# LARGE SCALE SHAKE TABLE TEST OF SLAB-TO-PIERS INTERACTION IN RC COUPLED WALLS

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## Abstract

A shake table test of the half-scale three-story specimen was conducted to study the slab-to-wall piers interaction. The specimen consisted of four rectangular walls coupled by three slabs. It was subjected to the series of seismic excitations with increasing intensity. In the last three tests the non-linear response of the slabs and wall piers was observed.

The considerable coupling between wall piers and floor slabs was observed. All slabs were fully activated. They were significantly damaged. The cracks were observed at their top and bottom surfaces over the entire width in between two rows of wall piers. The effective width of the slab was practically equal to its total width. Their damage was mostly flexural.

The coupling considerably influenced the response of the piers. It was considerably different from the response, which is typical for cantilever walls. The crack pattern was different from the cross-shaped damage pattern typical for cantilever walls. The cracks were opened on the outer side of the piers (opposite to the opening) and were propagated toward the opening when the seismic intensity was increased.

In the nonlinear range, the rocking of the wall pier subjected to tension was visible to the naked eye. In wall pier subjected to the compression the buckling of the longitudinal bars was observed at the outer edge of the wall.

The maximum drift of 1.1 % was observed within the last test. The fundamental period of vibration was increased from the initial 0.14 s to 0.32 sec.

The level of coupling was estimated considering the ratio of the overturning moment resisted by the flexural response of piers and the ratio resisted by the frame action of slabs (moment due to the axial forces in walls resulting from the accumulated shear in slabs). The share of the overturning moment resisted by the frame action induced by the slab was more than 50%. This result confirmed the indications of some experiments from the literature, that for certain building configurations only the slabs without beams can provide considerable coupling of wall piers. In such cases the common design, based on the assumptions that the walls respond as cantilever walls, can lead to significant underestimation of the demand in piers. This can further lead either to brittle shear failure of walls or to their failure caused by the buckling of the longitudinal bars induced by large compression stresses, which were underestimated in the design.

*Keywords: RC coupled walls, Shake table test, Floor-to-piers interaction, Large scale experiment*

## 1. Introduction

Several dramatic failures during the recent earthquakes in Chile in 2010 [1], [2] and in New Zealand in 2011 [3] increased research interest in the seismic response, strength and deformation capacity of RC structural walls. In several cases, their damage was clearly linked to the demand in piers, which was completely different from the one, which had been predicted in design. An unexpected damage of the wall piers in tension, compression and shear was observed. Major factor contributing to the wrong assessment was the inability of present design procedures and numerical models to simulate adequate coupling effect in walls induced by the floor system through coupling beams and slabs or even by the slab alone.

In the conventional design of RC walls, it is typically assumed that the slab alone (without coupling beams) is not strong enough to provide notable coupling of wall piers. When the walls are connected only by the slabs, they are typically design as cantilever walls. The walls are connected by the in-plane rigid diaphragm considering large axial strength and stiffness of the slabs, while the bending strength and stiffness of the slabs and their influence to the overall response are typically neglected.

There are some experimental evidences that in certain type of building this assumption can be quite arguable. The problem was identified and discussed in details more than thirty years ago, after the well-known full-scale tests of 7-storey RC frame-wall structure [4]. In this test, yielding of the boundary column of the wall triggered large uplift of the slab and connecting beams, which resulted in extensive plastification of the floor system in both directions.

The experiments [5], [6] clearly demonstrated the importance of the flexural response of the slab to the overall behavior of RC coupled walls. For example, in the experiment [5] (see Fig. 1a) slabs were supported on one side by the wall and on the other side by "gravity columns". There was a small gap intended to assure unrestricted vertical movement of the slab at the location of gravity columns. In spite of that, the gravity columns restricted the upward movement of the slabs, causing the 3D interaction between the tested wall and the gravity columns by means of the flexural deformations of the slabs.

