REBLOCK 100 ®
by SERIANA

Summary report of the
tests performed

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Patented system for the seismic strengthening of concrete block masonry walls

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Seismic behavior of tall infill masonry walls reinforced with a polymeric membrane
Reblock 100 by Seriana®

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Seismic behavior of tall infill masonry walls reinforced with a polymeric membrane

Reblock 100 by Seriana ®

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SERIANA S.p.A.

Contents

1. Introduction

This document presents a concise summary of the experimental and numerical research conducted at the **EUCENTRE Foundation** to assess the seismic performance of a **tall infill masonry wall**, typical of Italian industrial buildings, **strengthened by applying the Reblock 100 by Seriana®️ polymeric membrane**.
The study involved **full-scale shaking-table tests** on two infill panels: one in the **as-built (unretrofitted)** condition and one **retrofitted**, both tested on a shaking table. A **numerical study** was also developed, including **model calibration** against the experimental results and a **parametric analysis**. The experimental phase further comprised the **mechanical characterization of the masonry material**.

1.1 Motivation and objectives of the testing program

Seriana S.p.A. is an Italian company specialized in the seismic strengthening of precast reinforced-concrete structures for predominantly industrial/commercial/tertiary use. Seriana's R&D division has patented a strengthening solution based on a sprayed polymeric membrane applied to the outer faces of masonry infill panels. The patent covers Italy, the USA, and Türkiye. The system, **Reblock 100 by Seriana®️**, is a registered and monitored trademark with exclusive commercialization granted to Seriana S.p.A.

Following promising preliminary in-house static tests, Seriana S.p.A. and Eucentre launched an experimental campaign to investigate the seismic application of the system through dynamic tests performed at Eucentre's ShakeLab.

The testing focuses on the use of the Reblock 100 polymeric membrane as a strengthening material for unreinforced infill panels, with the objective of reducing seismic risk in industrial structures, where infill panels are commonly tall and slender.

The seismic vulnerability of such panels is a topic of major relevance—both because of their widespread use across the country and because they are highly vulnerable even in low-to-moderate seismicity areas. Moreover, there is a near absence of specific scientific studies, particularly experimental ones, on the dynamic response of tall infill panels. This gap is largely due to the logistical and power requirements of conducting full-scale shaking-table tests on such large specimens.

Some features of the Reblock 100 by Seriana®️ system share superficial analogies with existing solutions such as FRCM (Fabric/Fiber-Reinforced Cementitious Matrix) and TRM (Textile-Reinforced Mortar), while being materially and conceptually different in nature.

Although some studies have examined the seismic response of concrete block infill walls, current scientific and technical literature does not include specific experimental investigations on tall infills, such as those commonly used in industrial or precast structures.

In this context, advanced experimental and numerical research on the seismic behavior of tall masonry infills is particularly relevant and provides a significant contribution to both national and international literature. Moreover, exploring innovative retrofit methods to reduce the seismic vulnerability of infills further increases the interest and value of the research.

Among the possible solutions for seismic improvement, Seriana S.p.A., through its R&D division, proposed an innovative intervention for strengthening tall infills based on applying the Reblock 100 **polymeric membrane** to the masonry surface.

The Reblock 100 polymeric membrane is a spray-applied elastomer, a type of polymer that cures rapidly (within seconds) to form a flexible and durable coating.

1.2 Use of the Reblock100 Polymer Membrane as a Strengthening Solution

The use of the Reblock100 polymer membrane as a strengthening system for tall masonry infill walls represents an innovation at the global level. This research aims to evaluate the influence of this material on the seismic response of the non-structural element.

Dynamic shake table tests were carried out under two conditions:
• **As-built** (original state)
• **Strengthened** (with application of the Reblock100 polymer membrane)

To reproduce realistic boundary conditions of the tall panel, a steel reaction frame was designed to remain within the elastic range throughout all tests on both specimens. This conFiguretion ensured that both the base and the top of the panel received the same input from the shake table, thanks to a specifically upgraded steel frame developed from a pre-existing setup for the experimental campaign. The steel frame itself was designed to remain elastic and extremely stiff during the dynamic tests.

The first test was conducted on the as-built conFiguretion (concrete blocks and mortar), while the second concerned the same type of infill wall, strengthened with the proposed system.

Before and after each run, dynamic characterization tests were performed to evaluate the natural frequencies through the application of a random excitation signal.

For the dynamic input, an artificial accelerogram compliant with ACI 156 was selected, representing the reference standard for seismic acceptance criteria of purely non-structural elements. The accelerograms were progressively scaled, starting from low-intensity values up to high-intensity levels consistent with the objectives of the experimental campaign.

In addition to the dynamic tests, mechanical characterization tests were conducted on concrete blocks, mortar, and masonry specimens.

Numerical model

The study also proposed a numerical modeling strategy to simulate the out-of-plane (OOP) behavior of the tested infill panels, using a macro-model approach based on the finite element method (FEM).

Frame elements were used to represent the characteristic behavior of vertical masonry panels, enabling the simulation of one-way infills in both the as-built and strengthened conFiguretions.

The model was developed to:
• capture the global response,
• identify the key parameters governing out-of-plane performance,
• reproduce the experimental results,
• perform parametric analyses to assess the influence of different parameters on the structural response.

2. Description of the Specimens and Strengthening Solution

This chapter examines the masonry infill panels, with particular focus on their structural design, the applied strengthening solution, and the different phases of the strengthening process. It also presents the design methodology and describes the main features of the test frame used during the dynamic runs.

2.1 Structural frame of the infills

(a)   (b)

Figure 2.1 – View of the resisting frame and of the infilled frame.

Two composite reaction frames were fabricated to obtain a realistic conFiguretion of the masonry infill, with particular attention to the concrete foundation block and the top beam.

Each frame consisted of:

• a reinforced-concrete foundation at the base,

• a reinforced-concrete beam at the top,

• two supporting steel columns.

The structural frame was designed to be sufficiently stiff to effectively transfer the dynamic motion from the concrete foundation to the top beam. The choice to use a concrete base and a concrete top beam was made to replicate real cases commonly observed in industrial applications.

Main dimensions:

• Reinforced-concrete foundation: 5.00 m × 1.70 m in plan, thickness 0.40 m.

• Top reinforced-concrete beam: 3.70 m length, 0.45 m width, 0.30 m height.

• Steel columns: HEA 300 sections, nominal height 5 m.

• Diagonal steel braces: HEB 180 sections.

• All steel components were fabricated with S275 grade steel.

The masonry infill panels were built within the composite frames. Figure 2.2a–b shows the frame geometry and a photograph of the setup. Both specimens were constructed identically; however, the strengthening solution was applied to one of the two, which will be described in detail in the following section.
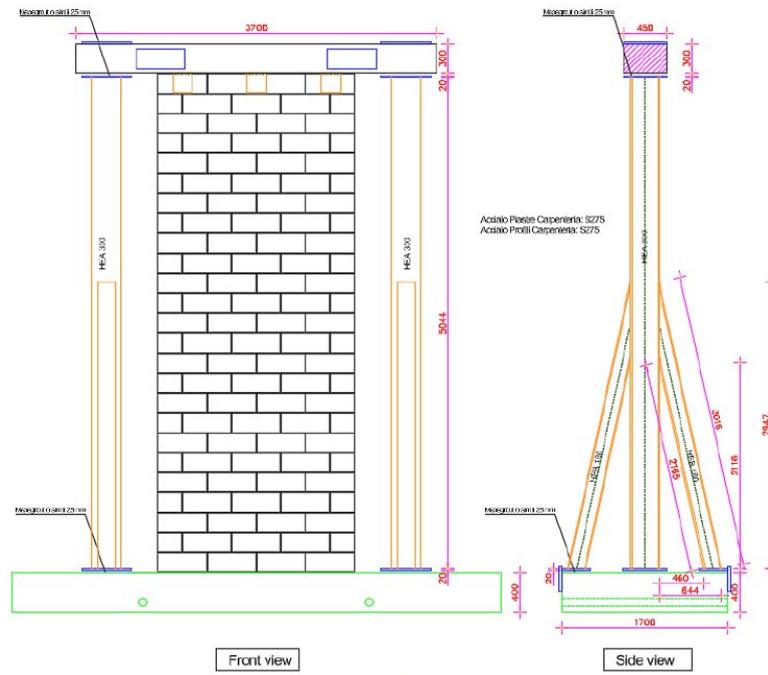
(a)

(b)

Figure 2.2 – Details of the infill frame: drawings of the frame (a) and photos (b).

2.2 Infill Panel

The infill panel represents a typical conFiguretion found in industrial buildings, where the load-bearing structure usually consists of a single-story precast reinforced-concrete frame.

The design of the infill panel began with a preliminary study aimed at defining the most representative type of masonry infill commonly used in Italy. A pre-design phase followed, based on statistical data regarding the geometric characteristics of existing infills and the mechanical properties of the masonry materials.

Once the panel dimensions were defined, attention was focused on the construction details, by examining real case studies of similar panels and the most common building practices historically adopted for their construction. The upper boundary condition, in contact with the reinforced-concrete beam, was also designed with particular care in accordance with the experimental objectives.

The usual boundary conditions of the infill are vertical (at the base and at the top of the panel). In some real cases, the infill is also laterally constrained through rigid connections to the columns. For the purpose of this study, only vertical boundary conditions were considered, as they are the most representative and significant, while maintaining the typical slenderness ratio and the materials commonly adopted in such constructions.

The masonry panel was built using vertically hollow concrete blocks, each with two vertical voids, and standard M2.5 mortar.

Nominal dimensions of the panel:
• width: 2.0 m
• height: 5.0 m
• thickness: 0.20 m

The infill panels and the small masonry walls used for material characterization tests were constructed outside the laboratory and transferred indoors after a curing period of at least 28 days.

Subsequently, the panels were installed on the unidirectional shake table of the EUCENTRE ShakeLab and anchored using post-tensioned bars fixed into the concrete foundation. The top beam of the framed specimen was also connected to the rigid steel reaction frame to ensure consistent transmission of the dynamic input to both the base and the top of the panel.

Masses:
• infill panel: 2.2 tons
• frame with reinforced-concrete foundation: 10.5 tons
• total weight on the shake table: 12.7 tons

The materials selected for the infill panel were:
• **concrete blocks;**
• **lime-cement mortar.**

This construction technique is widely used in Italy (see Figure 2.3) and is consistent with the field surveys conducted by Seriana S.p.A.

Figure 2.3 – Examples of infill panels

The mechanical properties of the concrete blocks and mortar were selected to be representative of "tall" masonry infill panels typically found in industrial structures. For this purpose, the following materials were chosen:

• Concrete block named "Blocco 20x20x50 EI 120 Facciavista," with nominal dimensions of 500 × 200 × 200 mm, produced by Viprapac Geo (Figure 2.4).
• Lime–cement masonry mortar (M2.5), composed of hydrated lime, Portland cement, and selected sands, produced by Fassa Bortolo (Figure 2.6).

