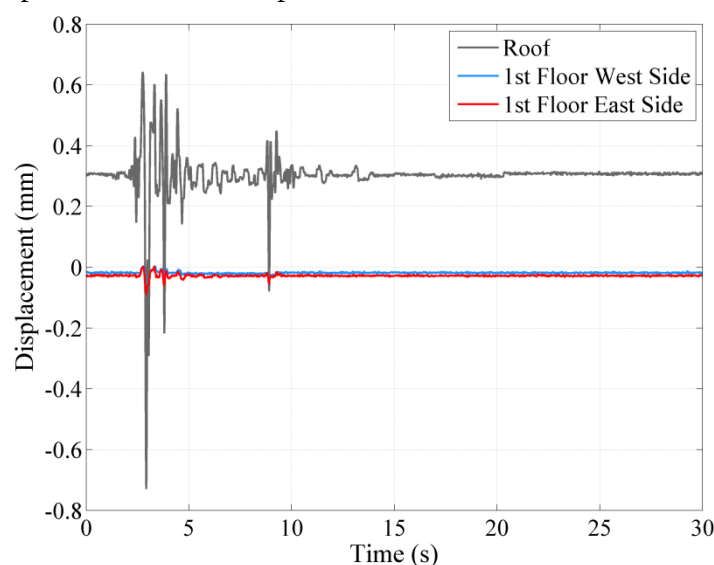


Figure 277. Test #13_50%_SC2_076: Displacement Histories.

Figure 278a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 278b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

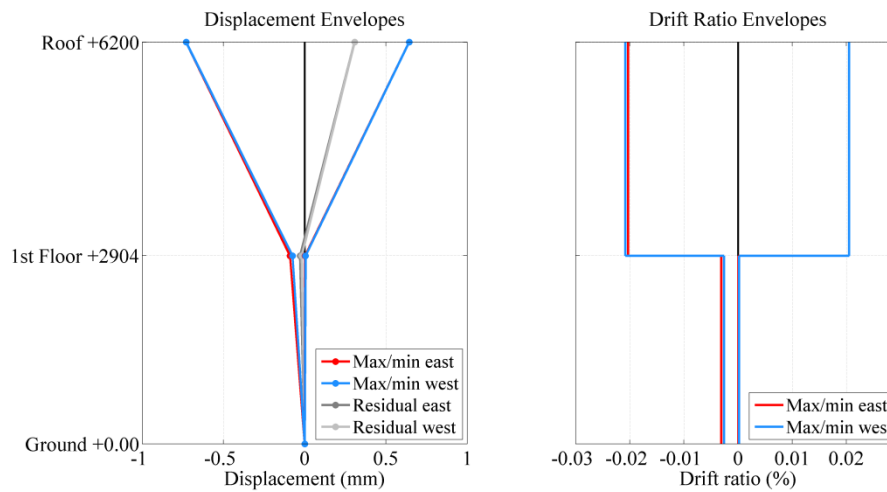


Figure 278. Test #13_50%_SC2_076: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 279 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

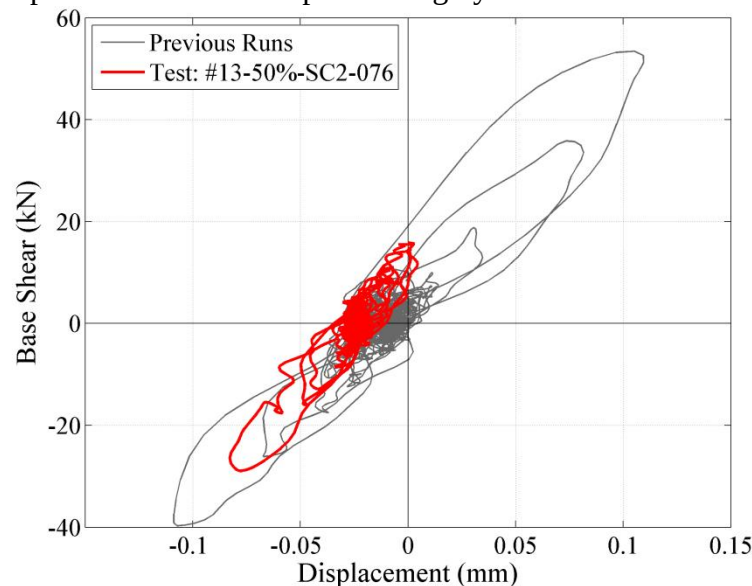


Figure 279. Test #13_50%_SC2_076: Base Shear vs 1st Floor Average Displacement.

Figure 280 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

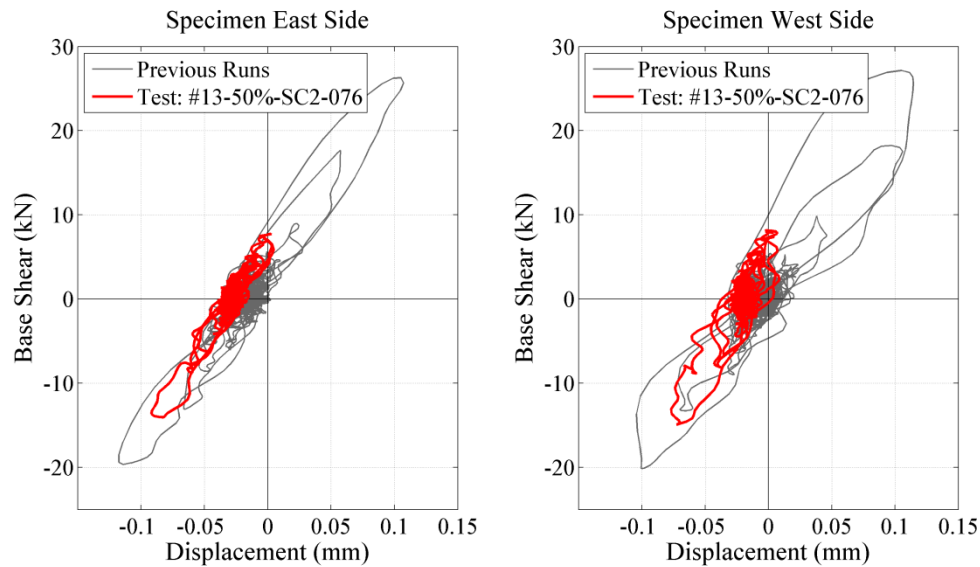


Figure 280. Test #13_50%_SC2_076: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 281 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

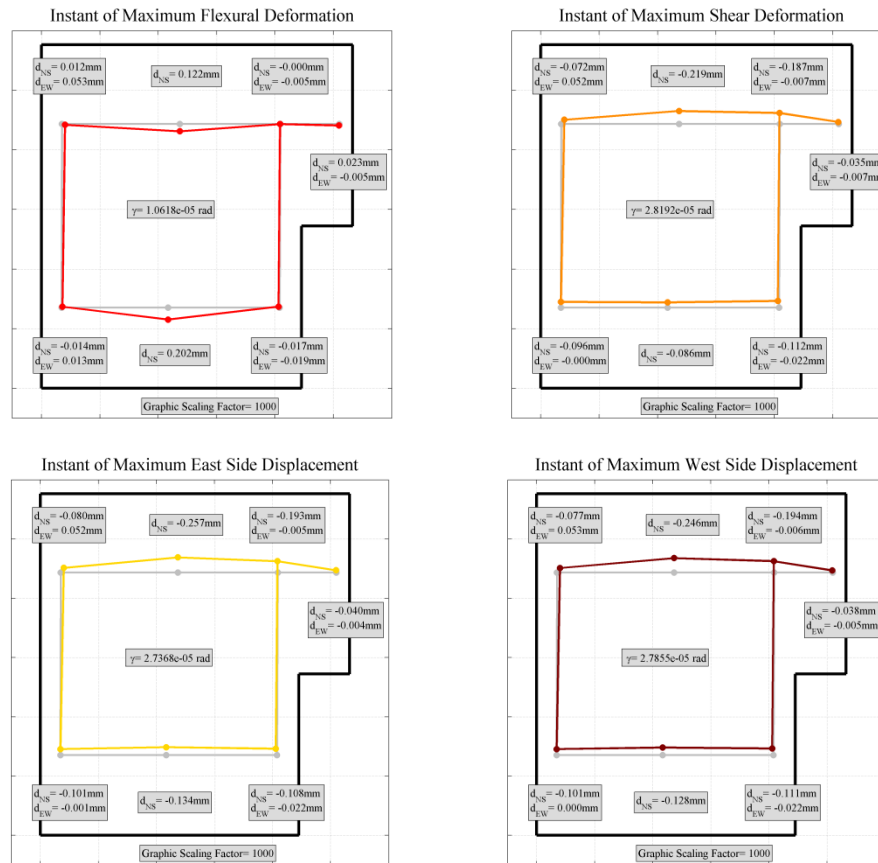


Figure 281. Test #13_50%_SC2_076: Floor deflected shapes at four test instants.

Figure 282 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

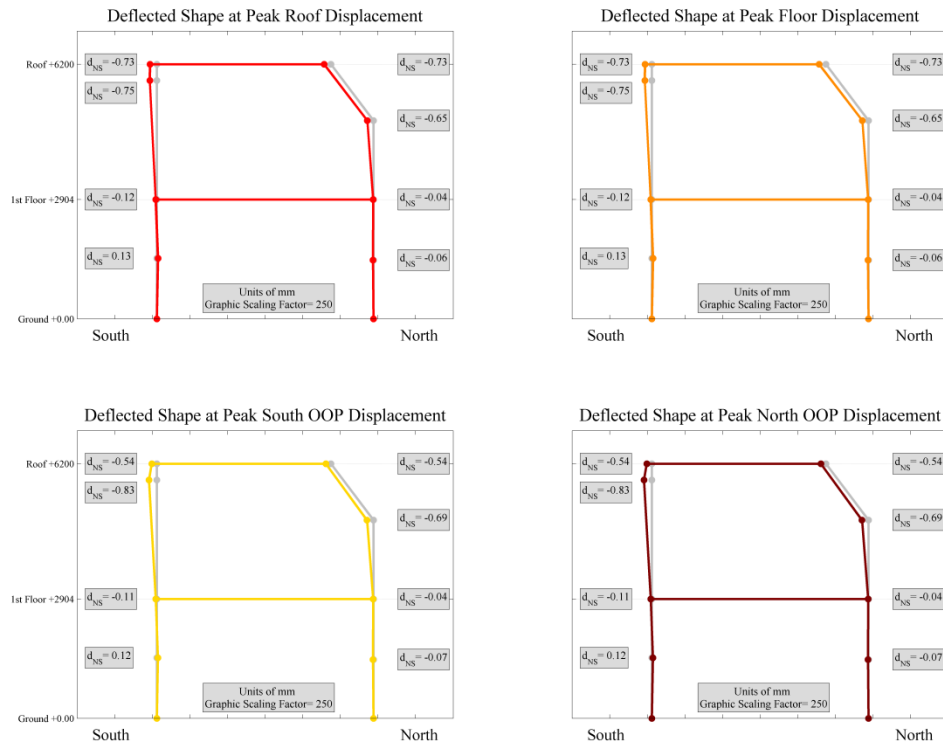


Figure 282. Test #13_50%_SC2_076: Longitudinal Cross Section Deformed Shape

5.4.7 Test #15_100%_SC2_152

Figure 283 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

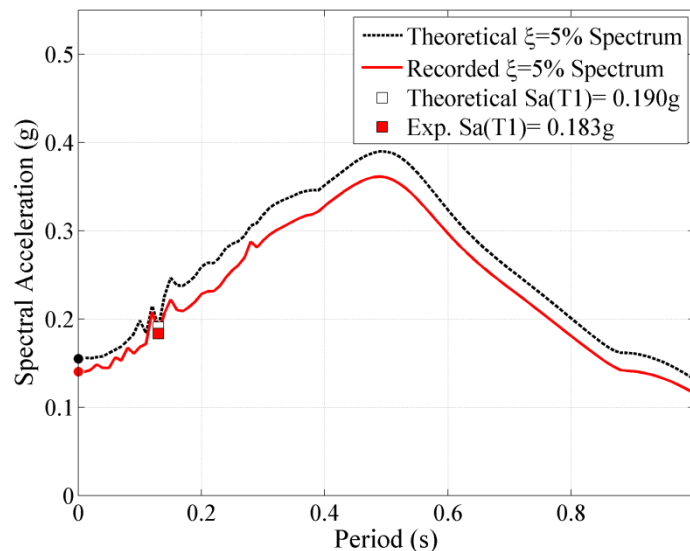


Figure 283. Test #15_100%_SC2_152: Theoretical-Experimental Spectral Comparison.

Figure 284 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

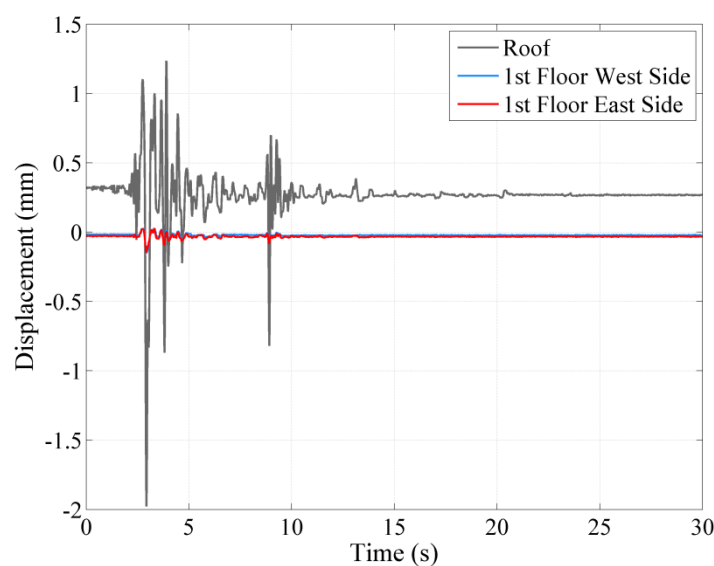


Figure 284. Test #15_100%_SC2_152: Displacement Histories.

Figure 285a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 285b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

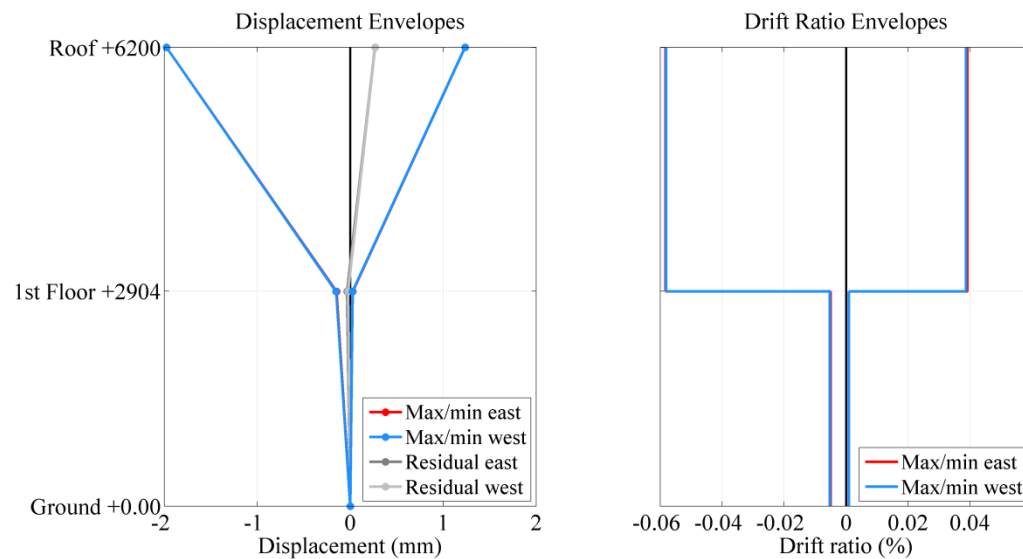


Figure 285. Test #15_100%_SC2_152: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 286 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

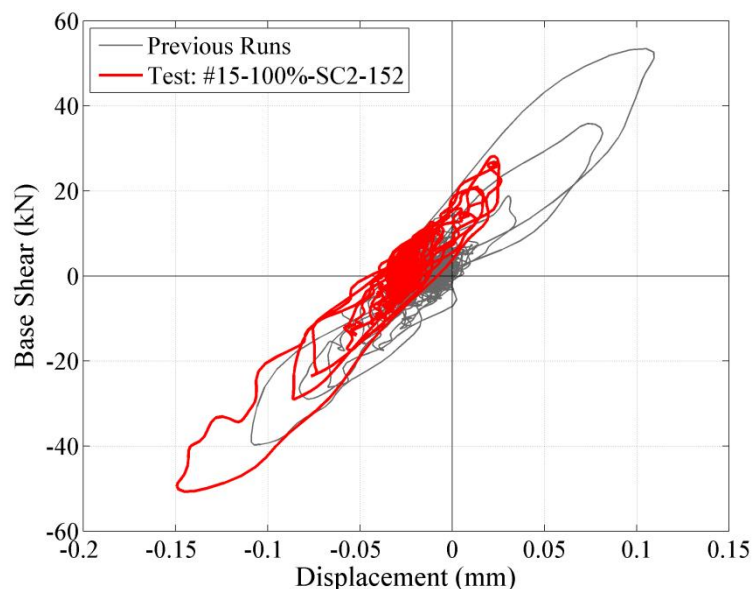


Figure 286. Test #15_100%_SC2_152: Base Shear vs 1st Floor Average Displacement.