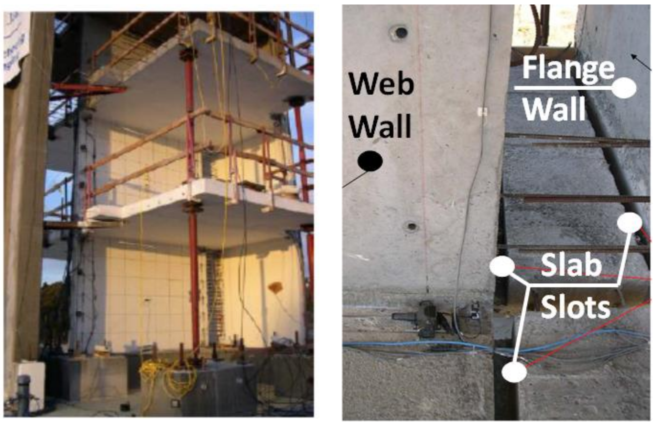



Fig. 1 – Cantilever RC wall tested at shake table at UCSD:
a) the wall and "gravity columns", used to support slab and provide the stability in the direction perpendicular to the wall plane b) the slab slots (courtesy Panagiotou MM et al. [5])

In the same experiment it was demonstrated that relatively thin slab can also provide considerable coupling between wall piers. The analyzed rectangular wall was connected by the perpendicular stabilizing wall only by slabs in order to avoid their interaction. To additionally minimize the coupling effect, slabs were slotted at the connection with the walls. They were only 5 cm thick at the slot (see Fig. 1b). A considerable shear forces were generated along the whole length of the slots, resulting in the substantial increase of the axial force in the tested wall. The bending moments and shear forces in the wall were also increased, due to the induced axial forces.



The interaction between the RC walls and slabs was recently studied within the SERA-TA project "Influence of the floor-to-wall interaction on the seismic response of coupled wall systems". The large-scale shake table test was performed at the shake table at IZIIS, North Macedonia. The main observations are summarized in the following sections: a) in Section 2 the main issues of the floor-to-wall piers interaction are summarized, b) the tested specimen is presented in Section 3, c) the main observations about the response are summarized in Section 4, including the estimated level of coupling provided by the slab.

## 2. The main issues of the floor-to-wall piers interaction

The interaction between piers and the floor system significantly depends on the ability of the floor system to couple wall piers (see Fig. 2). If the coupling is weak, the response is relatively simple and depends mainly on the bending response of piers (Fig. 2a). If the level of coupling is important the response mechanism becomes quite complex (see Fig. 2b).

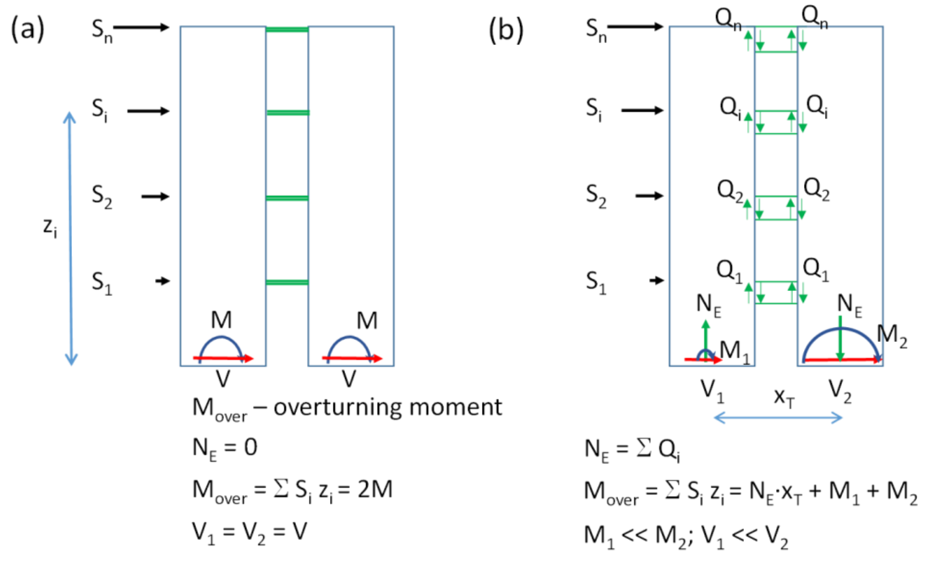



Fig. 2 – Resisting mechanisms: a) cantilever walls (very weak coupling), b) coupled walls (significant coupling of wall piers)

The coupling depends on the strength and stiffness of the coupling elements (typically beams and slabs), which determines the variations of the axial forces in wall piers. The fluctuating axial forces further determine the variations of the stiffness and strength of piers and the redistribution of forces in the coupled walls and coupling elements. According to limited experimental results that can be found in the literature [7, 8] the redistribution of demand between wall piers in the nonlinear range can double bending moments and shear forces in piers, which are subjected to compression axial forces (compared to the linear range). This considerable increase of the demand is typically not considered in the design, which is mostly based on the elastic analysis. Consequently, considerable damage and even failure can be expected in this type of buildings when they are subjected to strong earthquakes.