Additionally, a running bond pattern—the most commonly used bond type—was adopted, as shown in Figure 2.6.

(a)

Blocco
20x20x50 EI 120 facciavista

CARATTERISTICHE DIMENSIONALI

Tipo di impasto	calcestruzzo
Dimensioni modulari [cm] Lunghezza x Larghezza x Altezza	50x20x20
Dimensioni Nomirali [mm] Lunghezza x Larghezza x Altezza	495x195x195
Spessore minimo per parete esterna [mm]	28
Planarità [mm]	<2
Classe di tolleranza	D1 (L: +3-5 mm; W: +3-5 mm; H: +3-5 mm)
Percentuale/Classe di foratura	54% - F4

CARATTERISTICHE TECNICHE

Normativa di riferimento	UNI EN 771-3:2011
Gruppo Eurocodice (EN 1996-1)	Gruppo 2
Massa volumica netta/lorda (kg/m³) (±10%)	2300/925
Peso elemento standard (kg)	20,00
Blocchi in opera [n°/ m²]	10,00
Peso medio della muratura (kg/ m²)	220,00
Assorbimento per capillarità (g/ m² s^{0,5})	<35
Resistenza media a compressione (N/mm²)	8,0
Conducibilità equivalente λ_{eq} (W/mK) a secco	0,297
Resistenza termica R (m²K/W)	1,796
Trasmittanza U (W/ m²K) a secco	3,00
Potere fono isolante Rw (dB)	47,00
Resistenza al fuoco (min)	EI 120
Reazione al fuoco	Euroclasse A1 – Classe 0 (zero)
Permeabilità al vapore (μ)	5/15
Spostamenti dovuti all'umidità (mm/m)	0,40
Aderenza al taglio (N/mm²)	0,15
Colore	Grigio, Antracite, Rosso, Ocra, T.Moro, Muschio, Bianco, Avorio, Ghiaccio, Giallo, Senape, Verde
Pezzi per imballo	50
Pezzi speciali compresi nell'imballo	20 testa piana divisibili
Note	-

(b)

Figure 2.4 – Details of the units used for the infill panel.

(a)

Technical Data

Specific gravity of the powder	approx. 1,400 kg/m³
Minimum thickness	10 mm
Granulometry	< 1.5 mm
Mixing water	22-24 %
Yield	approx. 13.3 kg/m² with 10 mm thickness
Density of hardened plaster	approx. 1,530 kg/m³
Compressive strength after 28 days (EN 1015-11)	approx. 2.5 N/mm²
Modulus of elasticity after 28 days	approx. 3,000 N/mm²
Water vapour diffusion resistance factor (EN 1015-19)	$\mu \leq 14$ (measured value)
Capillary water absorption coefficient (EN 1015-18)	W0
Thermal conductivity coefficient (EN 1745)	$\lambda = 0.55$ W/m·K (tabulated value)
Compliant with standard EN 998-1	GP-CSII-W0

The performance values listed above are obtained by mixing the product with 23% water in a controlled temperature and humidity environment (20±1°C and 60±5% RH)

(b)

Figure 2.5 – Details of the mortar used for the infill panel.
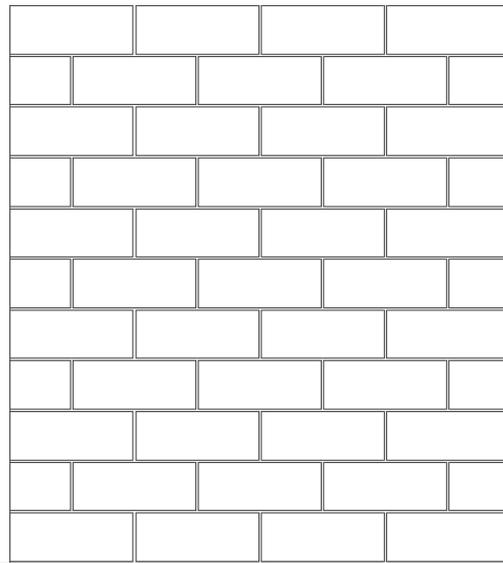
Figure 2.6 – Typical masonry texture for the definition of masonry for running bond.

Two full-scale unreinforced masonry infill walls were constructed, identical in geometry and configuration.

• The first specimen, named TIU-1 (Tall Infill Unreinforced), represents the as-built condition.
• The second specimen, named TIR-1 (Tall Infill Reinforced), represents the strengthened condition.

The TIU-1 specimen, together with the related characterization tests, is classified as the as-built state, while the TIR-1 specimen and its corresponding tests are classified as the strengthened state. The details of the strengthening solution are presented in the following section; the construction details of the as-built condition are reported here.

The tested as-built infill type corresponds to a traditional "weak" unreinforced masonry wall, widely used in industrial applications, built with vertically hollow concrete blocks having a nominal void volume ratio of 54%.

For the mortar joints, a general-purpose lime–cement mortar type M2.5 was used, with a nominal compressive strength of 2.50 MPa, consistent with common construction practices.

The unit weight of this masonry type is 11 kN/m³, resulting in a total panel weight of approximately 22 kN.

Figure 2.7 – A view of concrete units on the concrete foundation

Both specimens measured 2 m in length, 5 m in height, and 20 cm in thickness. The out-of-plane slenderness ratio (λ) of the infill panels was approximately 25 (height-to-thickness ratio).

Both specimens were constructed identically:
• built on a base mortar bed in direct contact with the reinforced-concrete foundation;
• the upper joint, between the top of the masonry panel and the reinforced-concrete beam, was filled with mortar along the top edge;
• along the sides, the panels had free vertical joints with respect to the structural columns, allowing the development of a vertical bending/arching failure mechanism while avoiding the influence of horizontal or bidirectional bending/arching mechanisms.

The horizontal mortar joints were approximately 1 cm thick, while the head joints were only partially filled, in order to faithfully reproduce the typical Italian construction practices of the 1960s–1980s.

Figures 2.8a and 2.8b show photographs taken during and after the construction phase, respectively.

(a)

(b)

Figure 2.8 – Construction of the infill panels (a) and the general view of the specimen (b).

2.3 Description and Application of the Strengthening Solution

The Reblock100 polymer membrane is a high-performance spray-applied elastomer obtained through the rapid reaction between an isocyanate component and a resin blend. It is widely used in industrial applications thanks to its excellent mechanical, chemical, and environmental resistance properties.

The strengthening process of an unreinforced masonry panel using Reblock100 involves several fundamental steps designed to ensure proper adhesion and mechanical interlocking between the polymer membrane and the masonry substrate, as well as to achieve a uniform coating thickness on the surface. The application methodology, illustrated in Figures 2.10, is as follows:

• **Surface preparation**
Application of a two-component epoxy system mixed until completely homogenized and brushed onto the panel surface to obtain a uniform and continuous coating.

• **Quartz broadcasting**
While the epoxy layer is still fresh, manual distribution of spherical quartz aggregate (grain size 0.7–1.2 mm) over the surface to enhance the adhesion of the subsequent polymer membrane layer by creating a rough and textured substrate.

• **Epoxy curing**
Curing for 24 hours under ambient conditions, allowing the epoxy–quartz interface to fully harden.

• **Polymer membrane application**
High-pressure spraying of the two-component polymer membrane, mixed immediately prior to application. The chemical reaction was activated at a mixing temperature of approximately 78°C inside the spray gun.

• **Coating buildup**
Application in multiple passes—typically four coats—until a uniform thickness of approximately 4 mm was achieved. Each coat was applied with overlapping movements to ensure continuity and homogeneity. Due to the rapid polymerization of the material, the entire process must be carried out swiftly to minimize idle time between successive coats.

Figure 2.10 – The application process of the strengthening solution.

3. Configuration, Protocol, and Instrumentation of the Dynamic Test

The out-of-plane seismic behavior of the masonry infill panels was investigated through dynamic shake table tests. Two specimens were tested, representing the as-built and the strengthened conditions, respectively. Both out-of-plane tests were carried out up to the complete collapse of the infill wall.

The experimental tests on both panels (as-built and strengthened) were conducted separately, each over a duration of two days. Output data were inspected in real time at the end of each main seismic input run. Visual inspections were also performed to assess any damage to the structure.

The dynamic inputs consisted of artificial seismic signals, calibrated against a horizontal Required Response Spectrum (RRS) inspired by the ICC ES AC156 standard — the U.S. reference standard for the seismic qualification of non-structural components.

Before and after each run, dynamic characterization tests were performed to evaluate the natural frequencies by applying a random excitation signal.

The experimental setup was installed on the unidirectional shake table of the EUCENTRE ShakeLab, oriented to excite the specimens in one-way out-of-plane bending. Figure 3.1 shows several images of the test setup.

A rigid steel reaction frame was designed to ensure proper transmission of the dynamic motion from the shake table to the top of the panel while minimizing amplifications. The reaction frame was anchored to the shake table using steel rods and bolts.

The connection between the reaction frame and the top beam of the specimen was achieved through pairs of steel braces, ensuring that the panel was restrained both at the base and at the top. The braces were rigidly connected to the top beam through steel plates to prevent any relative rotation or translation.

To ensure proper stress distribution, a neoprene layer was inserted between the steel plates and the reinforced-concrete top beam. This configuration enabled the simultaneous transfer of the horizontal dynamic input from the table to the top of the panel.

The top restraint of the panel was obtained using an L-shaped steel profile system. The bottom of the infill rested on a mortar bed placed on the specimen's foundation, in accordance with standard construction practice.

Figure 3.1. General view of test setup: drawing of the resistance frame (a) and side view of the resistance frame (b) and (c).

Figure 3.2 – General view of the instrumentation with a detailed view of the cameras used for the optical acquisition system.

4. Results of the Shake Table Tests

This chapter compares the response of the masonry infill panels in both the as-built and strengthened conditions when subjected to the same nominal seismic intensity.
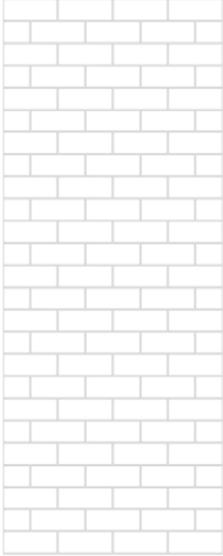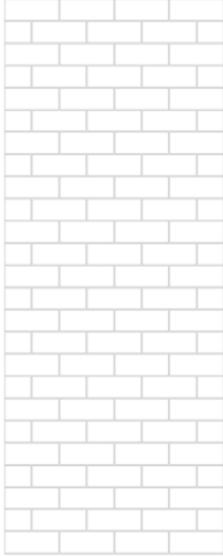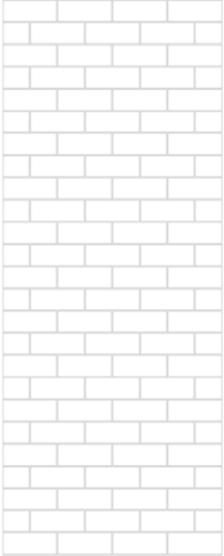
The parameters analyzed in this comparison include:
• crack pattern and damage distribution,
• accelerations,
• relative amplification with respect to the PGA,
• displacements,
• and the resulting envelope of the hysteresis curves.