Figure 287 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

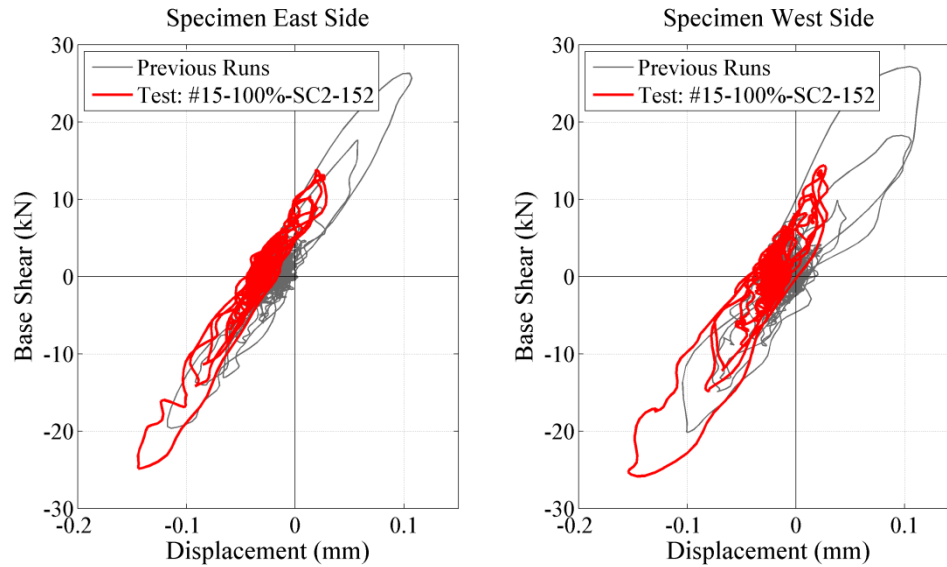


Figure 287. Test #15_100%_SC2_152: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 288 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

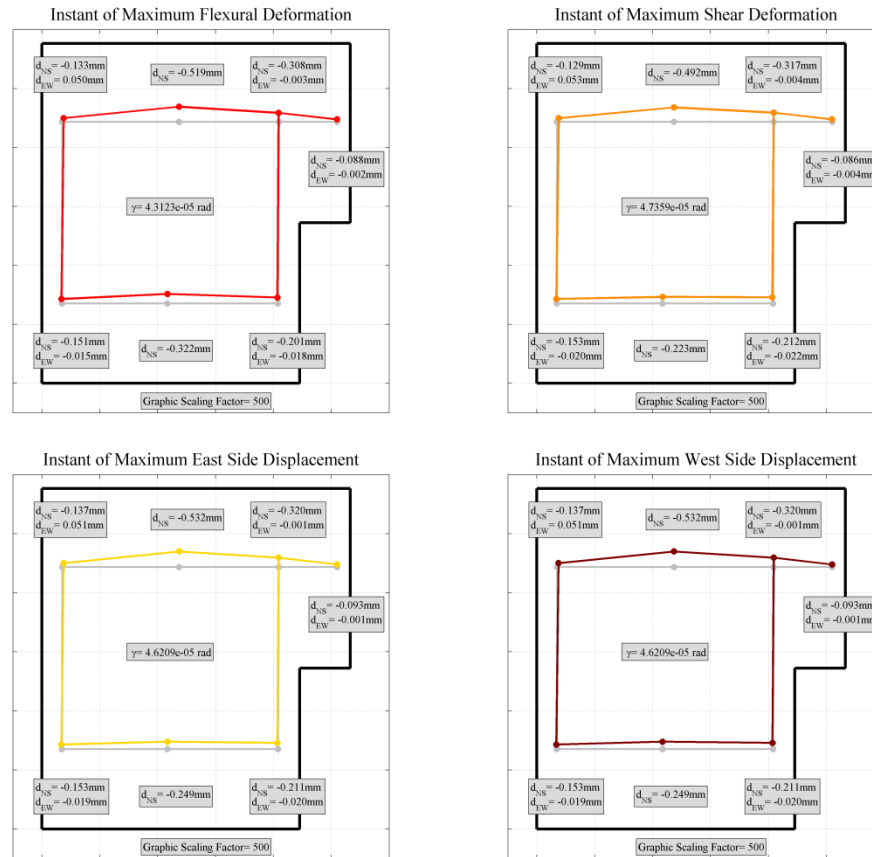


Figure 288. Test #15_100%_SC2_152: Floor deflected shapes at four test instants.

Figure 289 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

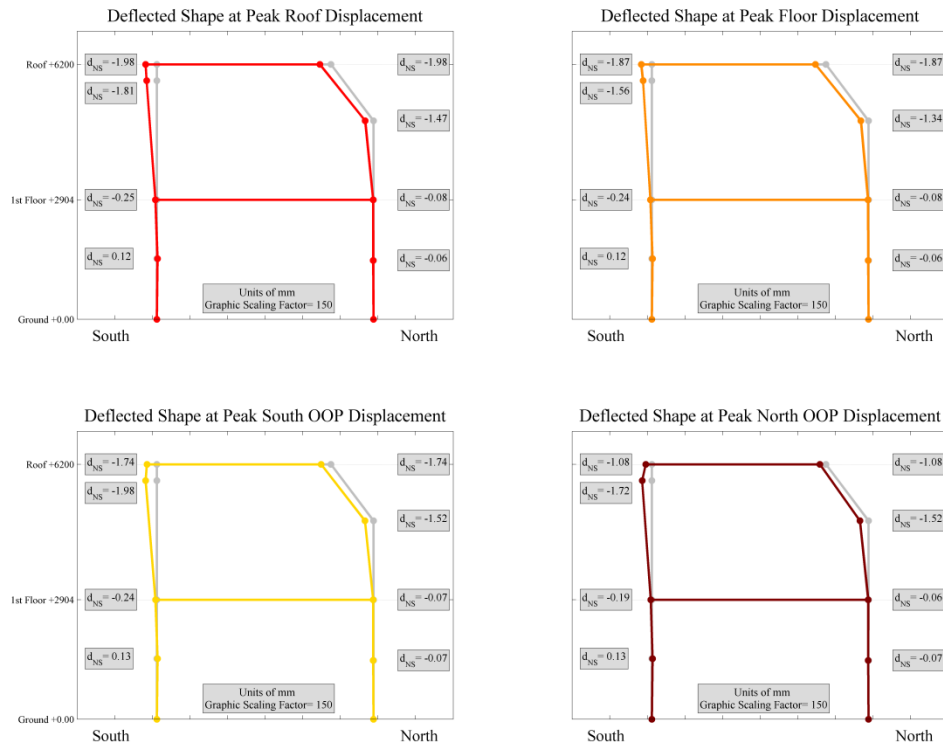


Figure 289. Test #15_100%_SC2_152: Longitudinal Cross Section Deformed Shape

5.4.8 Test #20_150%_SC2_228

Figure 290 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

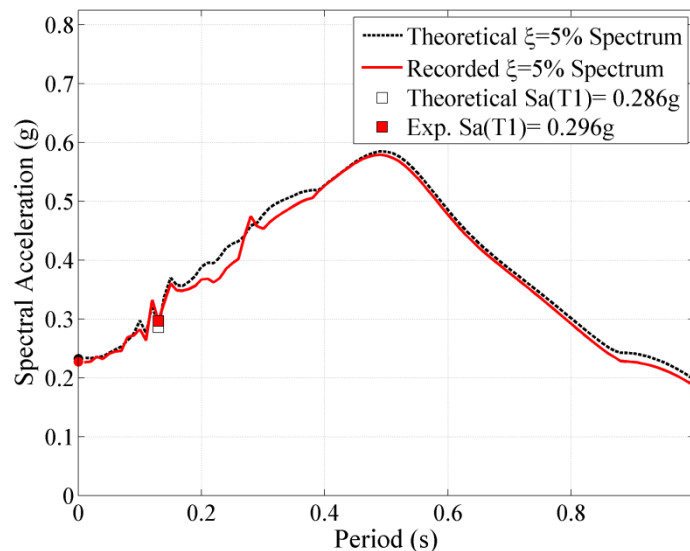


Figure 290. Test #20_150%_SC2_228: Theoretical-Experimental Spectral Comparison.

Figure 291 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

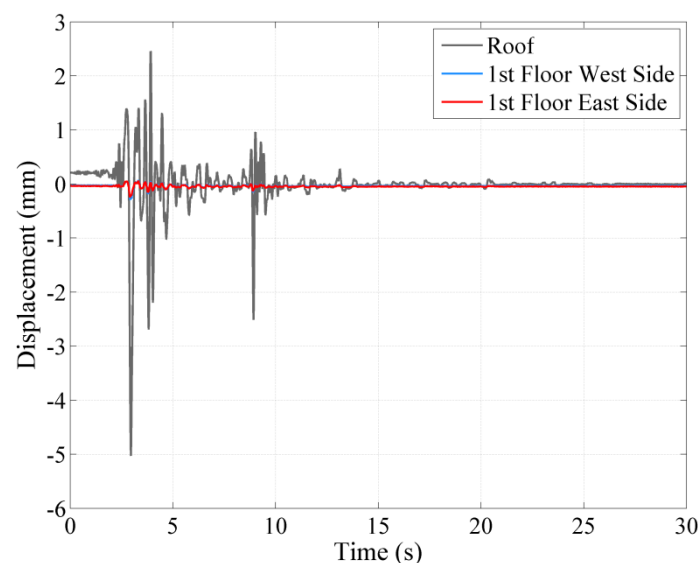


Figure 291. Test #20_150%_SC2_228: Displacement Histories.

Figure 292a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 292b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

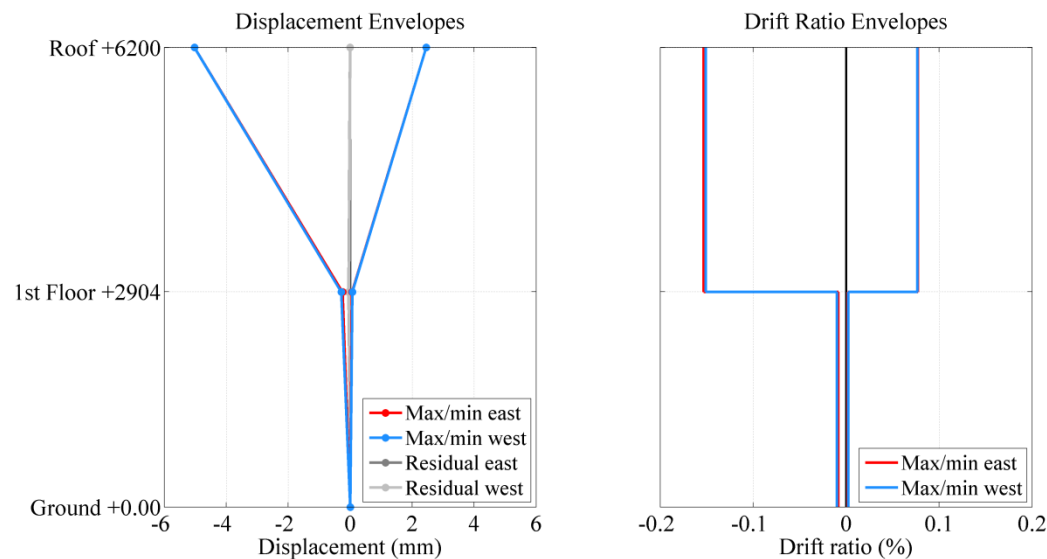


Figure 292. Test #20_150%_SC2_228: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 293 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

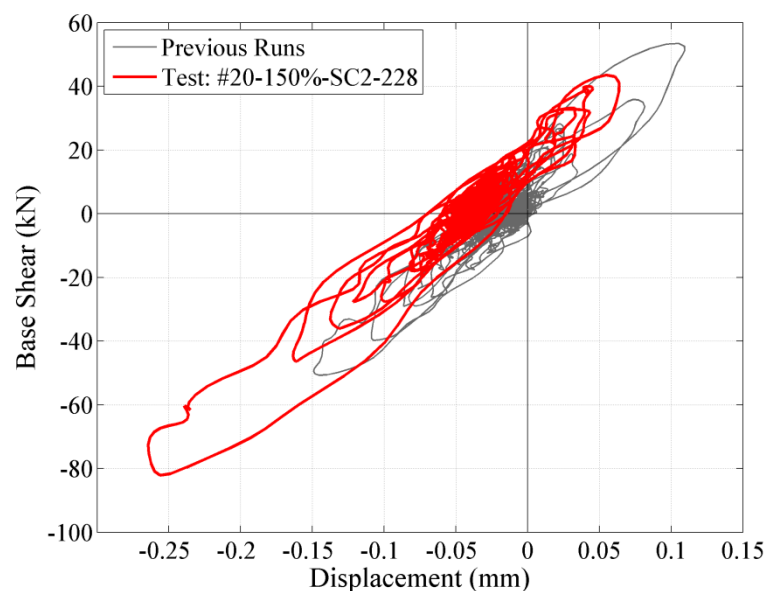


Figure 293. Test #20_150%_SC2_228: Base Shear vs 1st Floor Average Displacement.