The seismic response of coupled walls significantly depends on the strength of the floor system. The (over)strength of the coupling beams and slabs and their failure modes cannot be reliably quantified using existing design procedures. This has been confirmed by shake table experiments [7], where the strength of coupling beams was considerably larger than that predicted before the experiments. Moreover, doubts have occurred about the traditional assumption that flexural response of the slab alone can be neglected in the design of (cantilever) walls. There has been clear evidence that even slab alone can cause an excessive over-strength of the coupling system [4, 5], which may further cause unacceptable brittle types of failure in wall piers.



## 3. Description of the specimen, excitations and instrumentation

### 3.1 The geometry of the specimen

A shake table test of the half-scale three-story specimen was conducted. The specimen consisted of four rectangular walls coupled by three slabs (see Fig. 3). In order to get as realistic as possible information about the slabs-to-walls interaction, the maximum possible size of the specimen was selected, considering the limitations of the shake table regarding the overturning moment (about 500 kNm). Scale factors, which were considered in the design of the specimen, are summarized in Table 1.

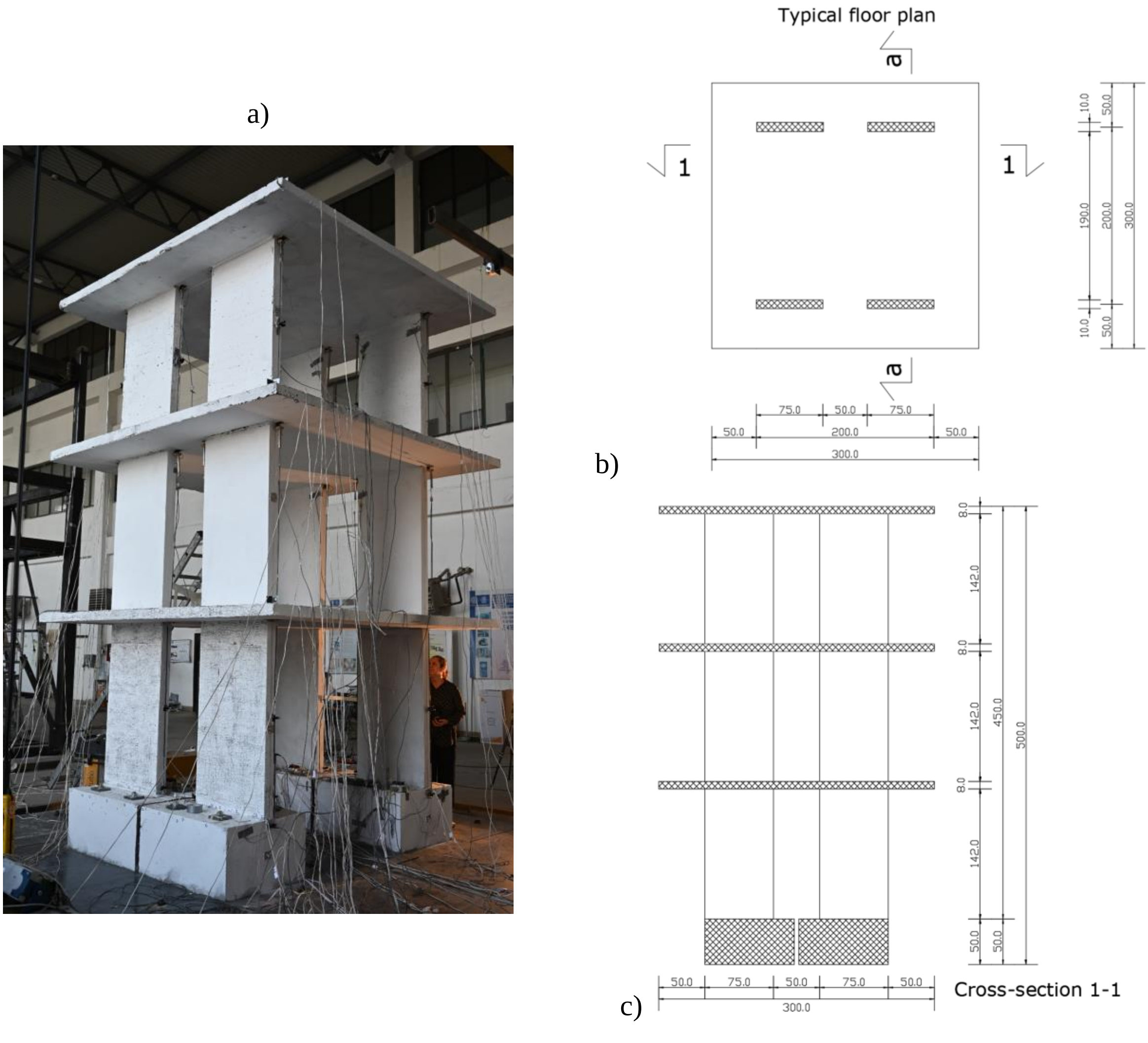



Fig. 3 – The specimen and its geometry: a) the appearance, b) floor plan, c) side view

The main goal of the experiment was to obtain the information about the varying floor-to-wall interaction at different levels of the response, in particular in the non-linear range. Thus, the proper balance between the realistic size (strength) of the structural elements and the limitations of the shake table was necessary to find. The selected height of the walls' cross-section (75 cm) enabled the yielding of the walls to be achieved when they are subjected to the maximum possible intensity of the seismic load limited by the performances of the shake table. At the same time, this dimension was realistic enough considering the dimensions of the realistic walls. The thickness of the walls (10 cm) was selected considering the typical thickness of structural walls in Slovenian design practice (20 cm). The aspect ratio of the walls' cross section was 7.5. The aspect ratio of the wall (height of the wall/ height of the cross-section) was 6. The clear distance between walls piers (see Fig. 3 c)) was 50 cm, which correspond to the 100 cm opening in the prototype (e.g. the opening for the doors).