4.1 Summary Tables

This section summarizes the main results of the tests conducted in both the as-built and strengthened conditions, presented in tabular form.

To clarify the comparison, the following parameters are reported: damage patterns, maximum recorded forces, corresponding displacements, fundamental frequency of the masonry panel, and the identified damage state for each test, as a function of the "base/table PGA" and the measured "average Sa" value.

Condizione originaria	**Condizione rinforzata**
PGA base/shake table = 0.12 g	PGA base/shake table = 0.15 g
Mean Sa feedback = 0.24 g	Mean Sa feedback = 0.31 g
Frequency = 10.61 Hz	Frequency = 17.06 Hz
Main cracks = -	Main cracks = -
Limit state = -	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [5.35kN, -4.45kN, 0.66mm, -0.69mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [4.75kN, -5.52kN, 0.55mm, -0.07mm]
Resisting Frame Side Non-Resisting Frame Side	Resisting Frame Side Non-Resisting Frame Side

Condizione originaria	**Condizione rinforzata**
PGA base/shake table = 0.22 g	PGA base/shake table = 0.27 g
Mean Sa feedback = 0.40 g	Mean Sa feedback = 0.57 g
Frequency = 8.54 Hz	Frequency = 16.75 Hz
Main cracks = -	Main cracks = -
Limit state = -	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [10.1kN, -9.07kN, 1.74mm, -3.73mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [7.54kN, -8.02kN, 0.17mm, -0.52mm]
Resisting Frame Side Non-Resisting Frame Side	Resisting Frame Side Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
PGA base/shake table = 0.39 g	PGA base/shake table = 0.38 g
Mean Sa feedback = 0.77 g	Mean Sa feedback = 0.85 g
Frequency = 5.80 Hz	Frequency = 17.20 Hz
Main cracks = -	Main cracks = -
Limit state = -	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [18.2kN, -16.3kN, 6.63mm, -4.54mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [12.5kN, -11.9kN, 0.80mm, -1.34mm]
Resisting Frame Side · Non-Resisting Frame Side	Resisting Frame Side · Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
PGA base/shake table = 0.52 g	PGA base/shake table = 0.51 g
Mean Sa feedback = 1.03 g	Mean Sa feedback = 1.14 g
Frequency = 4.93 Hz	Frequency = 16.83 Hz
Main cracks = around base + above 14th course	Main cracks = -
Limit state = Operational	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [27.1kN, -23.6kN, 12.7mm, -12.5mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [19.2kN, -21.9kN, 2.17mm, -3.78mm]
Resisting Frame Side · Non-Resisting Frame Side	Resisting Frame Side · Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
PGA base/shake table = 0.67 g	PGA base/shake table = 0.66 g
Mean Sa feedback = 1.28 g	Mean Sa feedback = 1.42 g
Frequency = 4.98 Hz	Frequency = 16.55 Hz
Main cracks = around base & above 14th course	Main cracks = -
Limit state = Moderate Damage	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [33.2kN, -28.7kN, 0.69mm, -0.66mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [23.2kN, -24.9kN, 3.56mm, -4.91mm]
Resisting Frame Side Non-Resisting Frame Side	Resisting Frame Side Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
PGA base/shake table = 1.07 g	PGA base/shake table = 1.05 g
Mean Sa feedback = 2.10 g	Mean Sa feedback = 2.13 g
Frequency = 4.61 Hz	Frequency = 16.62 Hz
Main cracks = around boundary & mid-height	Main cracks = -
Limit state = Severe Damage	Limit state = -
$[F^+, F^-, \delta^+, \delta^-]$ = [46.1kN, -40.6kN, 0.69mm, -0.66mm]	$[F^+, F^-, \delta^+, \delta^-]$ = [36.3kN, -35.7kN, 9.2mm, -7.6mm]
Resisting Frame Side Non-Resisting Frame Side	Resisting Frame Side Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
PGA base/shake table = 1.30 g Mean Sa feedback = 2.38 g Frequency = 3.88 Hz Main cracks = - Limit state = collapse [F⁺, F⁻, δ⁺, δ⁻] = [47.5kN, -32.8kN, 0.69mm, -0.66mm] -	PGA base/shake table = 1.56 g Mean Sa feedback = 2.96 g Frequency = 14.85 Hz Main cracks = - Limit state = - [F⁺, F⁻, δ⁺, δ⁻] = [46.0kN, -53.8kN, 16.4mm, -18.7mm] Resisting Frame Side Non-Resisting Frame Side

Condizione originaria	Condizione rinforzata
   -	PGA base/shake table = 2.01 g Mean Sa feedback = 3.78 g Frequency = 11.76 Hz Main cracks = - Limit state =- [F⁺, F⁻, δ⁺, δ⁻] = [66.5kN, -64.5kN, 43.3mm, -22.5mm] Resisting Frame Side Non-Resisting Frame Side
-	PGA base/shake table = 2.13 g Mean Sa shake table = 3.71 g Frequency = 7.54 Hz Main cracks = - Limit state = collapse [F⁺, F⁻, δ⁺, δ⁻] = [60.7kN, -60.3kN, 68.4mm, -87.5mm]

The comparison of the damage patterns highlights a clear difference between the unreinforced and the strengthened specimens.

The unreinforced wall exhibited horizontal cracking starting from the test with a nominal PGA = 0.40 g (actual ~0.50–0.55 g), which progressively intensified up to collapse at a nominal PGA = 1.0 g (actual PGA = 1.30 g, actual Sa = 2.38 g).

In contrast, the strengthened wall showed no visible cracking up to a nominal PGA = 1.50 g (with an actual PGA likely exceeding 2.13 g and an actual Sa = 3.71 g), collapsing only under an effective acceleration and spectral acceleration Sa approximately 60% higher than those recorded for the unreinforced case.

Table 4.2 presents the maximum accelerations and the corresponding mid-panel acceleration records for both specimens, in both directions.

Nominal PGA (g)	a_{max}^{centre} (g)			
	North face		South Face	
	As-built	Strengthened	As-built	Strengthened
0.10	0.36	0.33	-0.37	-0.31
0.20	1.16	0.45	-1.03	-0.44
0.30	1.82	0.76	-1.64	-0.74
0.40	3.01	1.51	-2.56	-1.36
0.50	2.96	1.77	-3.40	-1.56
0.75	5.18	2.38	-5.04	-2.28
1.00	5.15	3.71	-5.28	-3.32
1.25	-	4.52	-	-4.60
1.50	-	3.92	-	-5.50
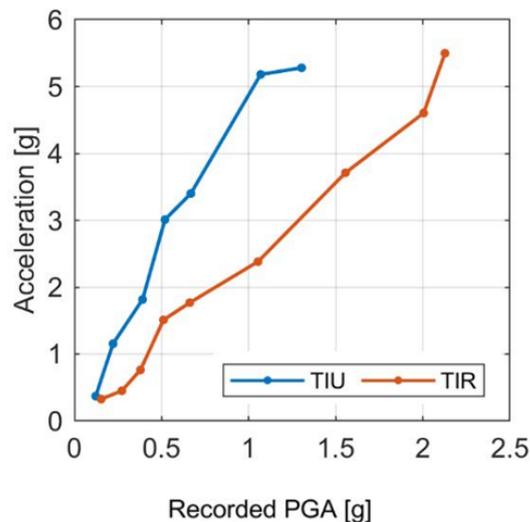
Table 4.2. Absolute maximum values of accelerations in both directions for as-built and strengthened specimens.

With the exception of the first seismic motion (nominal PGA = 0.10 g) and the motion with nominal PGA = 1.0 g, the measured acceleration was approximately twice as high in the as-built specimen compared to the strengthened one, in both directions.

This difference is mainly attributed to the lower fundamental frequency of the as-built specimen compared to the strengthened one, which places its period within the plateau of the response spectrum. In contrast, the higher stiffness of the strengthened specimen results in a higher fundamental frequency, shifting the period outside the plateau and leading to lower spectral acceleration values.

Table 4.3 presents the maximum values—both in the positive and negative directions—of the acceleration amplifications at the mid-height of the panel with respect to the effective PGA (along with the corresponding graphs).
The observed trend is similar to that of the absolute maximum acceleration values. The assessment of maximum amplifications shows that, following strengthening, the acceleration demand is significantly reduced.

For the as-built specimen, the ratio between the maximum acceleration at the mid-height of the panel and that at the base varied considerably, ranging from 2.92 to 6.38, with an average value of 4.86.
For the strengthened specimen, the variation was more contained, with values between 1.66 and 4.03, and an average of 2.36.

This difference is attributed to the higher stiffness provided by the Reblock 100 by Seriana strengthening system, in addition to the previously discussed causes affecting the acceleration values.
On average, the acceleration amplification in the strengthened infill wall was approximately 50% of that recorded in the as-built condition.

Nominal PGA (g)	Acceleration amplifications at mid-height [-]			
	North face		South Face	
	As-built	**Strengthened**	**As-built**	**Strengthened**
0.10	3.41	2.18	2.92	2.04
0.20	6.80	1.76	4.68	1.66
0.30	5.31	2.07	4.24	2.02
0.40	6.38	3.01	4.85	2.65
0.50	4.90	2.68	5.08	2.35
0.75	5.12	2.27	4.67	2.16
1.00	3.98	2.44	5.80	2.56
1.25	-	2.48	-	2.31
1.50	-	1.80	-	4.03
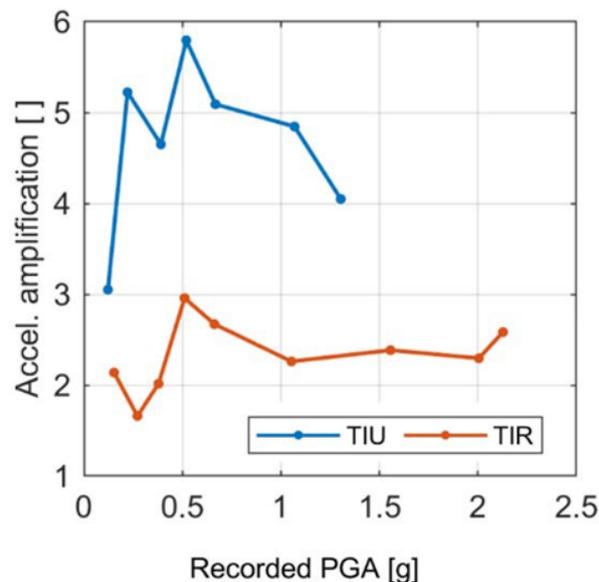
Table 4.3. Acceleration amplification values in both directions for the specimens in the as-built and strengthened conditions.

Table 4.4 reports the absolute force values, in both the positive and negative directions, along with the corresponding graphs.

The comparison between the as-built and strengthened specimens shows that the strengthened panel exhibits nearly double the load-bearing capacity of the as-built one; moreover, the latter displayed an extremely brittle post-peak response.