Figure 294 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

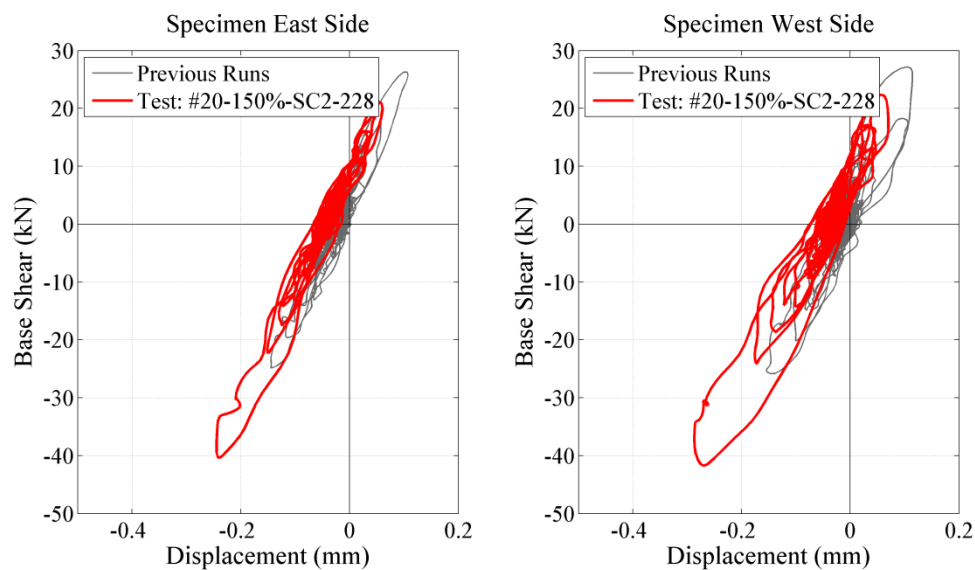


Figure 294. Test #20_150%_SC2_228: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 295 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

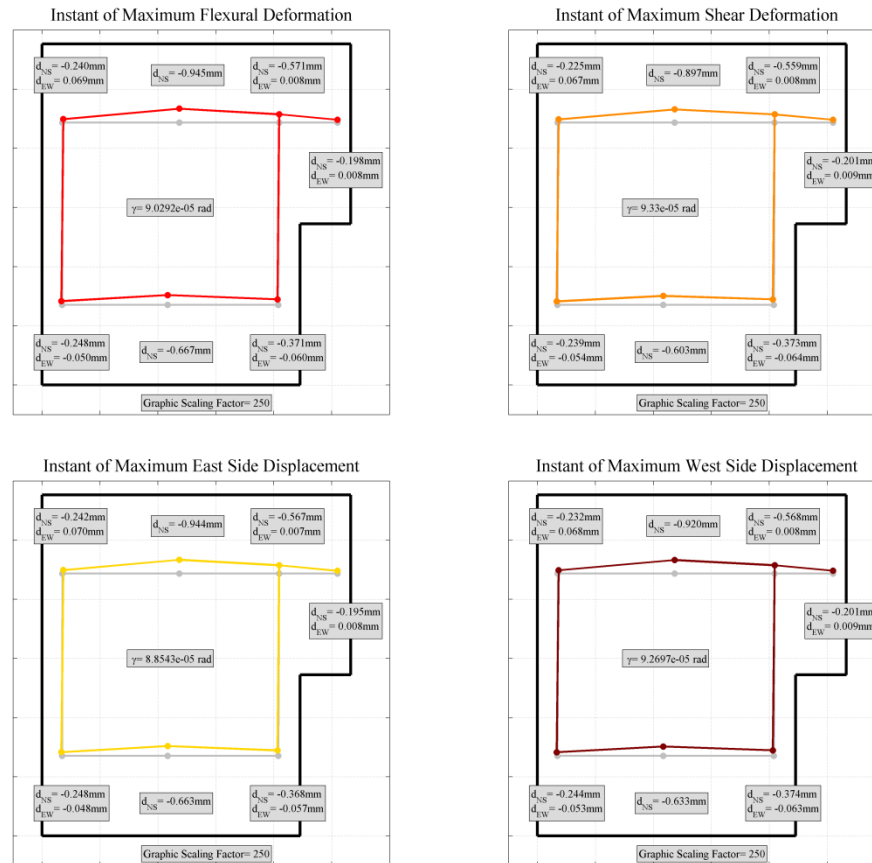


Figure 295. Test #20_150%_SC2_228: Floor deflected shapes at four test instants.

Figure 296 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

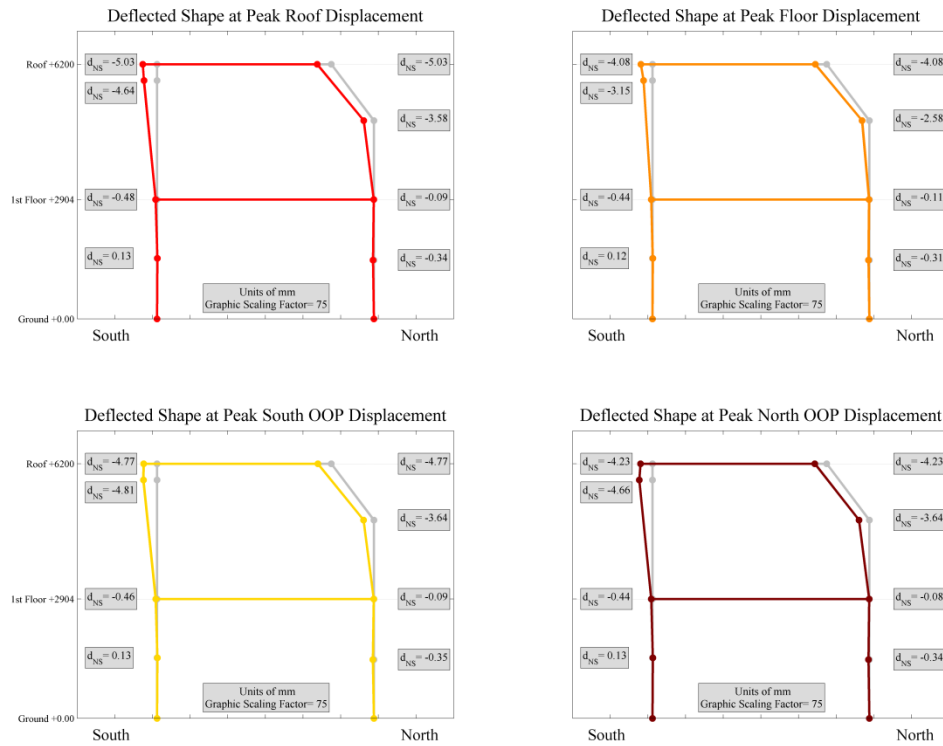


Figure 296. Test #20_150%_SC2_228: Longitudinal Cross Section Deformed Shape

5.4.9 Test #22_200%_SC2_304

Figure 297 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

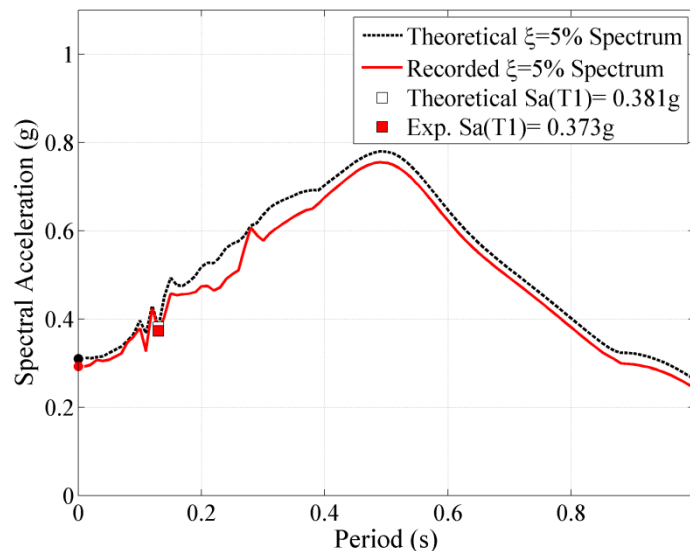


Figure 297. Test #22_200%_SC2_304: Theoretical-Experimental Spectral Comparison.

Figure 298 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

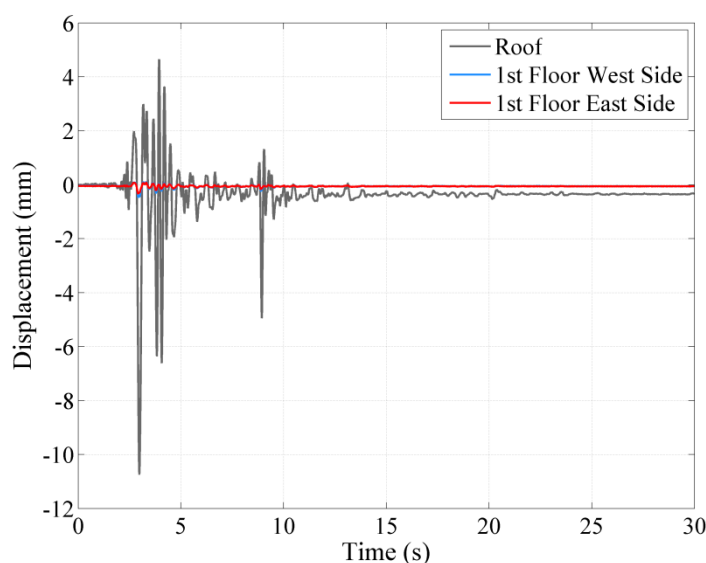


Figure 298. Test #22_200%_SC2_304: Displacement Histories.

Figure 299a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 299b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

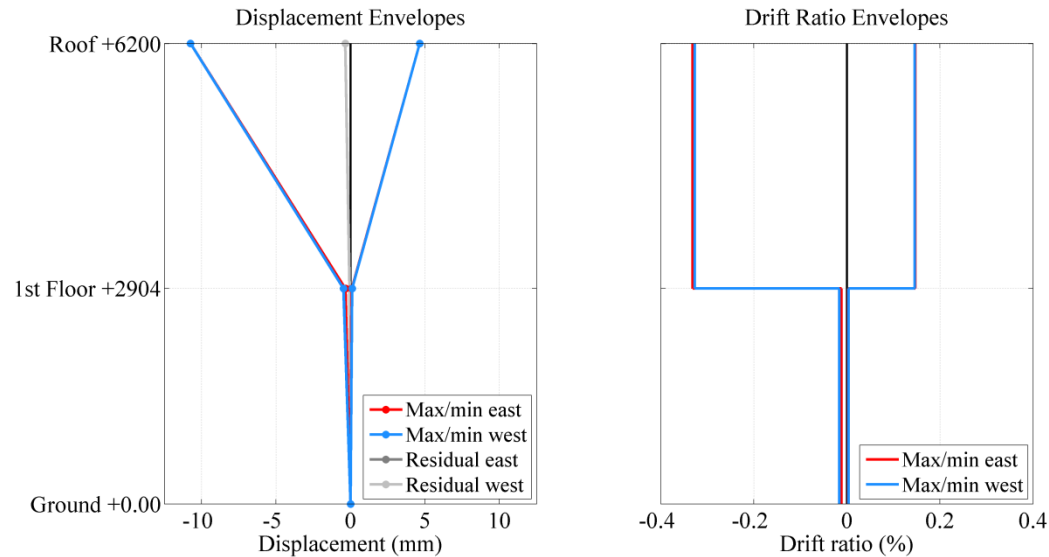


Figure 299. Test #22_200%_SC2_304: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 300 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

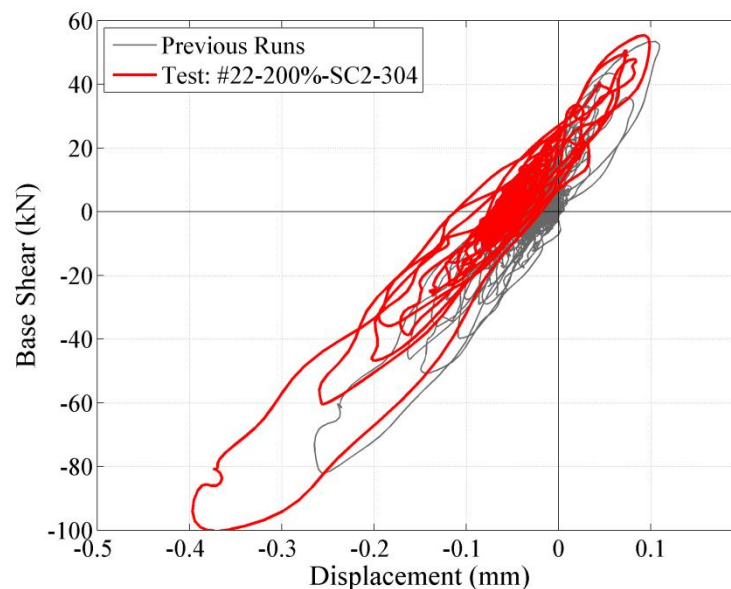


Figure 300. Test #22_200%_SC2_304: Base Shear vs 1st Floor Average Displacement.

Figure 301 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

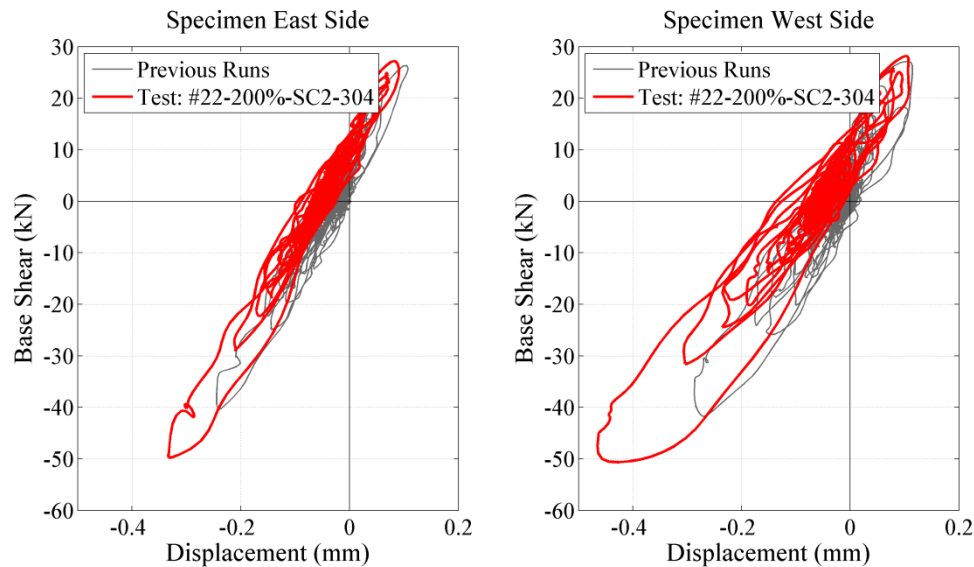


Figure 301. Test #22_200%_SC2_304: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 302 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

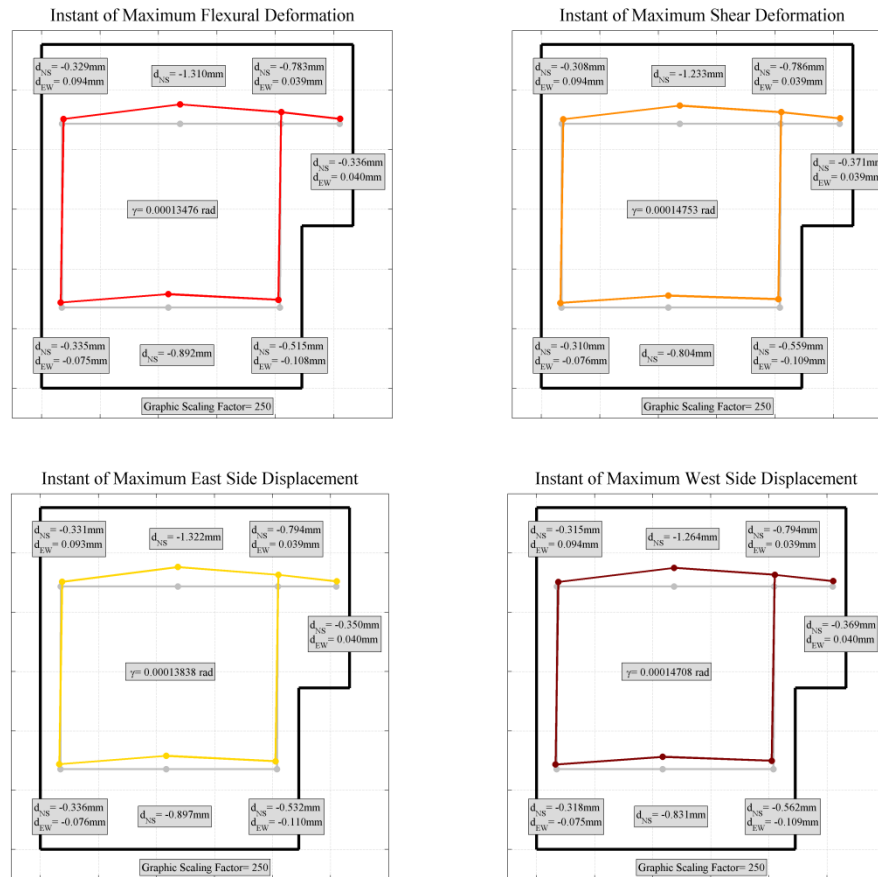


Figure 302. Test #22_200%_SC2_304: Floor deflected shapes at four test instants.

Figure 303 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

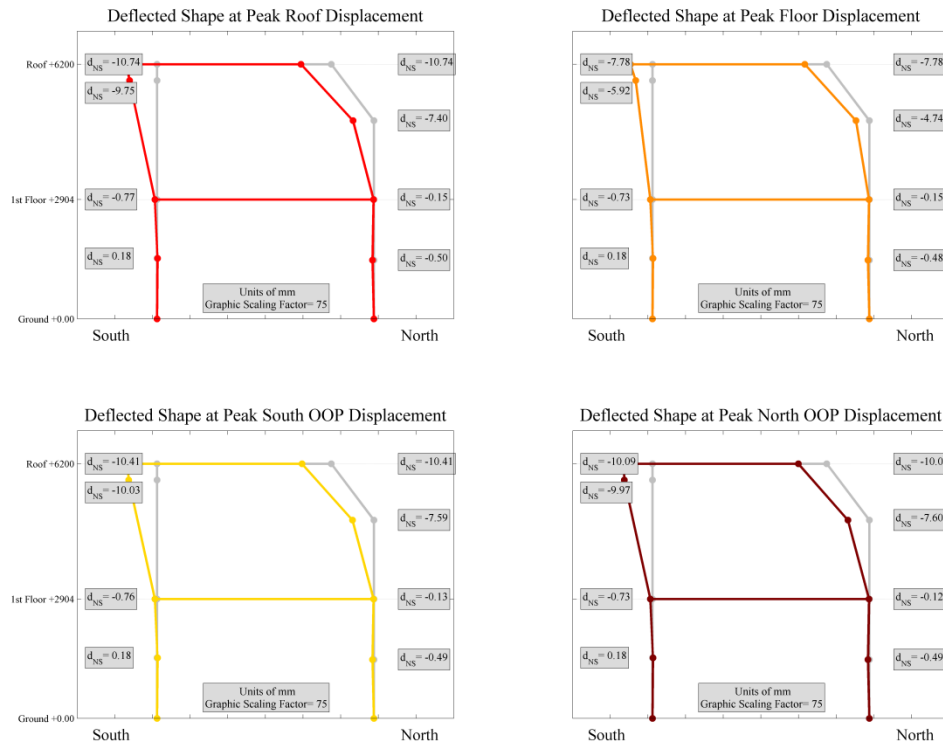


Figure 303. Test #22_200%_SC2_304: Longitudinal Cross Section Deformed Shape

5.4.10 Test #24_250%_SC2_380

Figure 304 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

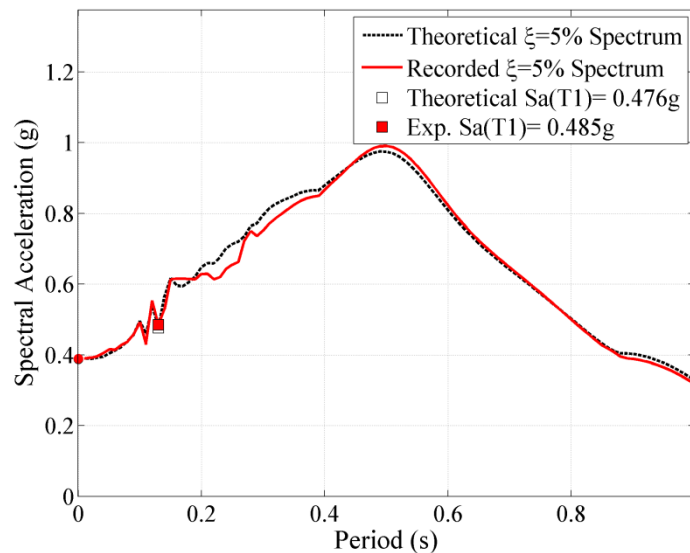


Figure 304. Test #24_250%_SC2_380: Theoretical-Experimental Spectral Comparison.

Figure 305 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

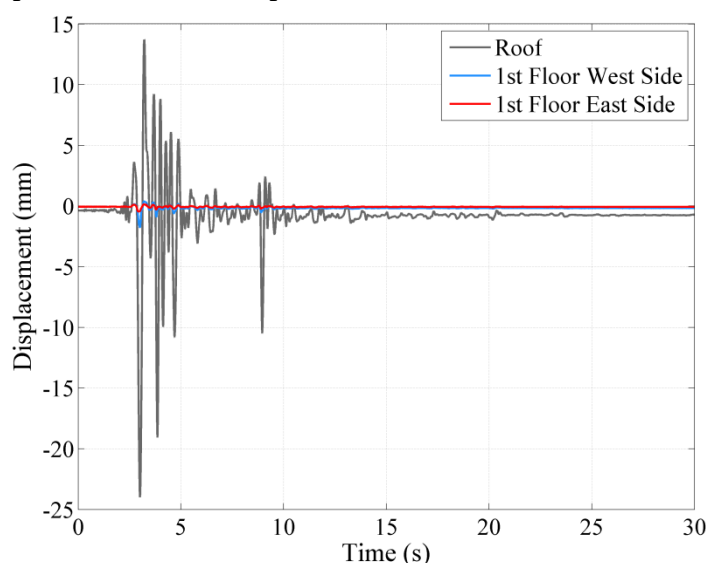


Figure 305. Test #24_250%_SC2_380: Displacement Histories.

Figure 306a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 306b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

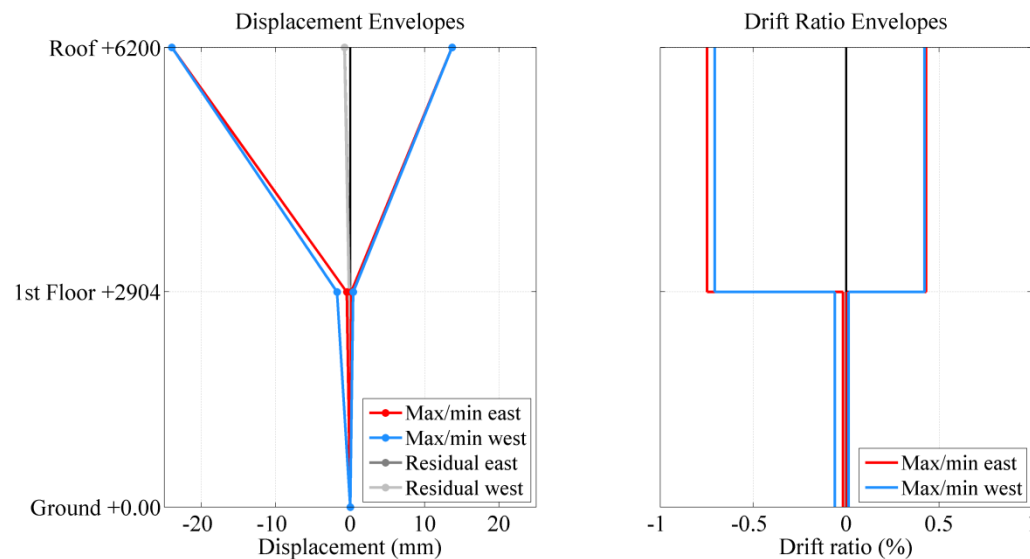


Figure 306. Test #24_250%_SC2_380: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 307 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

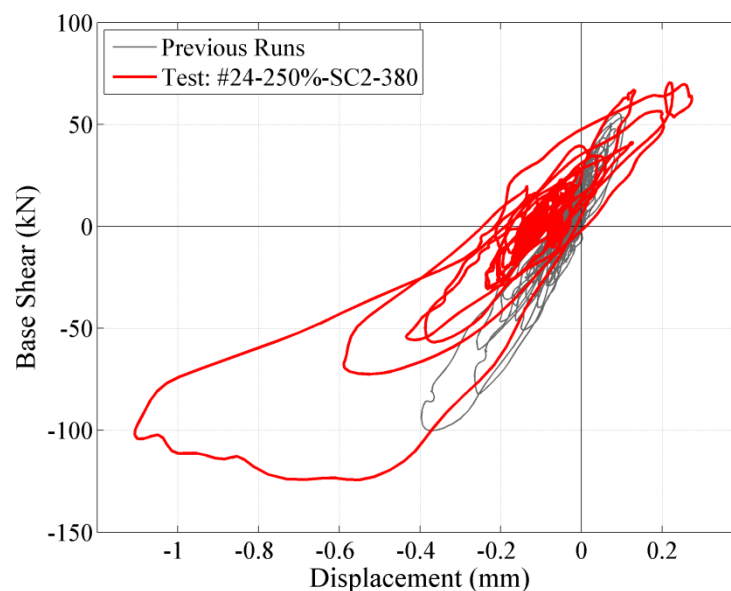


Figure 307. Test #24_250%_SC2_380: Base Shear vs 1st Floor Average Displacement.

Figure 308 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

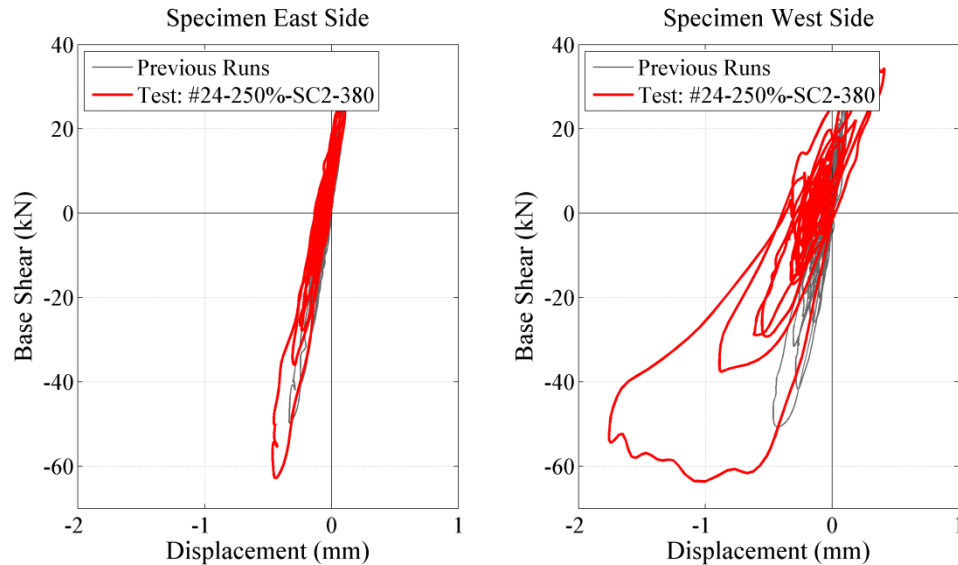


Figure 308. Test ##24_250%_SC2_380: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 309 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

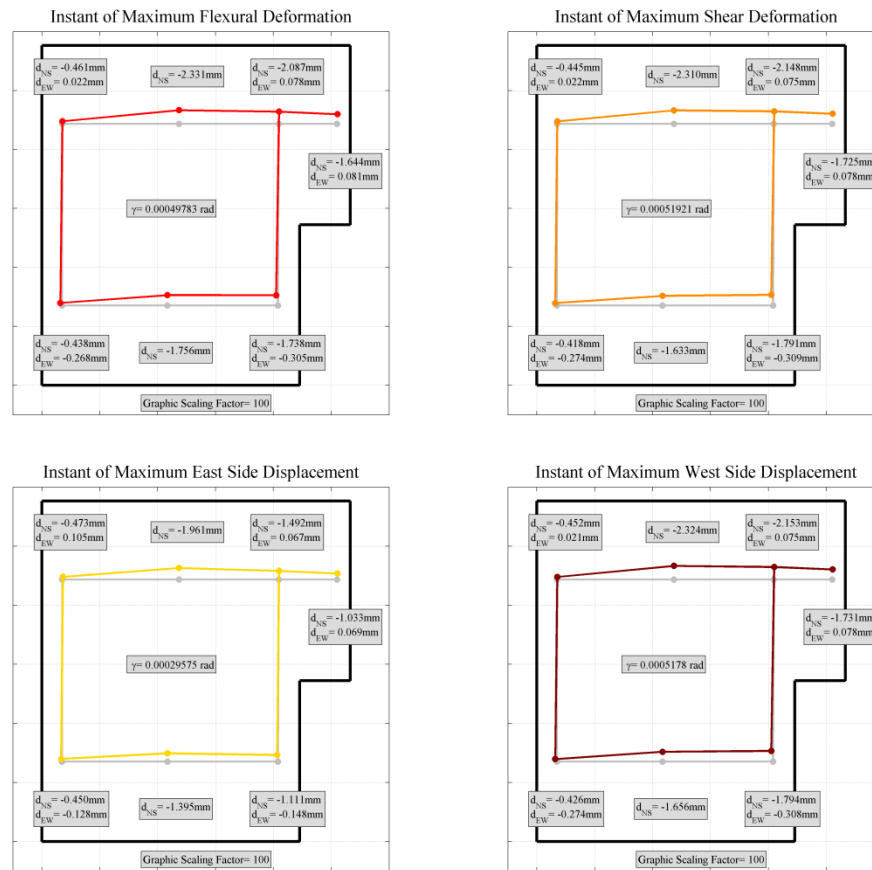


Figure 309. Test #24_250%_SC2_380: Floor deflected shapes at four test instants.

Figure 310 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

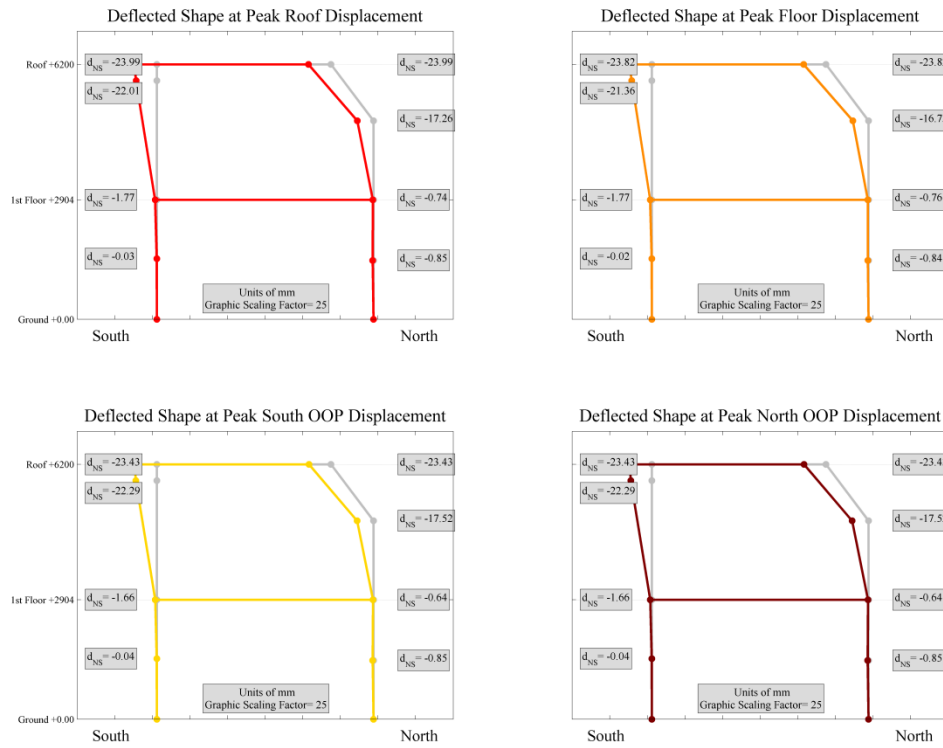


Figure 310. Test #24_250%_SC2_380: Longitudinal Cross Section Deformed Shape

5.4.11 Test #30_300%_SC2_456

Figure 311 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

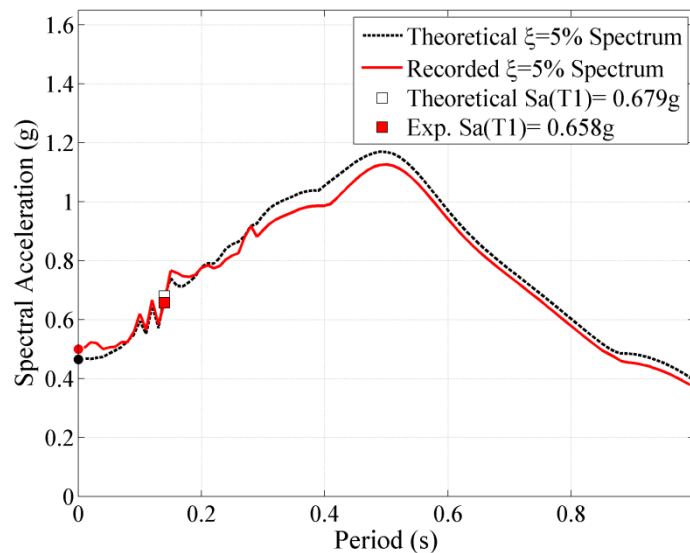


Figure 311. Test #30_300%_SC2_456: Theoretical-Experimental Spectral Comparison.