The strengthened specimen tends to show slightly lower peak forces compared to the original condition when subjected to nearly the same peak ground acceleration (PGA). However, the difference is minimal.

The strengthened solution was able to withstand seismic motions of higher intensity, thus achieving a greater maximum force value. Specifically, the peak strength of the infill wall treated with Reblock 100 was approximately 40% higher than that of the unreinforced panel.

Nominal PGA (g)	Force			
	North face		South Face	
	As-built	Strengthened	As-built	Strengthened
0.10	5.35	4.75	-4.45	-5.52
0.20	10.12	7.54	-9.07	-8.02
0.30	18.15	12.48	-16.30	-11.97
0.40	27.14	19.17	-23.61	-21.95
0.50	33.22	23.22	-28.65	-24.95
0.75	46.09	35.71	-40.60	-36.30
1.00	47.49	46.00	-32.83	-53.84
1.25	-	66.52	-	-64.51
1.50	-	60.70	-	-60.27
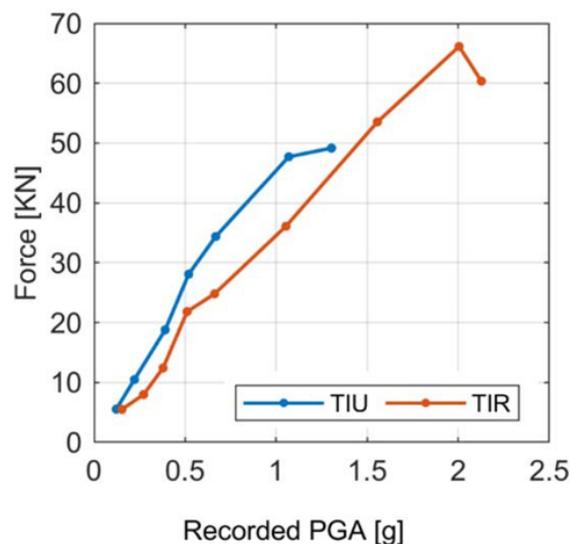
Table 4.4. Maximum force values in both directions for the specimens in the as-built and strengthened conditions.

Table 4.5 presents the maximum displacement values in both the positive and negative directions, along with the corresponding graphs.

It is evident that the as-built specimen exhibited significantly larger out-of-plane displacements compared to the strengthened specimen, even under the same seismic action level and already during the initial tests. This behavior is attributed to the formation of plastic hinges approximately at mid-height of the panel and to the lower stiffness of the unreinforced wall compared to the strengthened one.

On average, the out-of-plane displacements of the strengthened specimen were between 4 and 5 times smaller than those of the unreinforced specimen.

| Nominal PGA (g) | max displacements at mid-height | | | |
| | North face | | South Face | |
	As-built	Strengthened	As-built	Strengthened
0.10	1.54	1.85	-1.69	-1.57
0.20	3.94	0.93	-4.59	-0.98
0.30	9.06	1.33	-8.41	-1.44
0.40	12.74	2.75	-12.80	-3.79
0.50	19.38	3.57	-18.91	-4.91
0.75	50.45	7.73	-63.10	-9.43
1.00	collapse	16.43	collapse	-19.97
1.25	-	43.69	-	-59.13
1.50	-	collapse	-	collapse
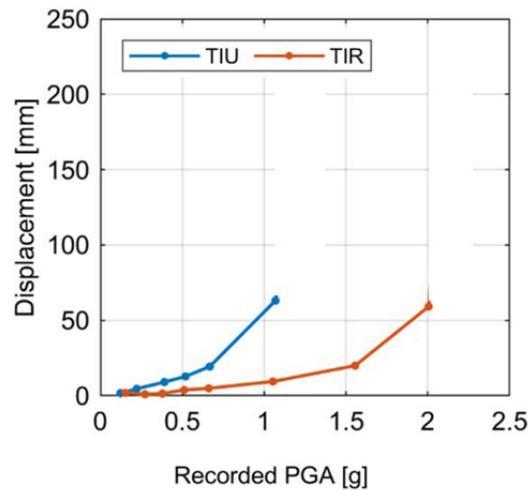
Table 4.5. Maximum displacement values in both directions for the specimens in the as-built and strengthened conditions.

4.2 Accelerations

Figure 4.1 shows, for each seismic intensity level, the comparison between the original (TIU) and the strengthened (TIR) conditions in terms of absolute peak acceleration.

Measurements were taken at seven different heights, listed from bottom to top:
• concrete base (Table),
• bottom of the infill (Bottom),
• one-quarter of the panel height (H/4),
• mid-height of the infill (H/2),
• three-quarters of the panel height (3H/4),
• top of the infill (Top),
• top concrete beam of the infill frame (Beam).

Except for the lowest nominal PGA values (0.1 g and 0.2 g for the as-built specimen, and 0.3 g for the strengthened one), both specimens exhibited maximum accelerations at mid-height, considering the absolute maximum values along the height (recorded at different instants).

Specifically, the as-built specimen (TIU) showed higher accelerations at various heights compared to the strengthened specimen (TIR). This difference is attributed to the lower stiffness of the TIU specimen, also reflected in its lower natural frequency.

In detail, the maximum acceleration peak recorded before the collapse of both specimens was approximately 5.30–5.50 g, with a slightly higher value for the strengthened specimen (TIR).

The distribution of the peak acceleration along the specimen height, although not referring to the same instant, remains similar throughout the test evolution; the strengthened infill walls (TIR) appear to respond with a slight delay relative to the applied seismic motion. For example, the values and distribution recorded for TIU at a nominal PGA of 0.50 g are comparable to those measured for TIR at a nominal PGA of 1.00 g.

This discrepancy is attributable to the different stiffness levels, distinct damage progression, and the activation of arching mechanisms.
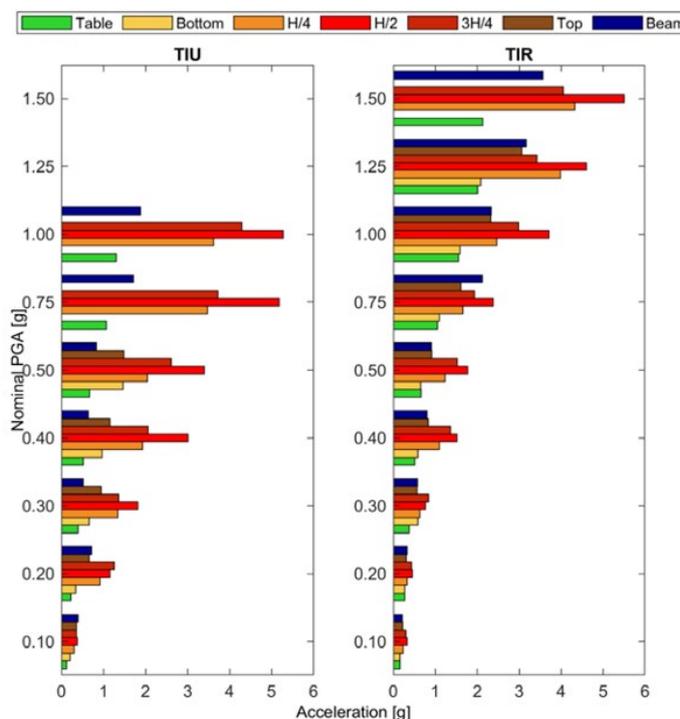
Figure 4.1. Comparison between the as-built and strengthened specimens in terms of absolute peak accelerations along the panel height, from bottom to top.

4.3 Amplifications

The following figures show the comparison between the as-built and strengthened specimens in terms of maximum positive and negative acceleration values and amplification factors at the seven measured heights, similarly to the previous section, for each test run.

As highlighted in the graphs, and consistent with the observations in the previous sections, the strengthened specimen exhibited significant reductions both in the magnitude of accelerations and in the amplification values along the height, compared to the as-built specimen.

In addition to the improved integrity provided by the strengthening, this reduction is mainly attributed to the higher stiffness and the resulting increase in the natural frequency of the strengthened panel. Conversely, the as-built specimen, characterized by lower stiffness and natural frequency, experienced higher demands and greater acceleration responses.

The difference became evident after the very first test, during which the as-built specimen developed visible cracking and began to show bending/arching deformation. This early onset of damage significantly influenced the dynamic response, leading to increased acceleration demand in subsequent runs.

The amplification was calculated by dividing each acceleration peak by the absolute maximum value recorded at the base / on the shake table. The initial tests, characterized by low PGA values and consequently low energy input, may not have triggered any structural mechanisms. Starting from the runs with nominal PGA = 0.30 g, the comparisons show that the as-built specimen (TIU) reached higher amplification values, also due to accumulated damage and the activation of the out-of-plane vertical arching mechanism.

The presence of the polymer membrane contributes to improving the infill wall response by preventing the activation of this mechanism; the amplification values recorded along the height remained almost constant within the range of tests performed on the strengthened specimen (TIR, up to nominal PGA = 1.00 g).