Figure 312 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

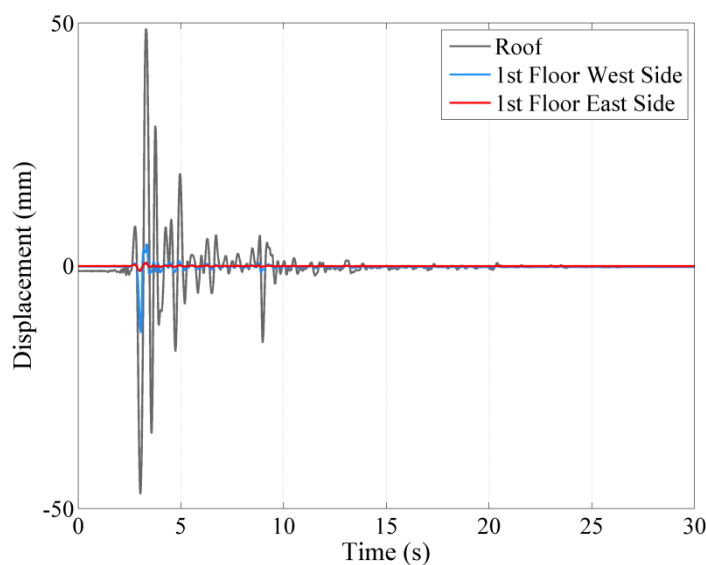


Figure 312. Test #30_300%_SC2_456: Displacement Histories.

Figure 313a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 313b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

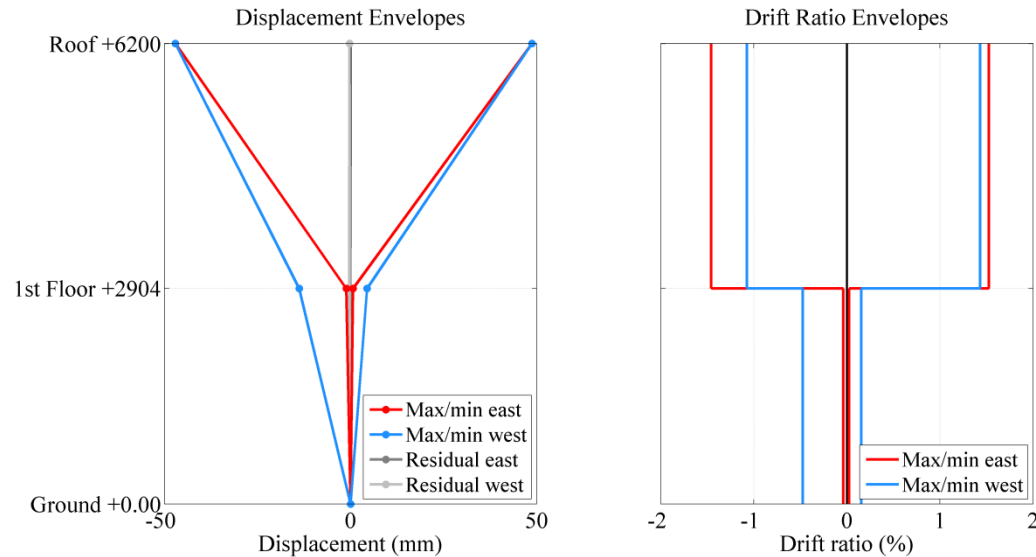


Figure 313. Test #30_300%_SC2_456: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 314 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

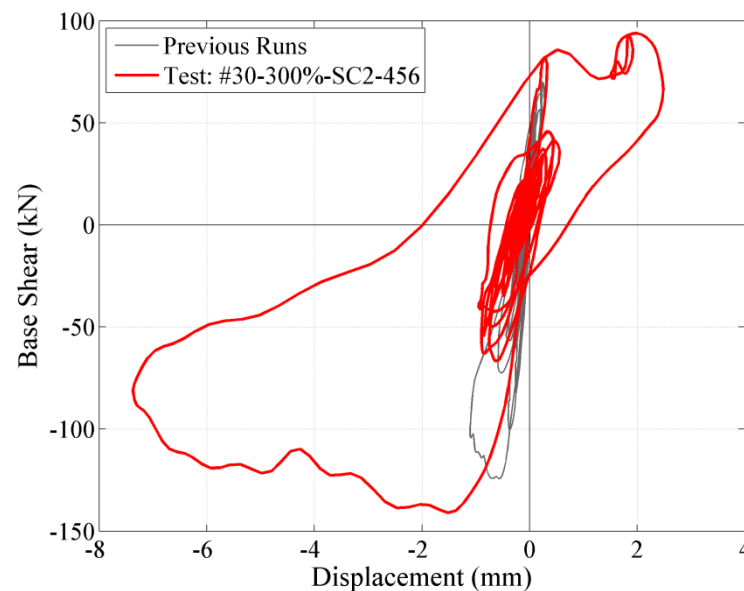


Figure 314. Test #30_300%_SC2_456: Base Shear vs 1st Floor Average Displacement.

Figure 315 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

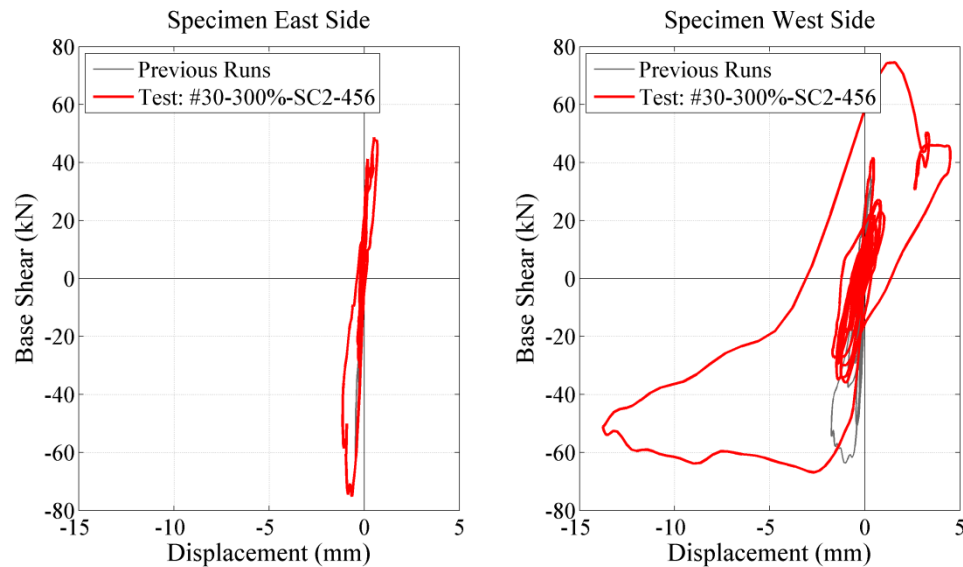


Figure 315. Test #30_300%_SC2_456: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 316 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

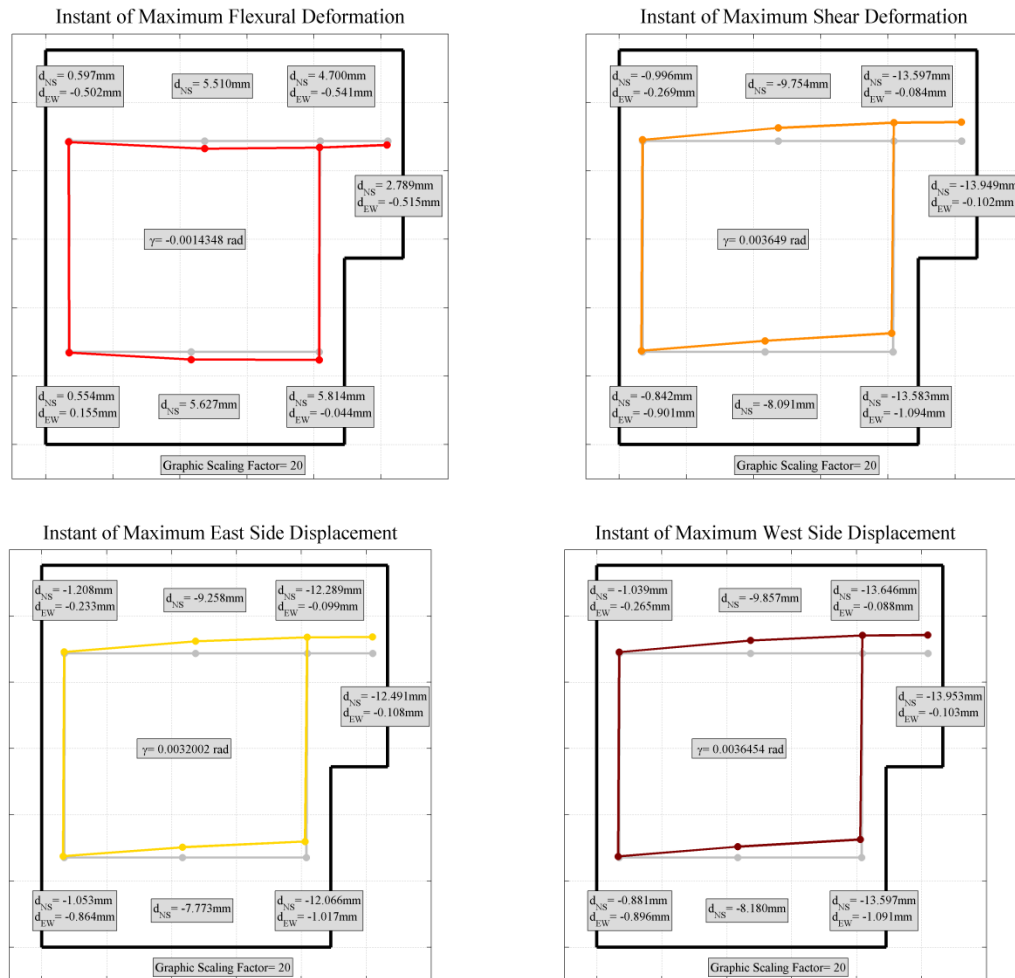


Figure 316. Test #30_300%_SC2_456: Floor deflected shapes at four test instants.

Figure 317 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

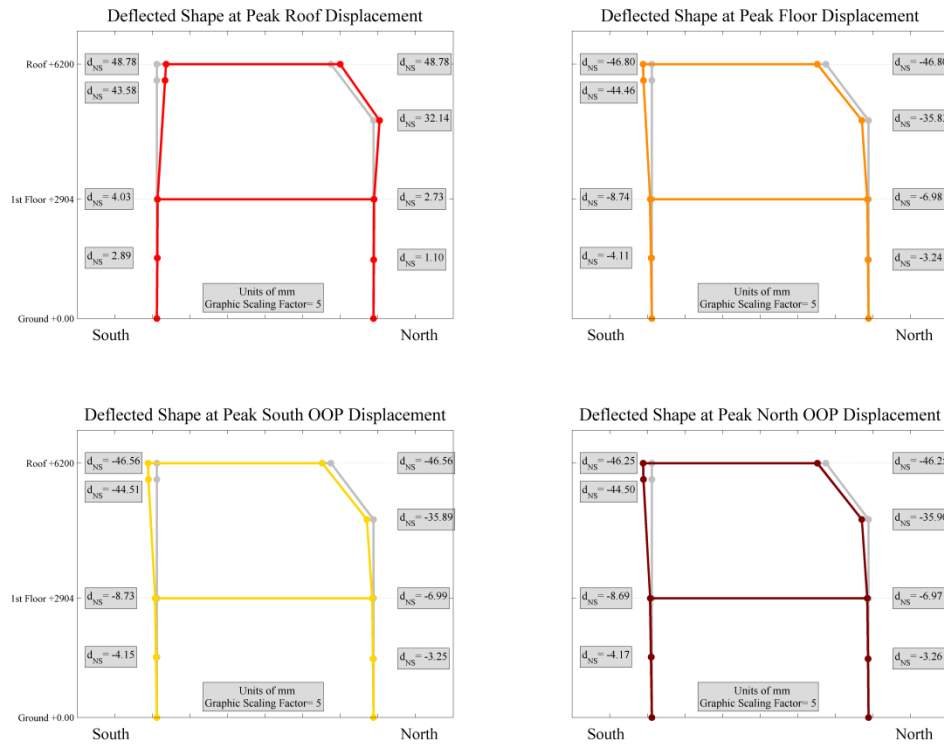


Figure 317. Test #30_300%_SC2_456: Longitudinal Cross Section Deformed Shape.

5.4.12 Test #36_400%_SC2_608

Figure 318 shows the comparison between theoretical and experimental acceleration response spectra; the experimental acceleration time history used for the spectrum comparison is the average of the recorded accelerations on the specimen foundation (accelerometers #24 and #25). Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

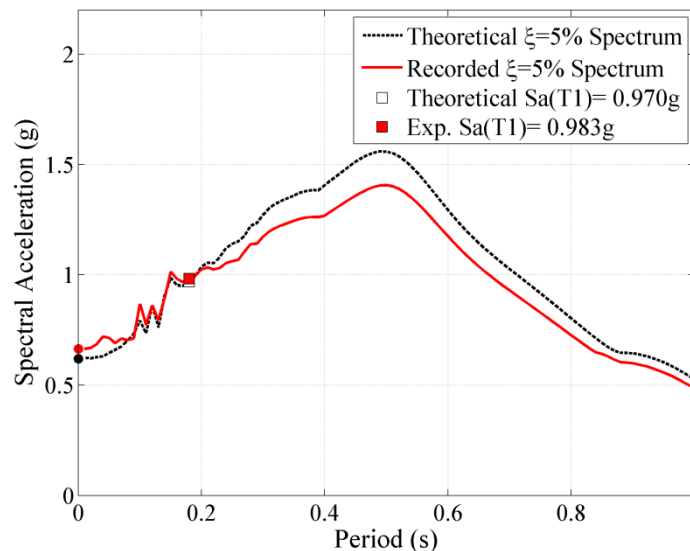


Figure 318. Test #36_400%_SC2_608: Theoretical-Experimental Spectral Comparison.

Figure 319 shows the displacement time histories of the first floor (East and West sides) and of the roof level. The first floor East displacement history has been obtained as the average of the two potentiometers #14 and #16, while the West displacement history has been obtained as the average of the two potentiometers #10 and #19. In all graphs the residual displacement from the previous test is taken into account.

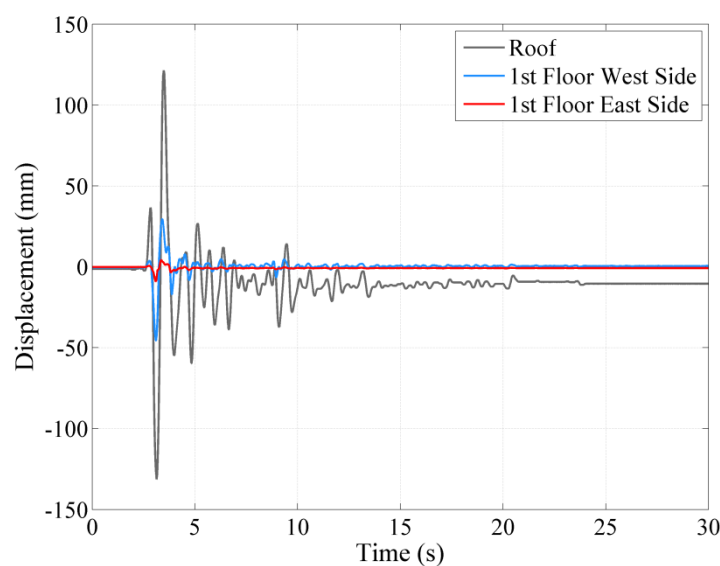


Figure 319. Test #36_400%_SC2_608: Displacement Histories.

Figure 320a shows the positive and negative displacement envelopes recorded at the first floor (East and West sides) and at the roof level. The residual displacements at the end of the test are also shown in grey. Figure 320b plots the drift-ratio envelopes. The roof drift ratio is computed considering the inclined length of the rafter (315 cm).

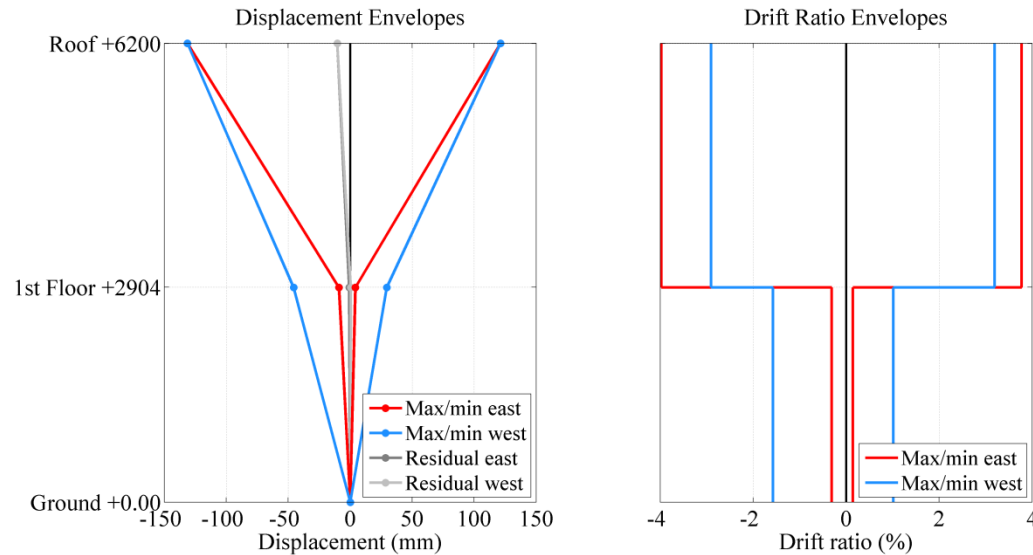


Figure 320. Test #36_400%_SC2_608: Envelope Deformed Shape (a) and Envelope drift ratios (b).

Figure 321 shows the hysteretic response in terms of base shear vs first-floor average displacement. The base shear has been obtained by summing the product of each acceleration history times the tributary mass of the accelerometer. The hysteretic response from previous tests is also plotted in grey.

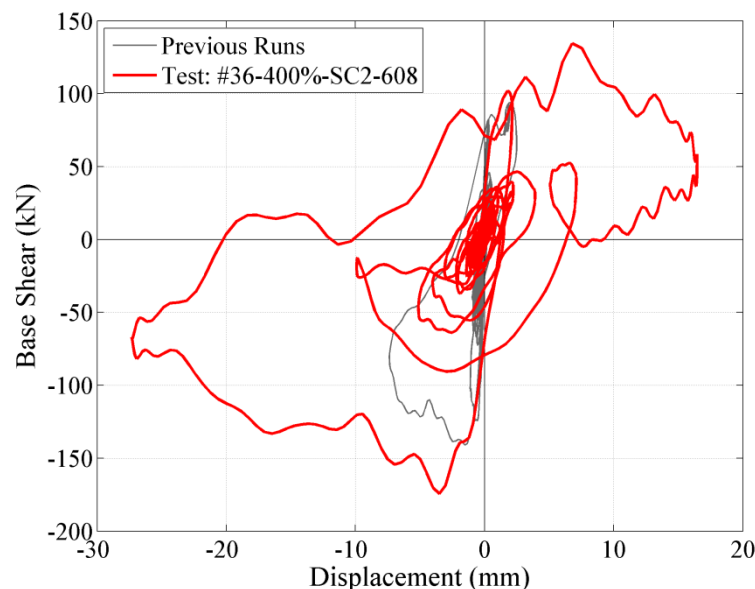


Figure 321. Test #36_400%_SC2_608: Base Shear vs 1st Floor Average Displacement.

Figure 322 shows the hysteretic response of each longitudinal wall (East and West sides) in terms of tributary base shear vs first-floor displacement of that side. The hysteretic response from previous tests is also plotted in grey.

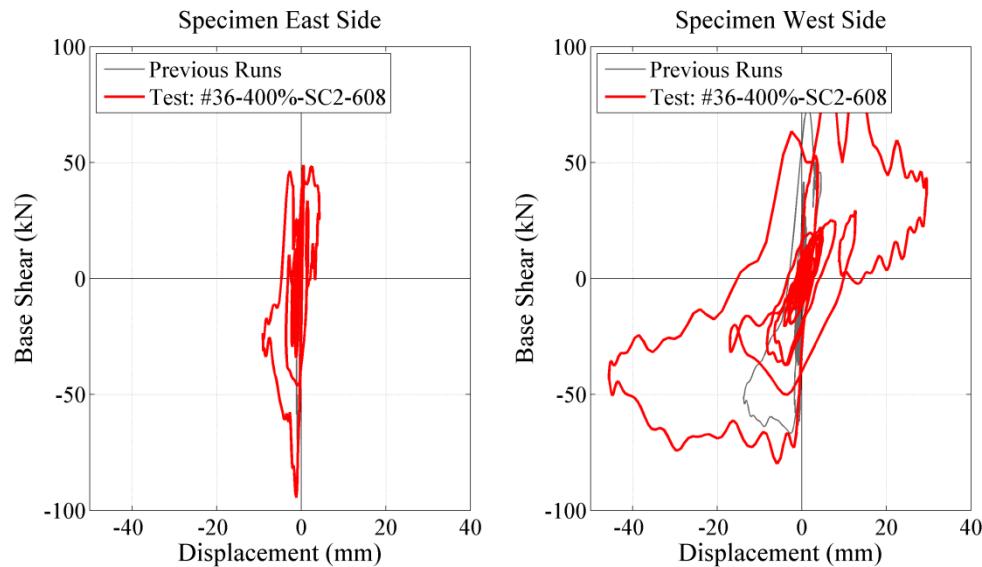


Figure 322. Test #36_400%_SC2_608: East and West Base Shear vs East and West 1st Floor Displacement.

Figure 323 presents the floor deflected shape and the associated shear deformation at four instants during the test:

- peak flexural deformation;
- peak shear deformation;
- peak East-side longitudinal displacement;
- peak West-side longitudinal displacement;

The floor deflected shape is magnified by the scaling factor indicated on the figure.

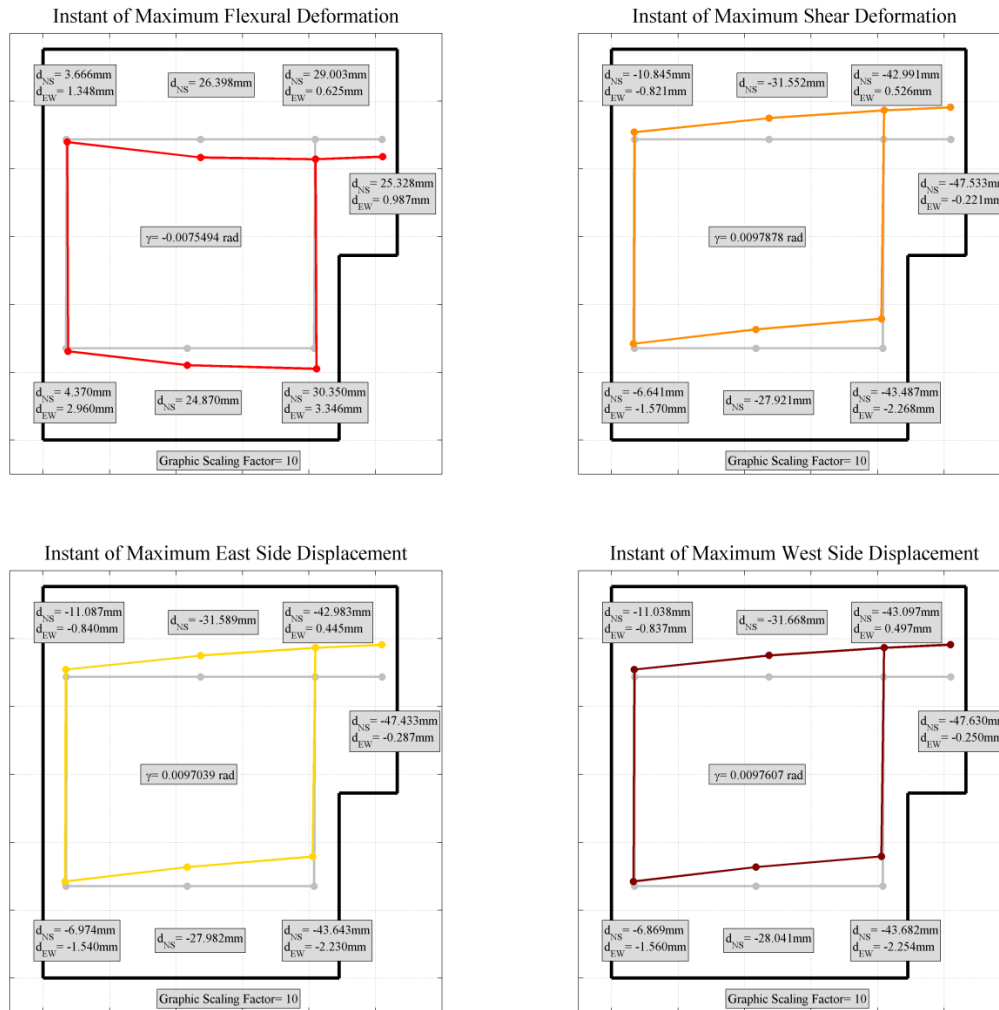


Figure 323. Test #36_400%_SC2_608: Floor deflected shapes at four test instants.

Figure 324 presents the specimen wall deflected shape by considering the horizontal displacements of the longitudinal cross-section of the building at four instants during the test:

- peak roof displacement
- peak average floor displacement
- peak out-of-plane (OOP) displacement of the South gable
- peak out-of-plane (OOP) displacement of the North gable

The deflected shape is magnified by the scaling factor indicated on the figure.

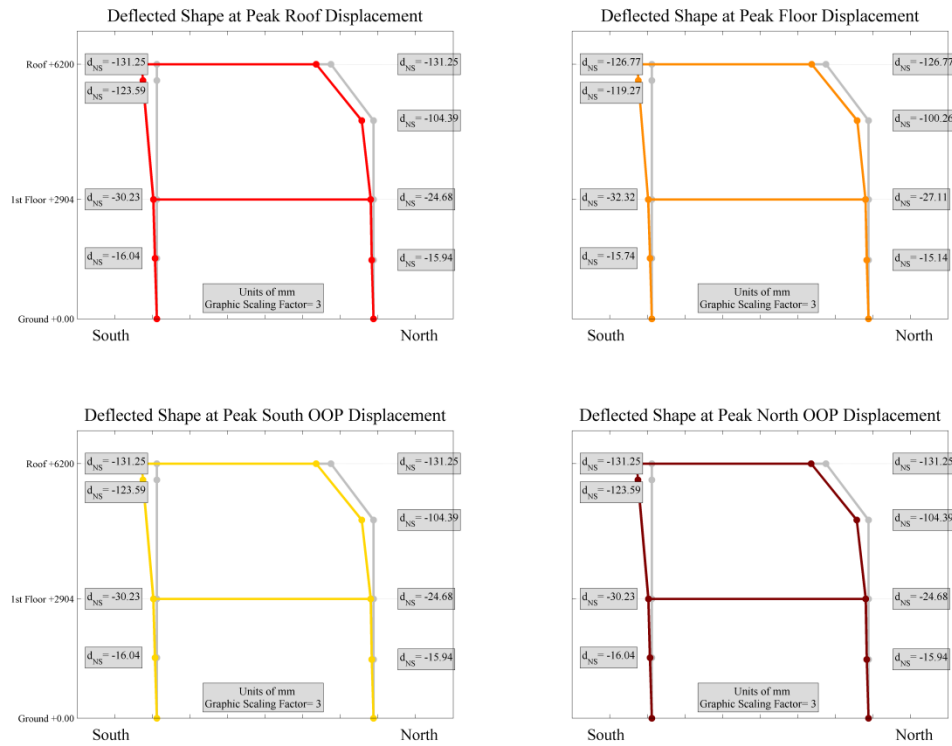


Figure 324. Test #36_400%_SC2_608: Longitudinal Cross Section Deformed Shape.

5.4.13 Test #41_200%_SC2_304

Figure 325 shows the comparison between theoretical and experimental acceleration response spectra; Notice that the theoretical and experimental spectral accelerations at the current fundamental period of vibration are very close.

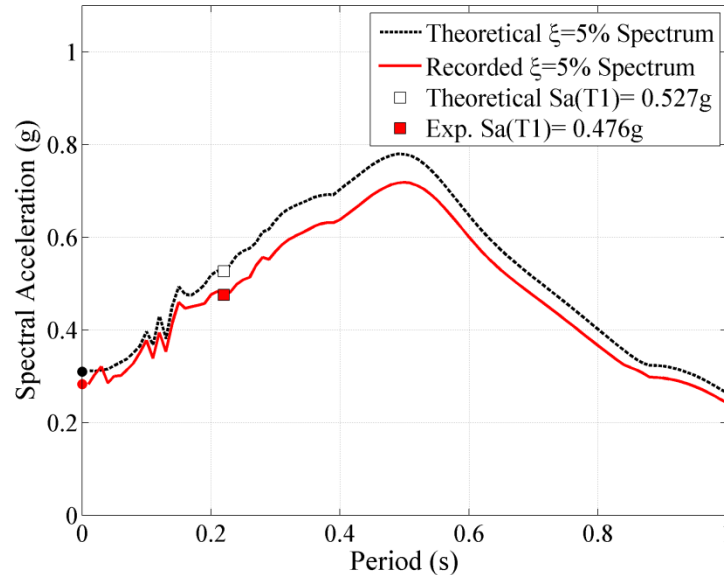


Figure 325. Test #41_200%_SC2_304: Theoretical-Experimental Spectral Comparison.

Figure 326 shows the hysteretic response in terms of base shear vs first-floor average displacement. The hysteretic response from previous tests is also plotted in grey.

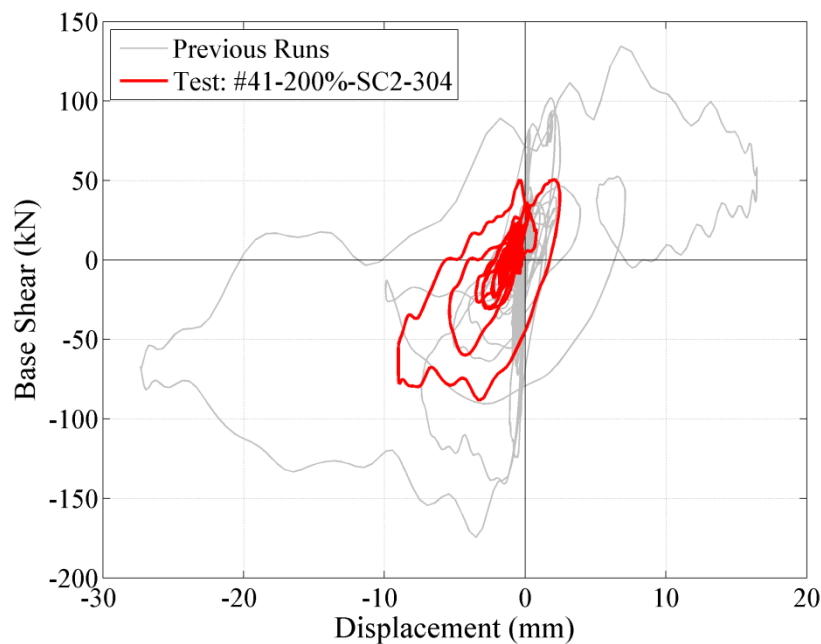


Figure 326. Test #41_200%_SC2_304: Base Shear vs 1st Floor Average Displacement.

5.4.14 Summary of the results

The present section briefly presents the results of the overall shaking table test. Figure 327 shows the peak floor and roof displacements and the roof residual displacements.