The direct comparison of the acceleration amplification factor was reported up to a nominal PGA = 1.00 g, since the as-built specimen (TIU) was unable to withstand further seismic motions beyond that point.
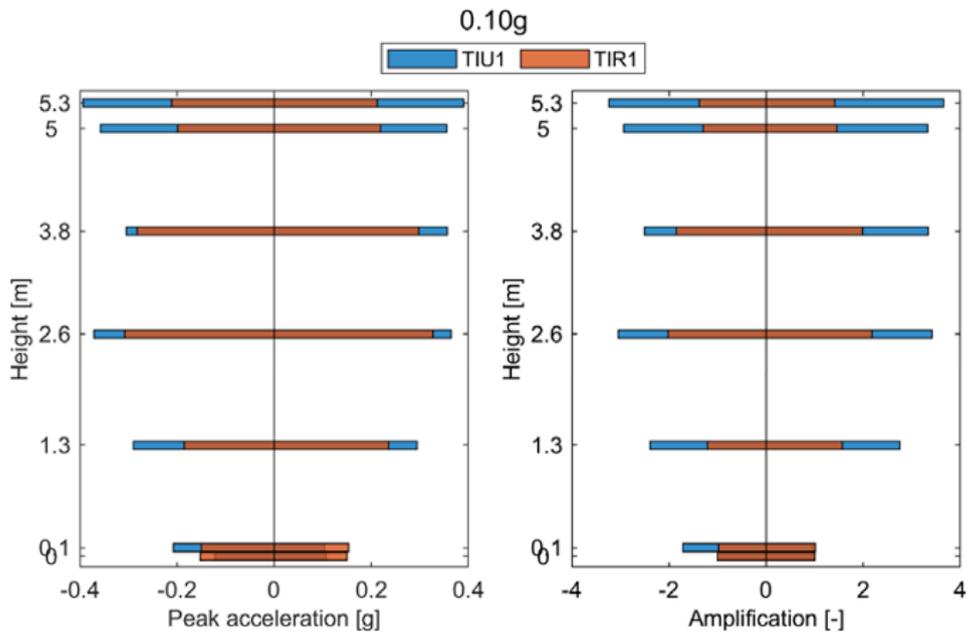
Figure 4.2. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.10 g.
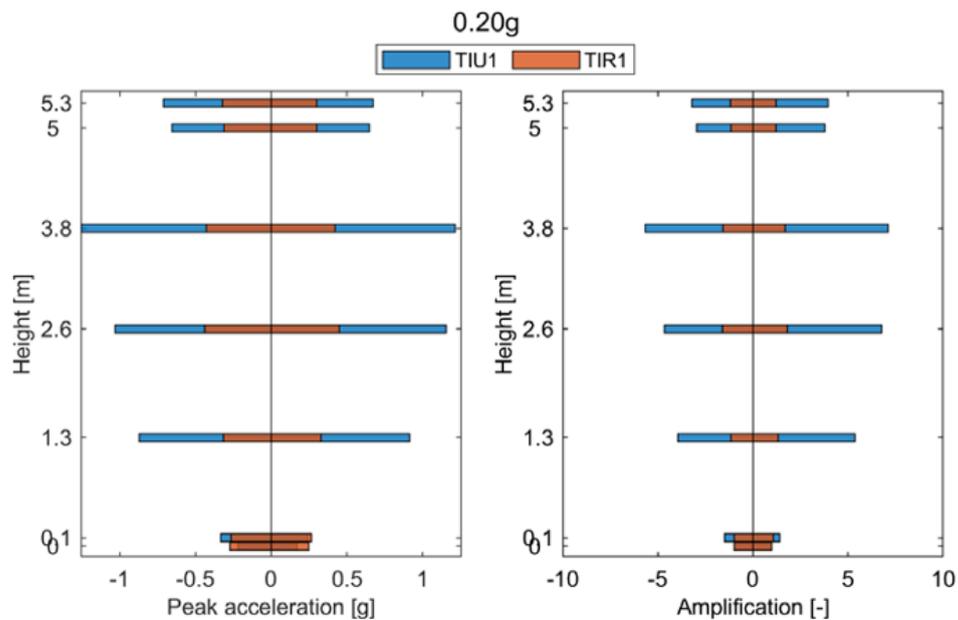

Figure 4.3. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.20 g.
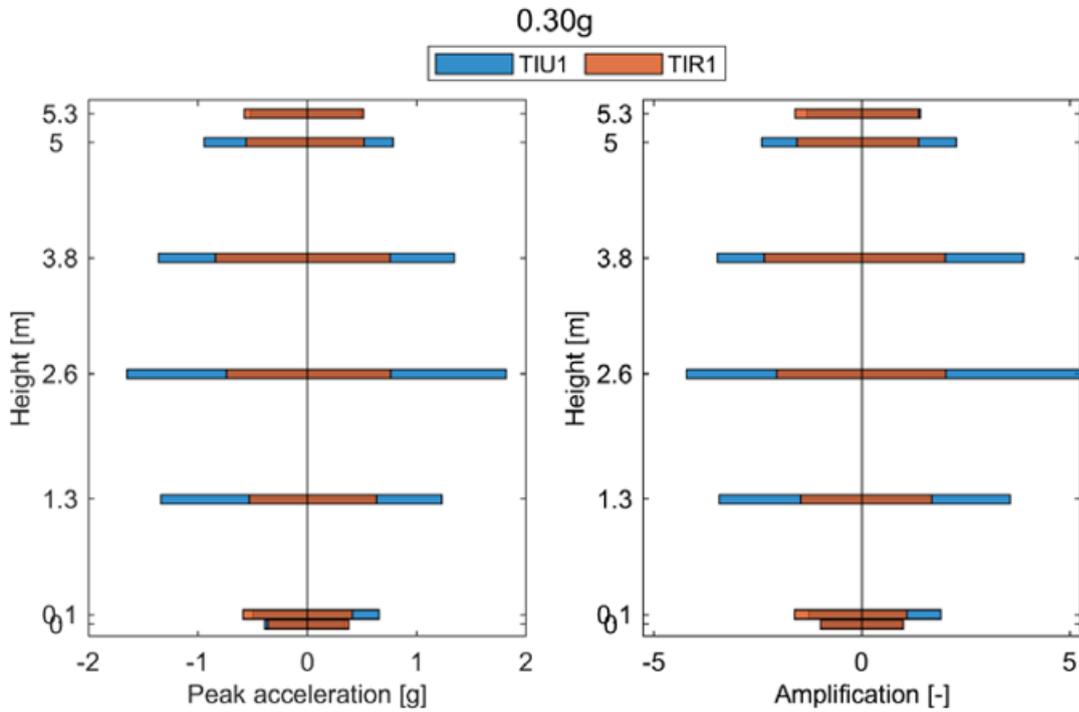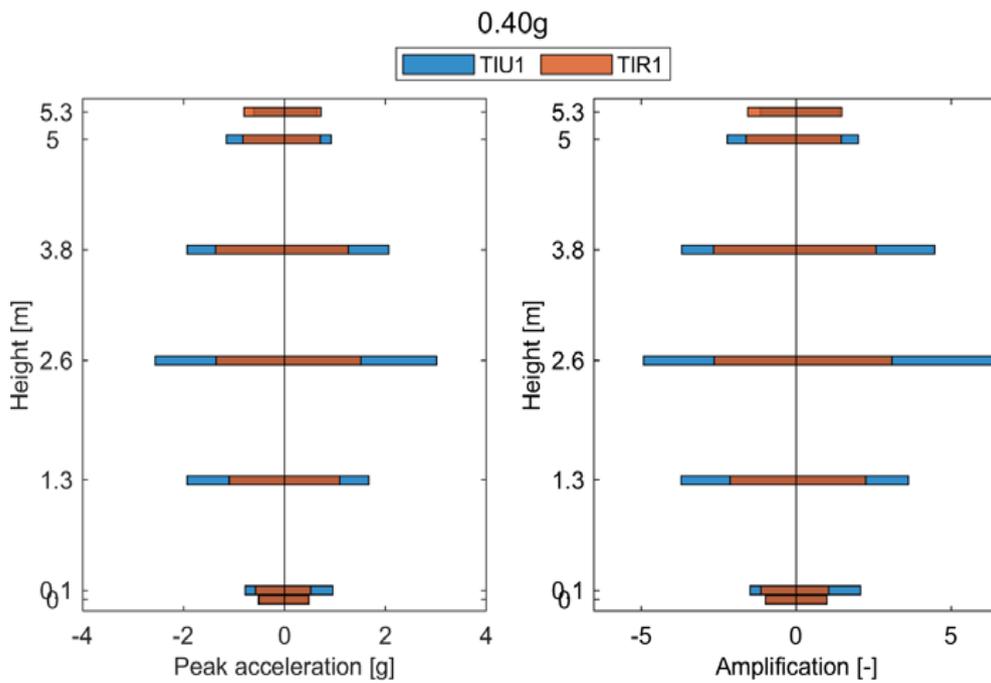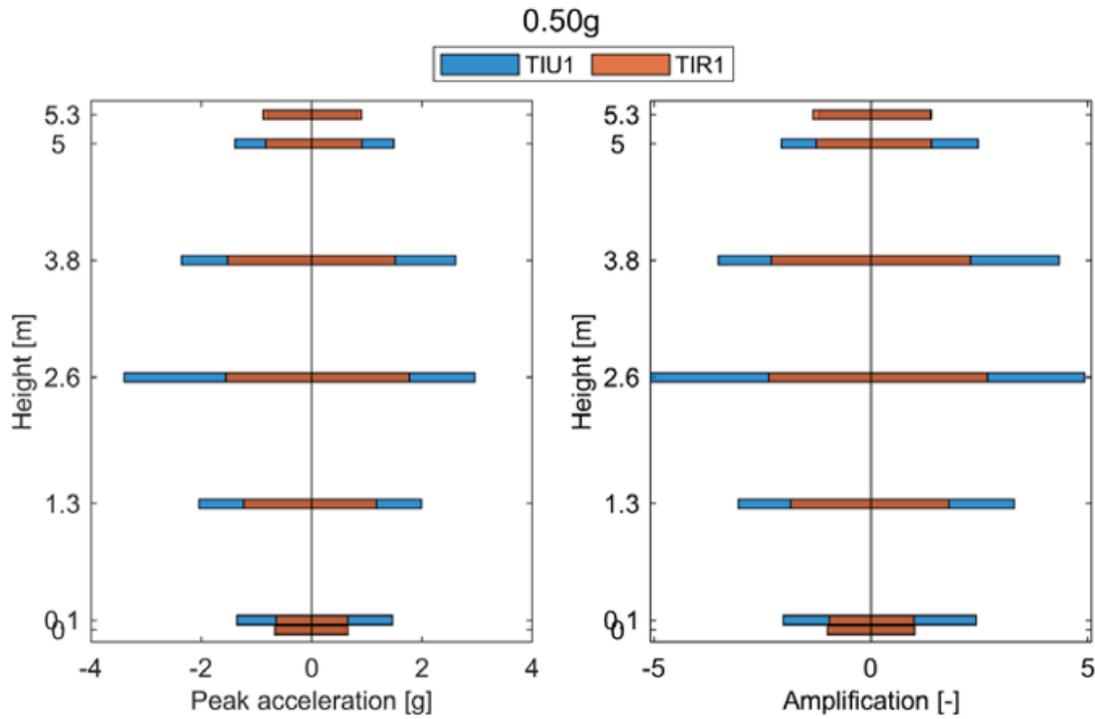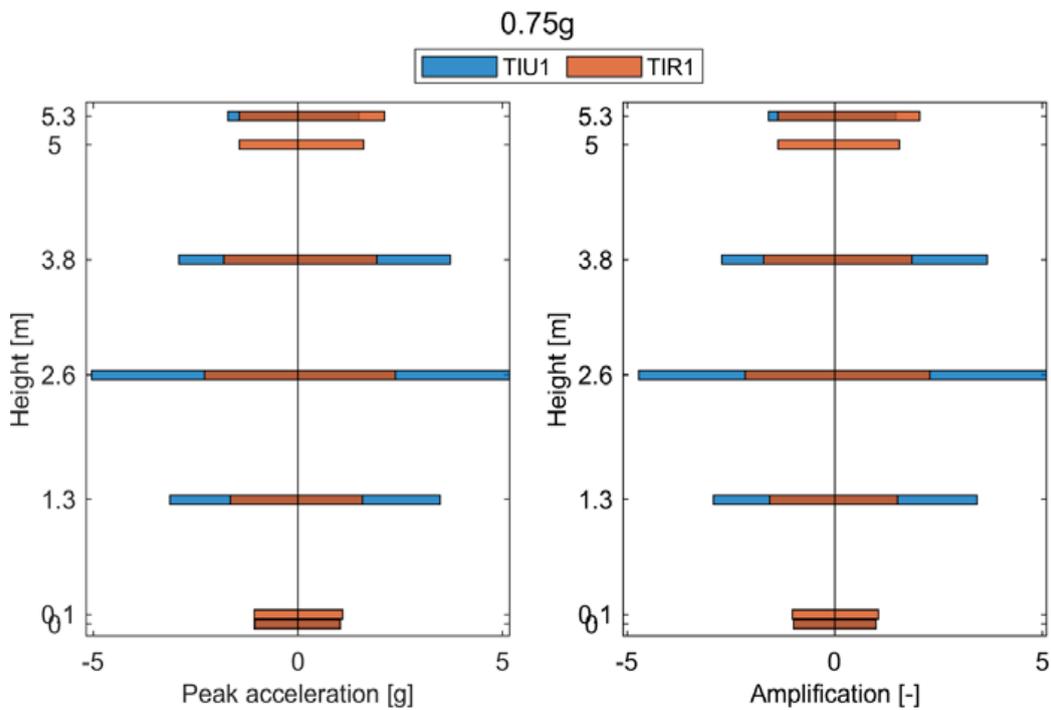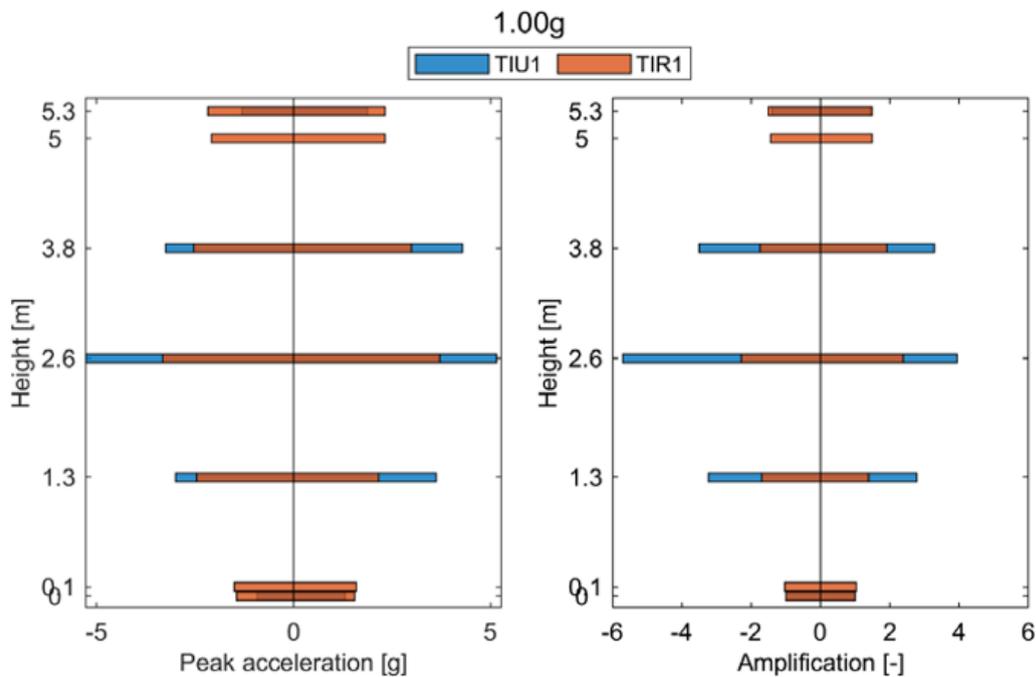
Figure 4.4. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.30 g.