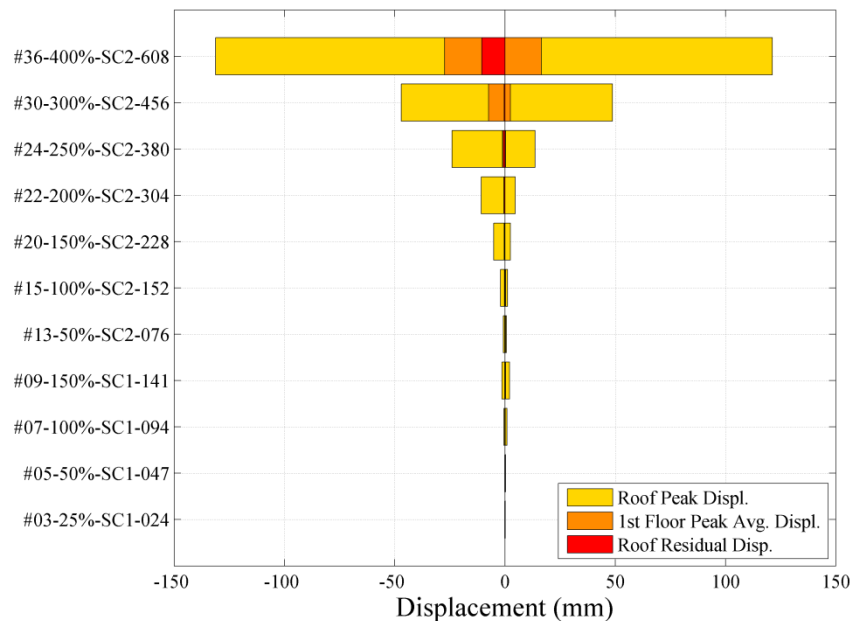


Figure 327. Experimental Displacement Response.

Figure 328 plots the specimen hysteretic response in terms of base shear vs first-floor average displacement till test #24-250%-SC2-380, where inelastic behaviour became evident in the West wall response.

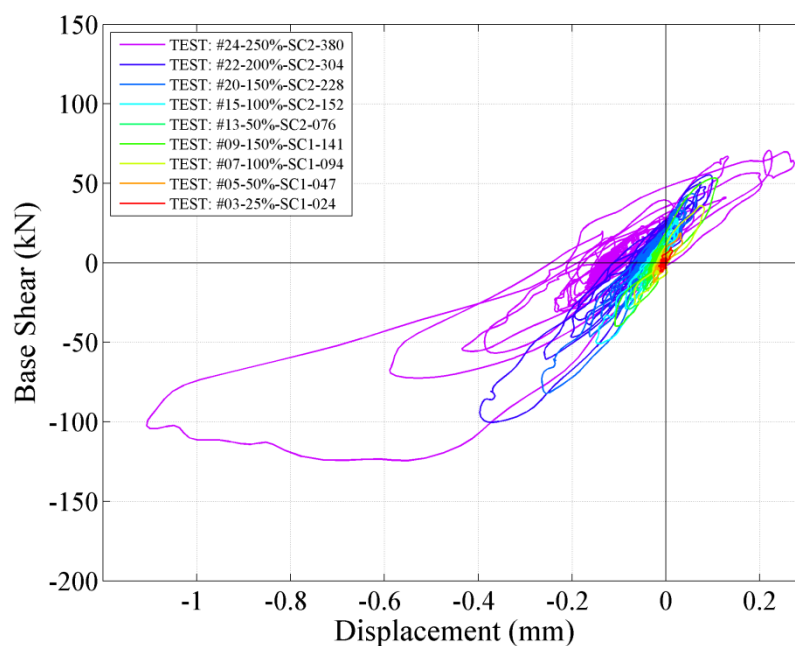


Figure 328. Base Shear vs First-Floor Average Displacement.

Figure 329 shows the specimen hysteretic response in terms of base shear vs first-floor average displacement for all tests.

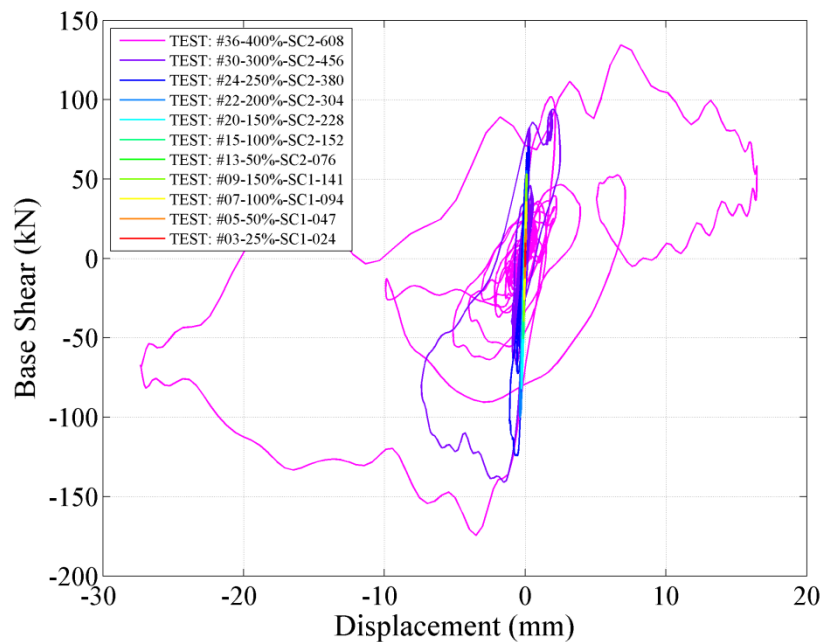


Figure 329. Base Shear vs First-Floor Average Displacement.

Figure 330 shows the specimen hysteretic response in terms of base shear coefficient vs first-floor average displacement for all tests. This acceleration was computed by dividing the total base shear (shown in the previous figure) by the entire specimen mass (approx. 33.5t).

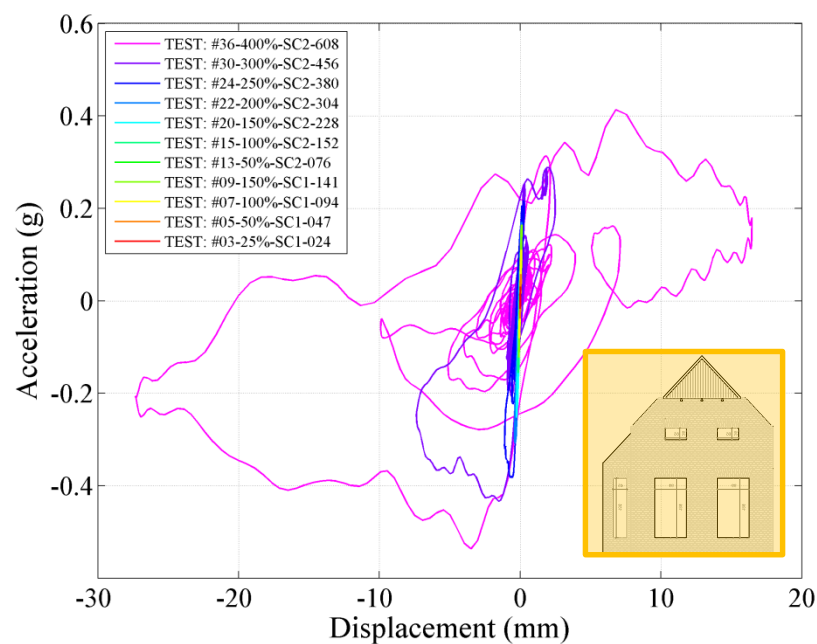


Figure 330. Average Acceleration vs First-Floor Average Displacement.

For comparison purpose, Figure 331 shows again the specimen hysteretic response in terms of shear coefficient vs first-floor average displacement for all tests. This acceleration was computed by dividing the shear computed at the mid-height ground floor level by the mass of the upper portion of the building (approx. 24.5t).

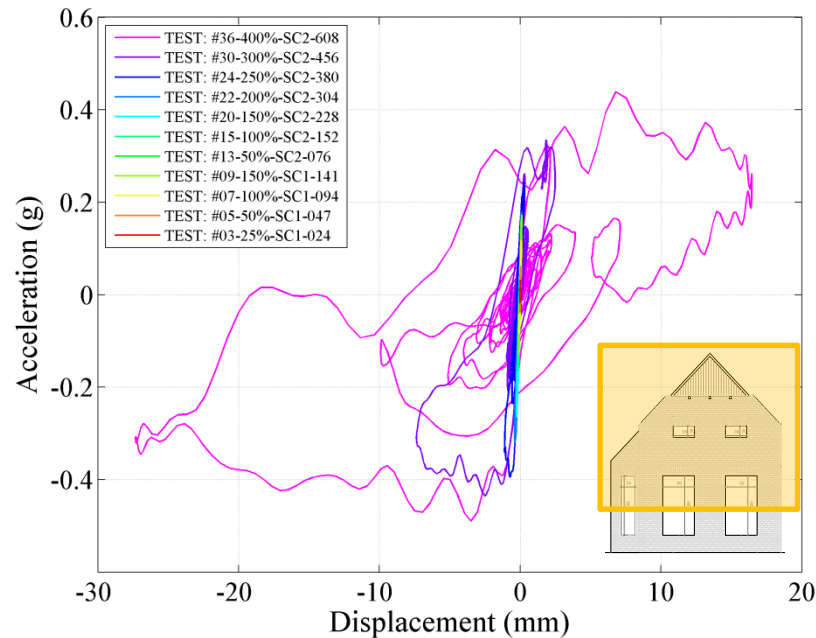


Figure 331. Average Acceleration vs First-Floor Average Displacement.

Figure 332 shows the base shear vs first-floor average displacement envelope identifying the points of peak base shear in red. The point of peak first-floor average displacement is shown in blue for the last test only.

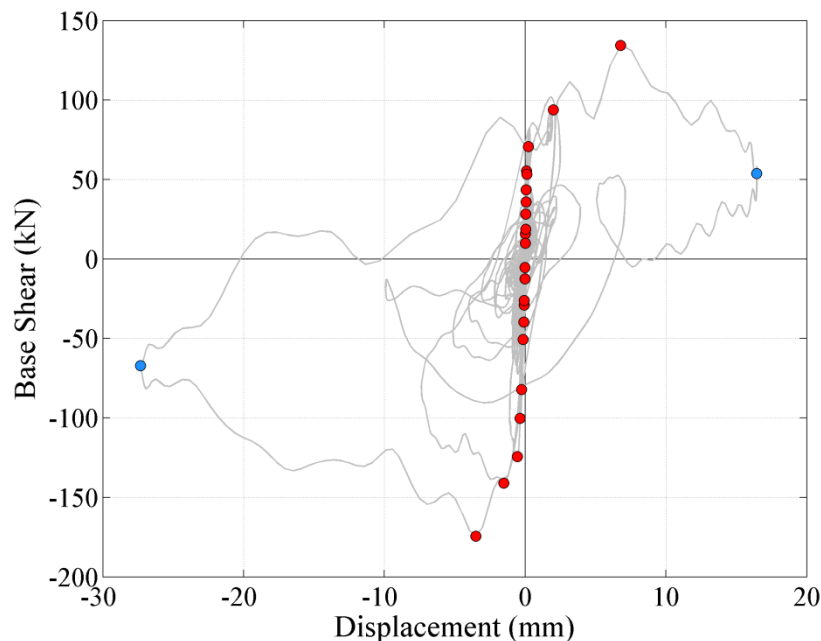


Figure 332. Base Shear vs First-Floor Average Displacement: Peak Responses.

Table 34 lists the maximum displacement and forces reached during each test.

Table 34. Base shear and First-Floor Average Displacement Peak Responses.

TEST #	Peak Base-Shear Responses				Peak Displacement Responses			
	Max F [kN]	D [mm]	Min F [kN]	D [mm]	Max D [mm]	F [mm]	Min D [mm]	F [mm]
25%_SC1_024	9,9	0,011	-5,4	-0,014	0,014	7,5	-0,016	-2,0
50%_SC1_047	18,8	0,031	-12,4	-0,028	0,035	12,9	-0,028	-12,4
100%_SC1_094	35,8	0,073	-26,1	-0,067	0,082	33,6	-0,067	-26,0
150%_SC1_141	53,4	0,105	-39,7	-0,107	0,109	51,2	-0,109	-39,1
50%_SC2_076	15,9	0,000	-28,9	-0,077	0,004	12,0	-0,083	-26,4
100%_SC2_152	28,2	0,023	-50,7	-0,145	0,027	22,6	-0,149	-49,5
150%_SC2_228	43,5	0,055	-82,1	-0,256	0,064	41,2	-0,264	-72,6
200%_SC2_304	55,4	0,094	-100,2	-0,369	0,099	52,3	-0,397	-94,1
250%_SC2_380	70,8	0,221	-124,2	-0,550	0,273	63,0	-1,107	-102,3
300%_SC2_456	93,9	1,996	-140,9	-1,521	2,479	66,1	-7,369	-80,4
400%_SC2_608	134,4	6,782	-174,3	-3,502	16,452	53,8	-27,331	-67,1

Figure 333 and Figure 334 show the peak recorded ground acceleration vs the maximum first-floor displacement for scenarios 1 and 2.

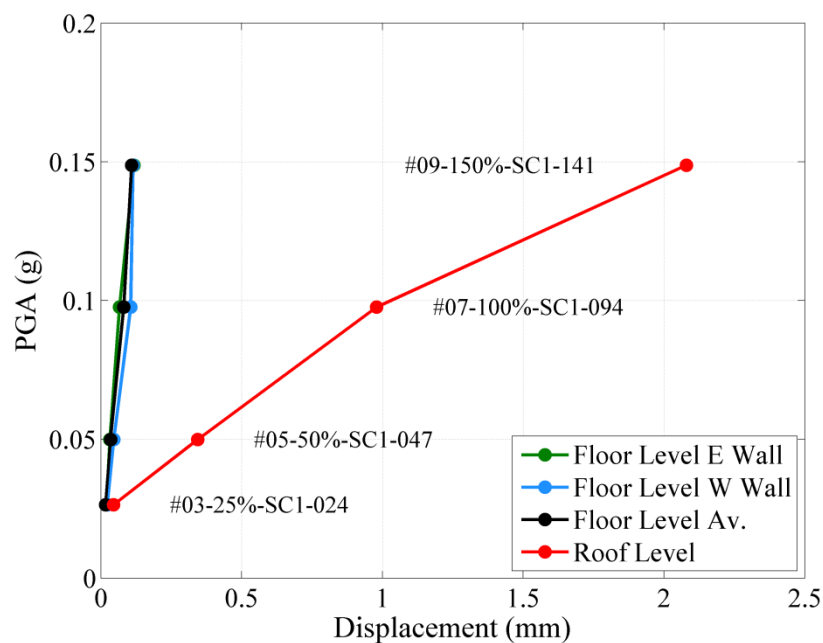


Figure 333. PGA vs Peak Displacement – Earthquake Scenario 1

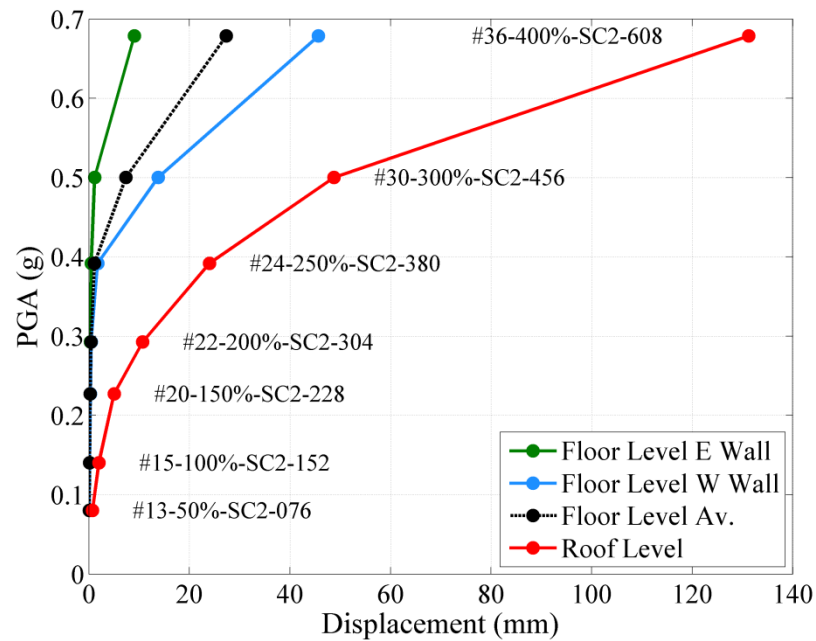


Figure 334. PGA vs Peak Displacement – Earthquake Scenario 2

Figure 335 shows the acceleration amplification at the floor and roof level through the test protocol.