Figure 4.5. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.40 g.

Figure 4.6. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.50 g.

Figure 4.7. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 0.75 g.

Figure 4.8. Comparison between the as-built and strengthened specimens in terms of maximum accelerations and amplification factors during the test with a nominal PGA of 1.00 g.

4.4 Displacements

In this section, unlike the previous ones focused on displacement profiles, the displacements corresponding to the maximum response state of the infill walls are presented, considering the instant at which the maximum displacement was recorded at each of the analyzed heights for both specimens.

The maximum displacements measured during each test are compared between the two specimens. Except for the first runs—where the accuracy of the results was affected by the very small displacement values—it was observed that both specimens exhibited a simple, nearly symmetric arch-shaped deformation pattern, consistent throughout the entire experimental campaign, with the maximum displacement generally located at mid-height of the panel.

Starting from the tests following the first run, the as-built specimen showed significantly larger maximum displacements compared to the strengthened one. This effect, as previously noted, is mainly due to the lower stiffness and natural frequency of the unreinforced wall, which made it more vulnerable to increasing deformations as the seismic input intensified.

Conversely, the higher stiffness of the strengthened specimen effectively limited the lateral displacements throughout the entire series of tests.
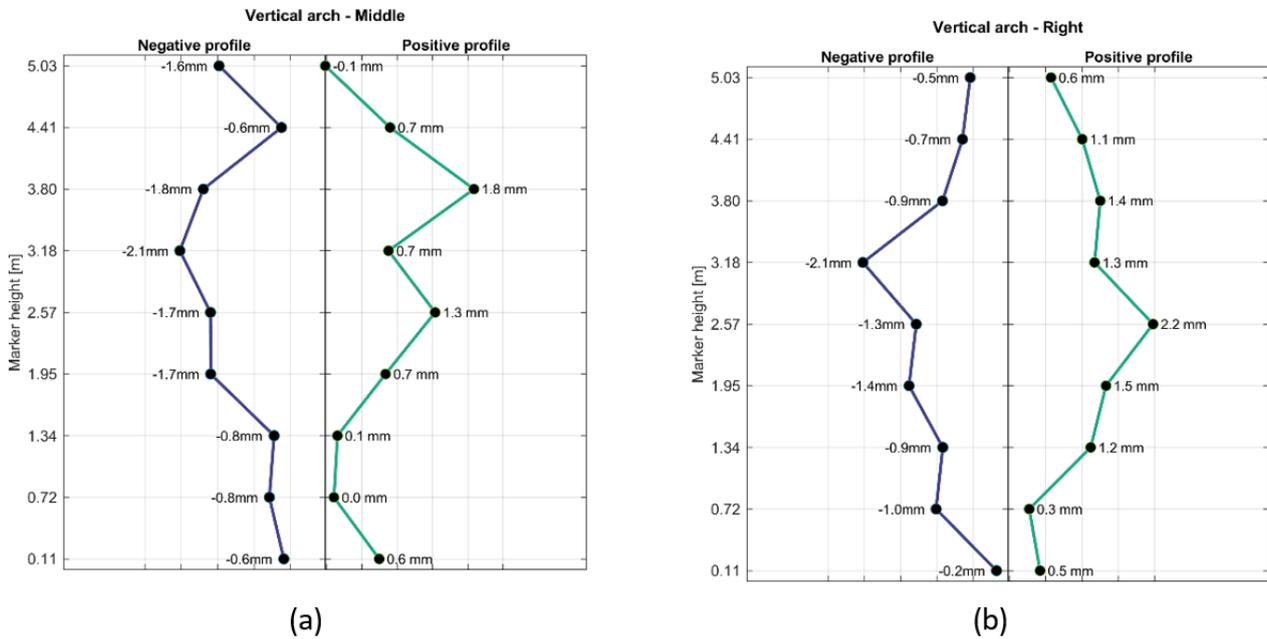
Figure 4.9. Comparison between the as-built and strengthened specimens in terms of displacement.
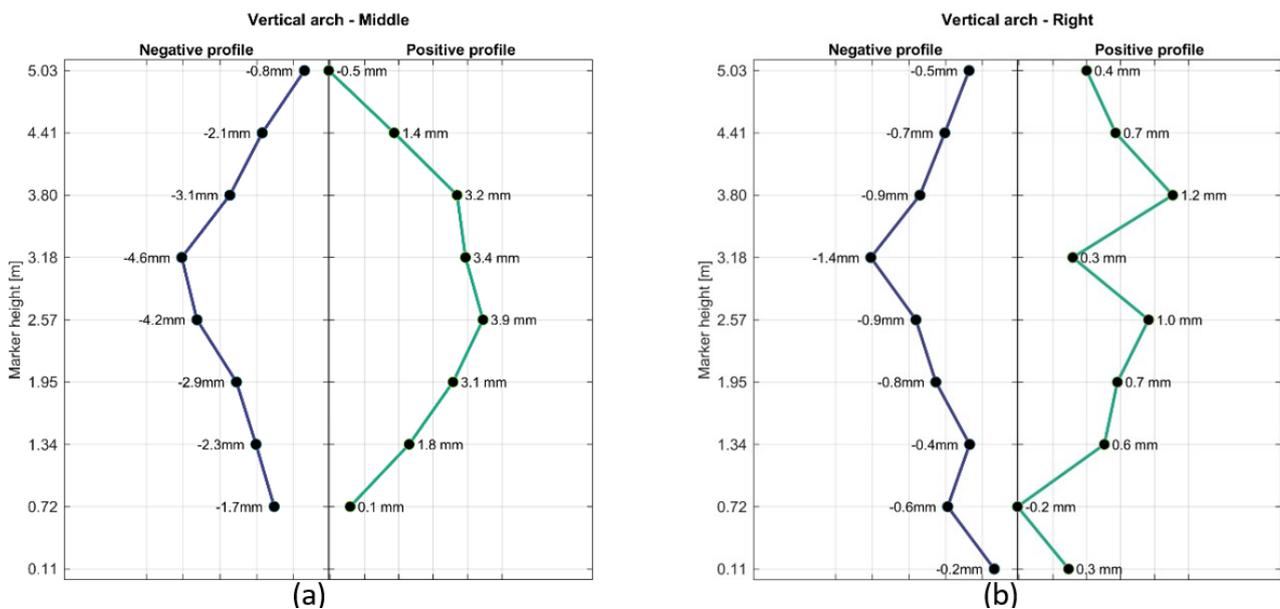

Figure 4.10. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 0.20 g.
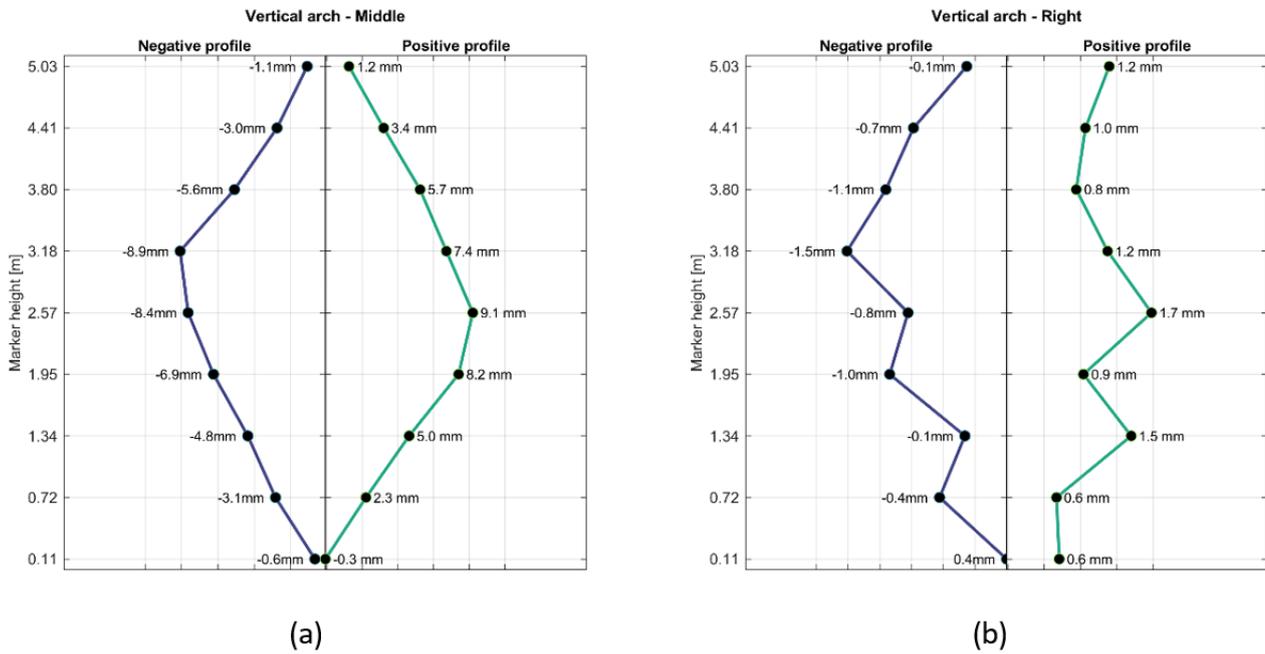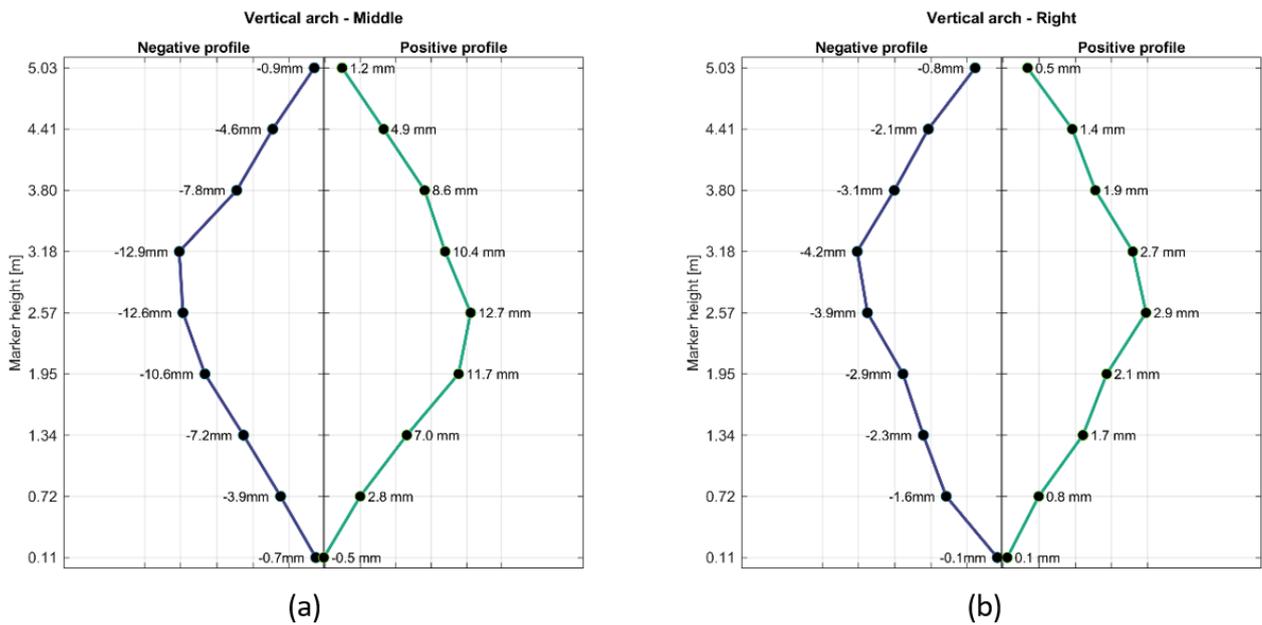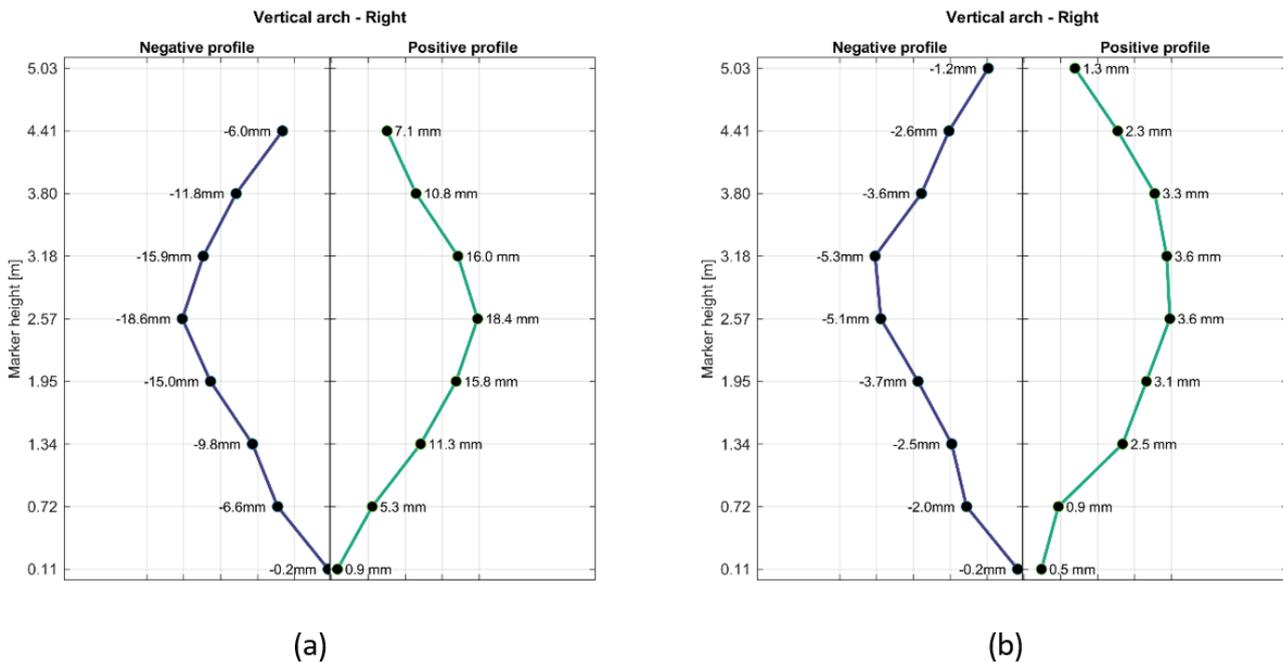
(a)   (b)

Figure 4.11. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 0.30 g.
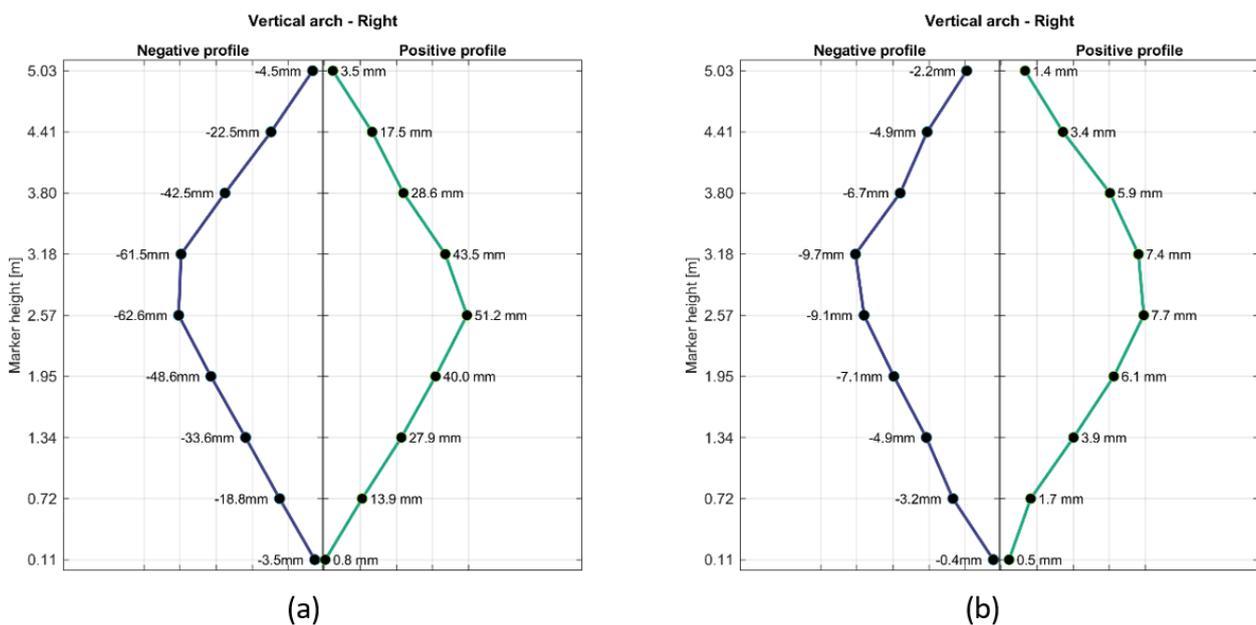

(a)   (b)

Figure 4.12. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 0.40 g.

Figure 4.13. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 0.50 g.