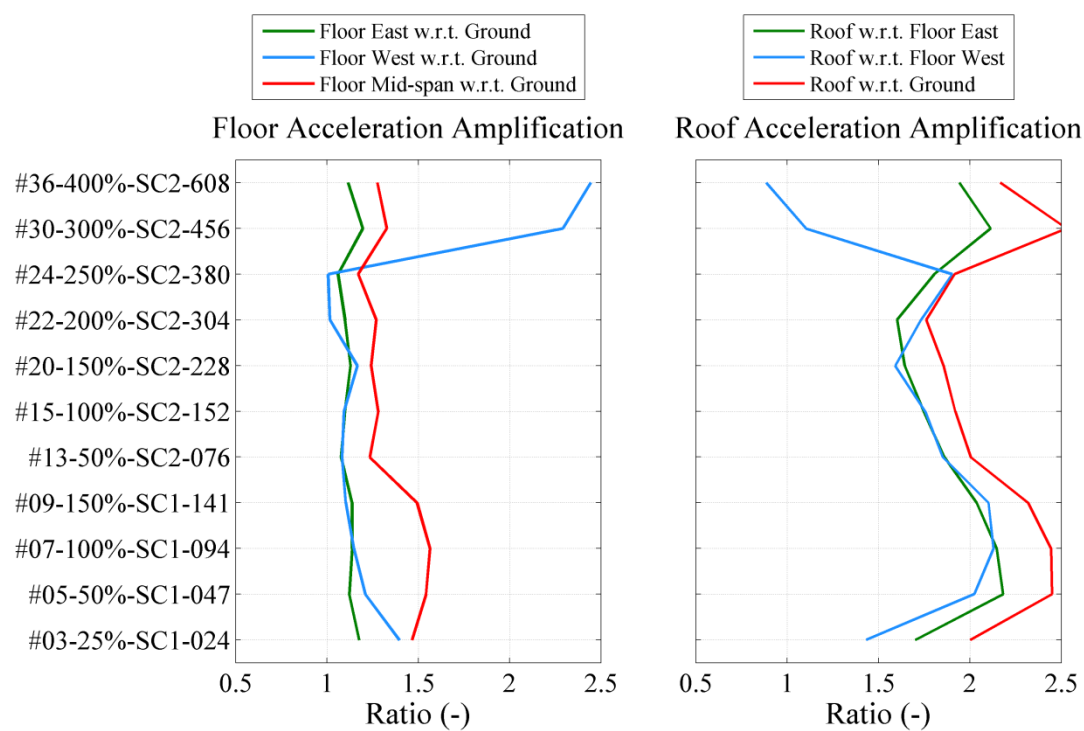


Figure 335. Acceleration Amplification.

5.4.15 Evolution of dynamic properties

The Output-Only modal identifications techniques, namely Operational Modal Analysis, are based on the dynamic response measurements of a virtual system under natural (ambient or operational) conditions, assuming that the excitations have random nature in time and in the physical space of the structure. Detailed info could be found in Appendix-C.

Dynamic Identification: results summary

In the following paragraphs a summary of the results obtained is presented in terms of singular values, frequencies of vibrations for each mode identified, of mode shapes and damage index.

Figure 336 reports the results from the Singular Value Decomposition, in particular the blue curves describe the variation of the first singular value in each of the tests while the red lines depict the change of the identified frequencies depending on the cumulating damage.

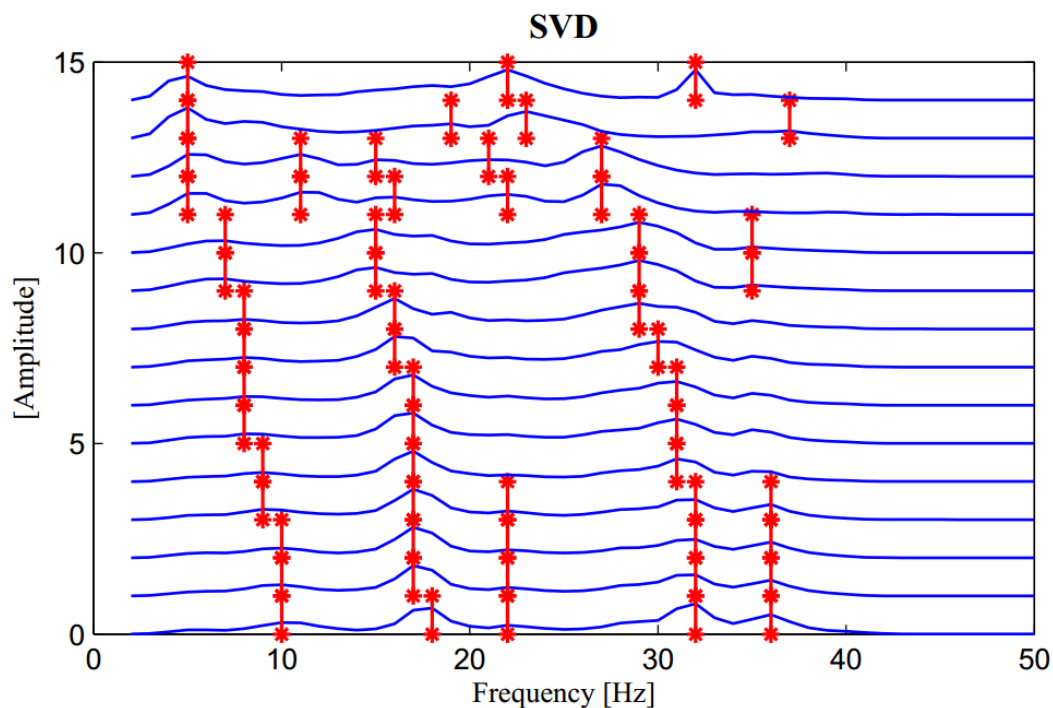


Figure 336. Singular Value Decomposition.



The following Table 35 summarises the frequencies of vibration and periods of the fundamental mode of the structure. The two methods PP and EFDD give the same results.

Table 35. Summary of the first fundamental frequencies and periods of vibration.

Test #	Test Name	Freq.	T
		[Hz]	[s]
1	01_RNDM_035	10.00	0.10
2	02_RNDM_035	10.00	0.10
3	25%_SC1_024	-	-
4	50%_SC1_047	-	-
5	50%_SC1_047	-	-
6	06_RNDM_035	10.00	0.10
7	100%_SC1_094	-	-
8	08_RNDM_035	9.00	0.11
9	150%_SC1_141	-	-
10	10_RNDM_050	9.00	0.11
11	50%_SC2_076	-	-
12	50%_SC2_076	-	-
13	50%_SC2_076	-	-
14	14_RNDM_035	8.00	0.13
15	100%_SC2_152	-	-
16	16_RNDM_050	8.00	0.13
17	50%_SC2_076	-	-
18	50%_SC2_076	-	-
19	50%_SC2_076	-	-
20	150%_SC2_228	-	-
21	21_RNDM_050	8.00	0.13
22	200%_SC2_304	-	-
23	23_RNDM_050	8.00	0.13
24	250%_SC2_380	-	-
25	25_RNDM_050	7.00	0.14
26	26_RNDM_050	7.00	0.14
27	100%_SC2_152	-	-
28	100%_SC2_152	-	-
29	100%_SC2_152	-	-
30	300%_SC2_456	-	-
31	31_RNDM_050	5.50	0.18
32	130%_SC2_200	-	-
33	130%_SC2_200	-	-
34	130%_SC2_200	-	-
35	35_RNDM_050	5.50	0.18
36	400%_SC2_608	-	-
37	37_RNDM_050	4.50	0.22
38	100%_SC2_152	-	-
39	100%_SC2_152	-	-
40	100%_SC2_152	-	-
41	200%_SC2_304	-	-
42	42_RNDM_050	4.50	0.22

The variation of the first fundamental period $T_{\#}$ at the different stage of the shaking table testing is depicted in Figure 337. The values reported in the histogram represent the initial period of the structure before the shake. The values of $T_{\#}$ is increasing with the damage, in particular the after the tests #30 300%_SC2_456 (+28%) and #36 400%_SC2_608 (+22%).

The light blue colour of the bars refers to the input SC1, while the blue bars refers to the input SC2.

The last bar named “Final” refers to the T_{42} , period of the structure at the end of all the test sequence.

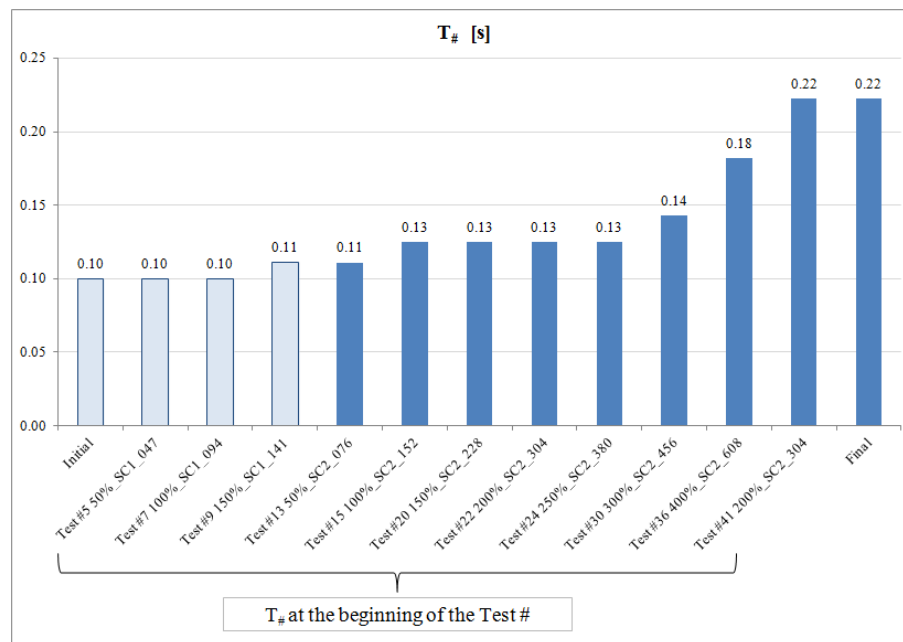


Figure 337. Variation of the initial fundamental period of vibration $T_{\#}$

A similar description of the effect of cumulative damage on the dynamic properties of the building is given by the index id_x , obtained as the ratio of the first period of vibration of the undamaged structure with respect to that of the damaged configuration $T_{\#}$, being equal to 1 when the structure is undamaged. In Figure 338 the variation of the $id_{\#}$ index is presented. The period $T_{\#}$ used to calculate the index $id_{\#}$ refers to the structure damaged after the shake #.

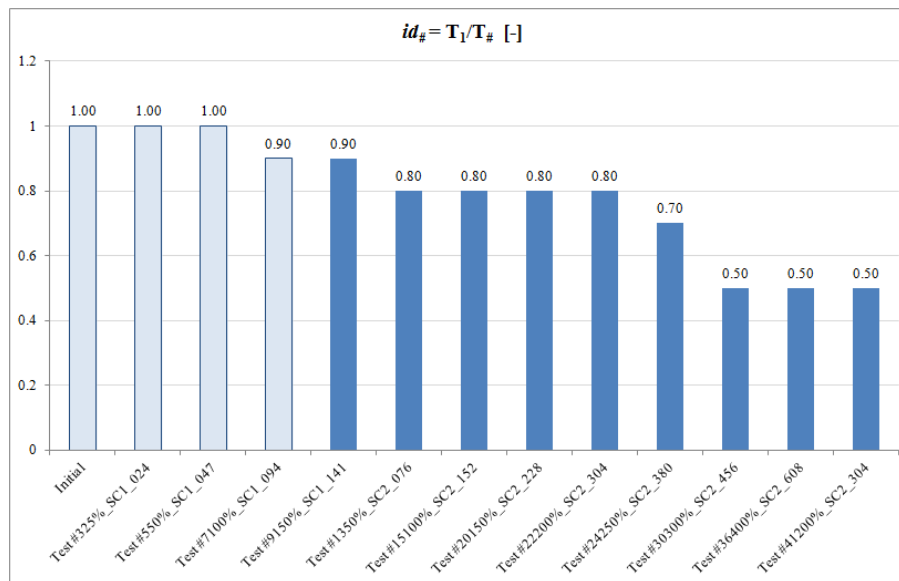


Figure 338. Variation of the damage index $id_{\#}$.

In Figure 339 the scheme of the geometrical undeformed configuration of the building is presented.

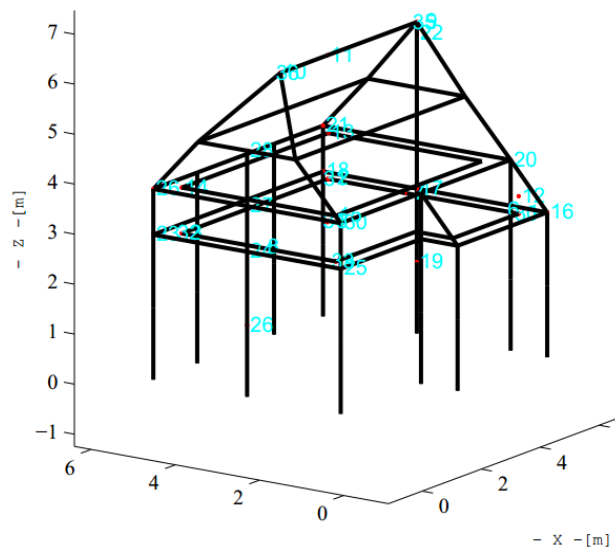


Figure 339. Undeformed configuration.

The first mode of vibration of the undamaged building T_1 has been identified at a fundamental frequency of 10 Hz.

Figure 340 depicts the deformed shape of the first mode deformed shape. It is possible to notice the considerable flexibility of the roof compared to the masonry walls.

The first modal shape of the structure is influenced mainly by the roof for all the tests performed on the building. A very similar deformed shape could be applied for all the runs.

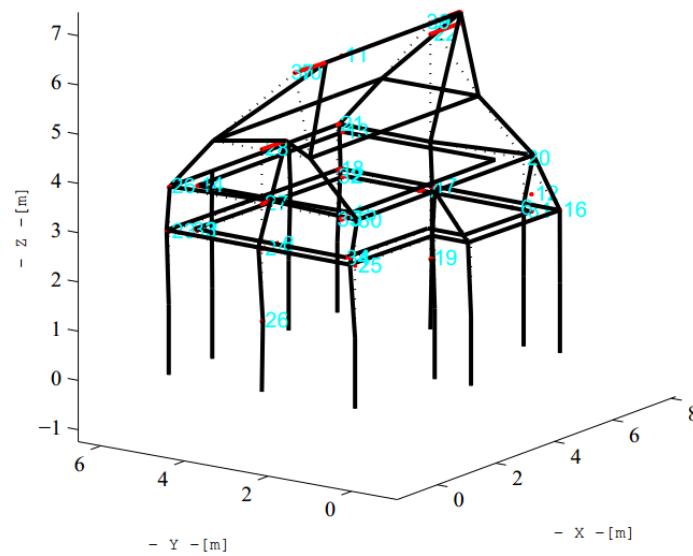


Figure 340. Test #1. Deformed shape of the first mode of vibration $T_1 = 0.1$ s.

The second mode of vibration of the undamaged building has been identified at a fundamental frequency of 18 Hz.

As possible to see in Figure 341 the modal shape of the 2nd mode is influenced not only by the roof but also by the walls at the ground floor as well as the gable walls.

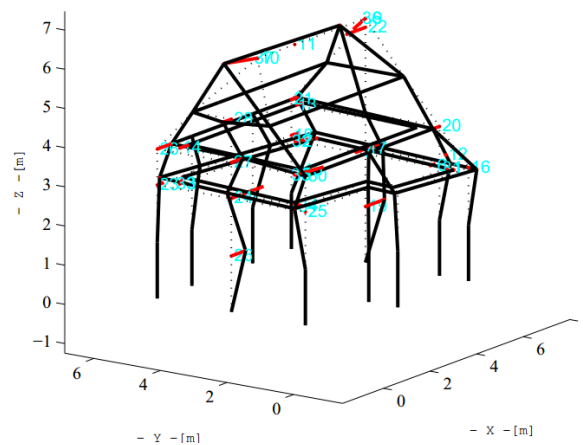


Figure 341. Test #1. Deformed shape of the second mode of vibration $T''_1 = 0.055$ s.

6 References

Arup [2016a] File note RFI0255 *Exposure Database V3*, Arup Report number 229746_031.0_REP1010

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