Figure 4.14. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 0.75 g.
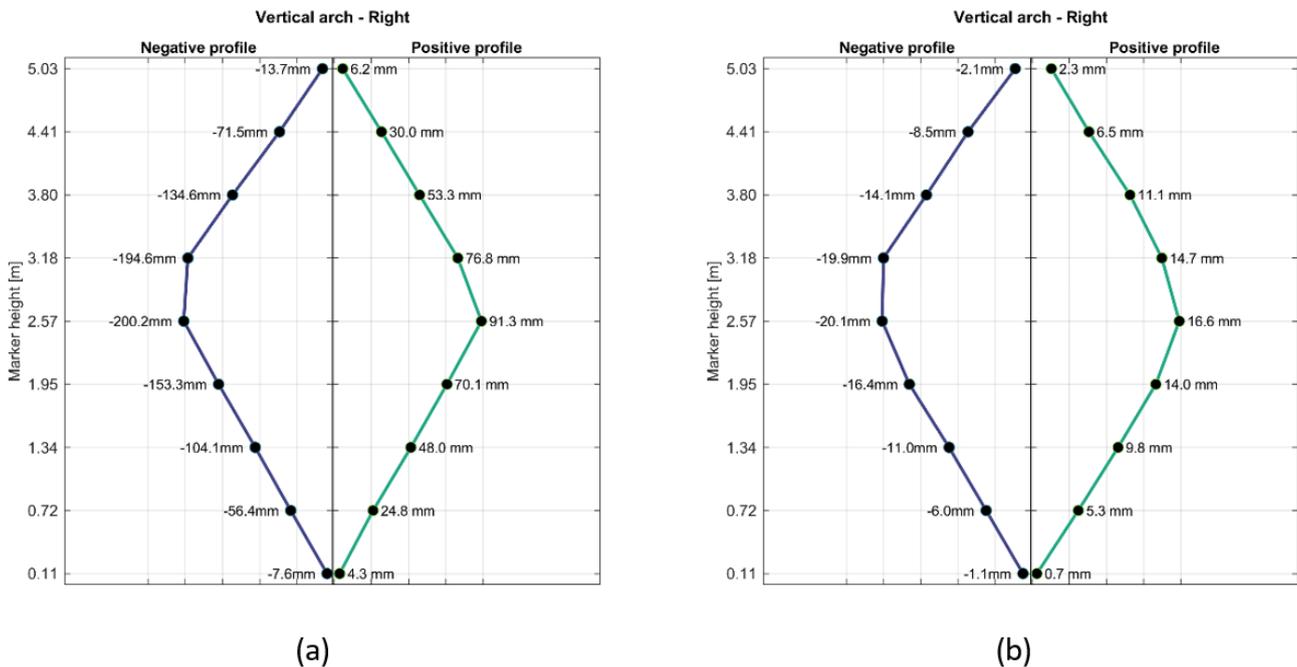
Figure 4.15. Comparison between the as-built and strengthened specimens in terms of displacement profile at the absolute maximum during the test with a nominal PGA of 1.00 g.

4.5 Force–Displacement Hysteresis Curves

The following figures compare the force–displacement hysteresis curves of the as-built and strengthened specimens, along with the corresponding envelope curves.

The force–displacement envelope curves show that the strengthened specimen exhibits a significant increase in both initial stiffness and peak strength, while the out-of-plane displacement at mid-height at the point of maximum load remains nearly unchanged.

In both curves, the post-peak behavior is represented by continuous lines based on the experimental results, whereas the dashed lines represent the idealized descending branch up to the ultimate displacement, corresponding to zero force at a displacement equal to the wall thickness.

However, the post-peak experimental points represented by continuous lines are affected by some uncertainty due to the sudden collapse of the specimens and the resulting abrupt loss of load-bearing capacity.
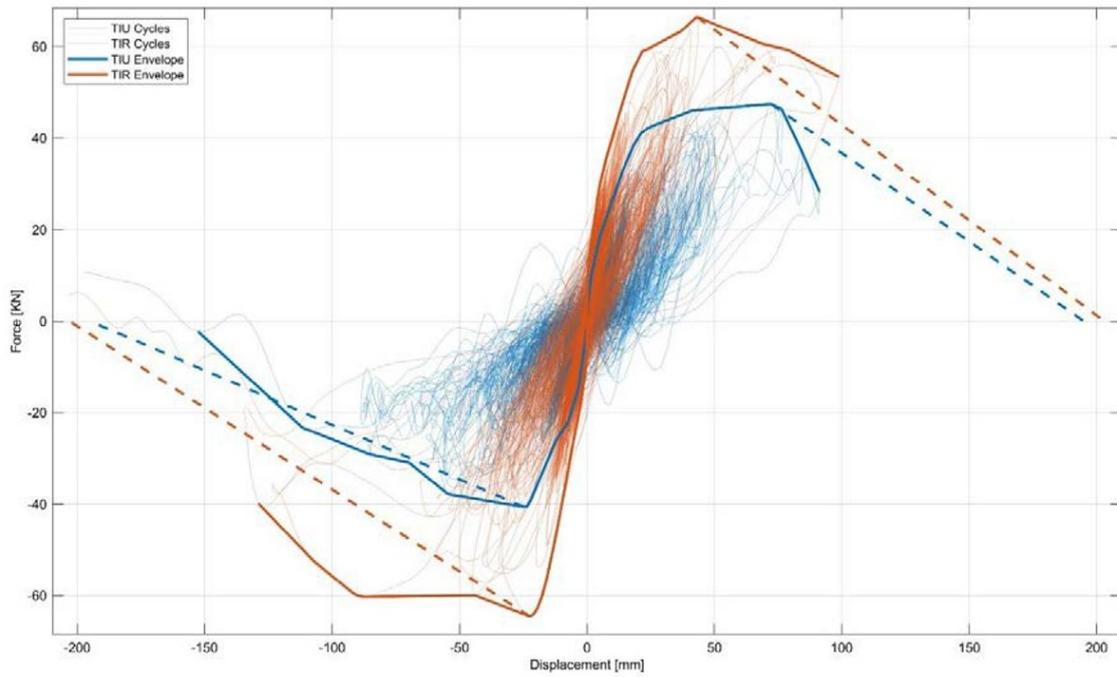
Figure 4.16. Comparison between the as-built and strengthened specimens in terms of force–displacement hysteresis curves for all tests.
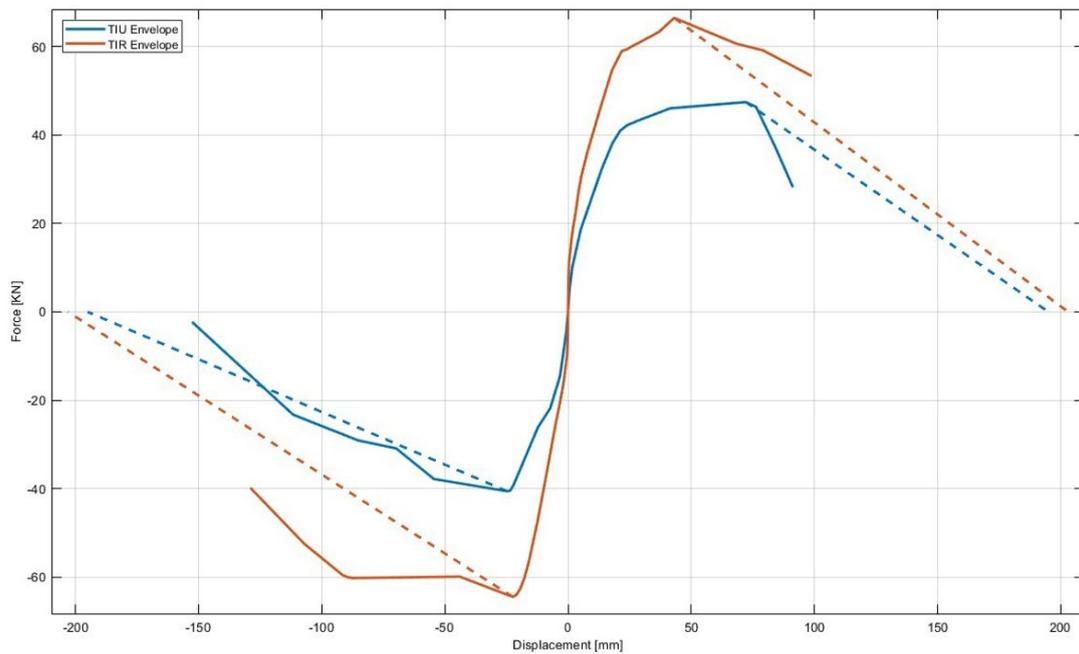

Figure 4.17. Comparison between the as-built and strengthened specimens in terms of the obtained envelope curve.

4.6 Strengthened Condition – Damage Evolution

It is important to note that the strengthened specimen did not exhibit any visible cracking on either side of the infill wall, as it was treated with Reblock 100, which coated both surfaces with an average thickness of approximately 4 mm.

This condition remained unchanged until the collapse of the panel, which occurred at a nominal PGA = 1.50 g, when the specimen displayed an out-of-plane rocking / vertical arching response characterized by the formation of "plastic hinges" along the edges and near mid-height, corresponding to the 14th row of blocks (slightly above mid-height).
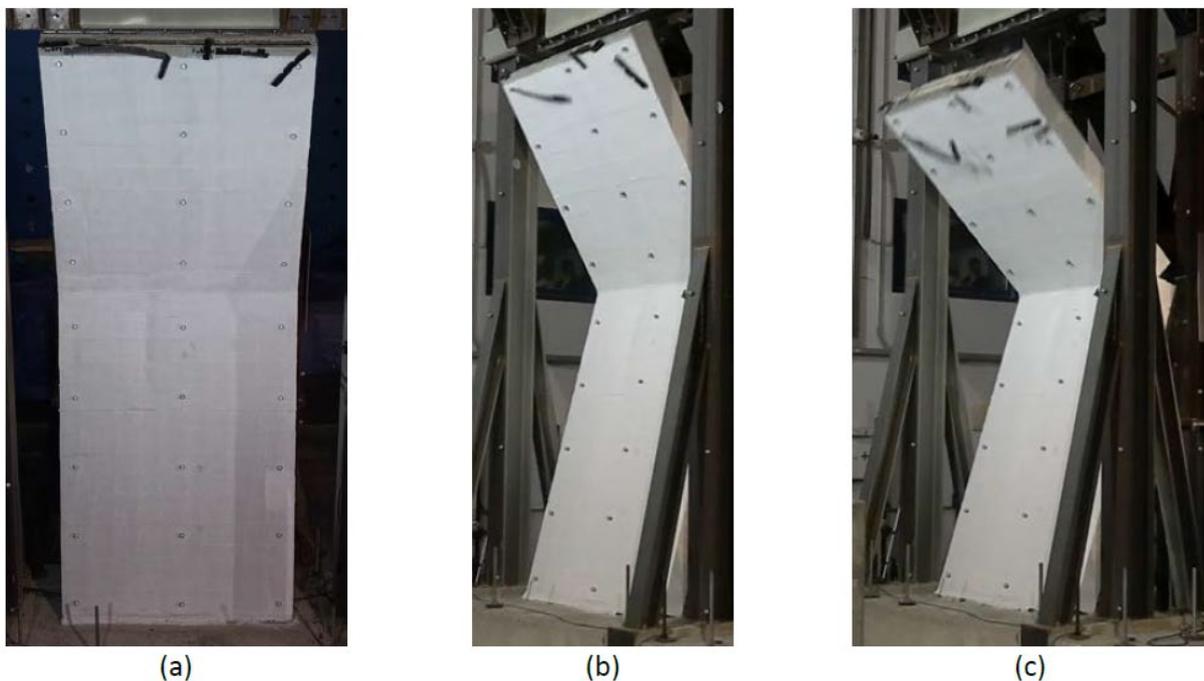

(a)   (b)   (c)

Figure 4.18. Evolution of the hinges at the base, mid-height, and top from images (a) to (c) during the out-of-plane collapse of the panel at a nominal PGA of 1.50 g in the negative direction.

CONCLUSION

The experimental results confirm the effectiveness of **REBLOCK 100 by Seriana®** as an innovative solution for the seismic strengthening of concrete block masonry, ensuring a significant improvement in performance in terms of safety, operational continuity, and structural reliability.

For further technical information, insights, or to evaluate the application of the system to your building, our team is at your disposal: info@serianaspa.it | +39 035 659 1371