The size of the slabs (3 m x 3 m) was defined following typical tributary area for walls in RC wall buildings in Slovenia (6 m x 6 m). The thickness of the specimen's slabs (8 cm) was defined considering the typical thickness of the slab in the prototype buildings (16 cm).

Table 1 – Scale factors

| Variable | Scale Factor | Value of the Scale Factor |
|---|---|---|
| Length | $S_L$ | 2 |
| Area | $S_L^2$ | 4 |
| Volume | $S_L^3$ | 8 |
| Moment of inertia | $S_L^4$ | 16 |
| Mass | $S_M$ | 10 |
| Stress | $S_\sigma$ | 1 |
| Strain | $1$ | 1 |
| Modulus of elasticity | $1$ | 1 |
| Force | $S_L^2$ | 4 |
| Moment | $S_L^3$ | 8 |
| Acceleration | $S_\sigma \cdot S_L^2 / S_M$ | 1·4/10 = 0.4 |
| Time | $\sqrt{S_M/S_L/S_\sigma}$ | √(10/1/2) = 2.24 |

The total mass of the specimen without foundations was 8.2 t. In general, in most of the shake table tests, additional masses are typically provided to obtain the realistic demand. For this reason, steel ingots are often installed at the slabs. In the studied case, this was not an option, since the ingots would affect the main properties of the floors (strength and stiffness), which have crucial influence to their interaction with wall piers. Instead of the added masses, the time and the accelerations were properly scaled (see Table 1) to obtain the realistic demand.

3.2 Material properties and the reinforcement

The strength of the used concrete was in average 26 MPa and 27.5 MPa for walls and slabs respectively.

In walls the minimum flexural (longitudinal) reinforcement was provided. Originally, it was planned to use 12 ribbed bars of diameter 6 mm. Since only the brittle bars of such diameter were available on the market, the walls were finally reinforced by 12 ductile plain bars of diameter 8 mm. The yielding and ultimate stress of the corresponding steel was 300 MPa and 420 MPa, respectively. The shear reinforcement $\phi$6 mm/7.5 cm was provided over the entire height of the walls.

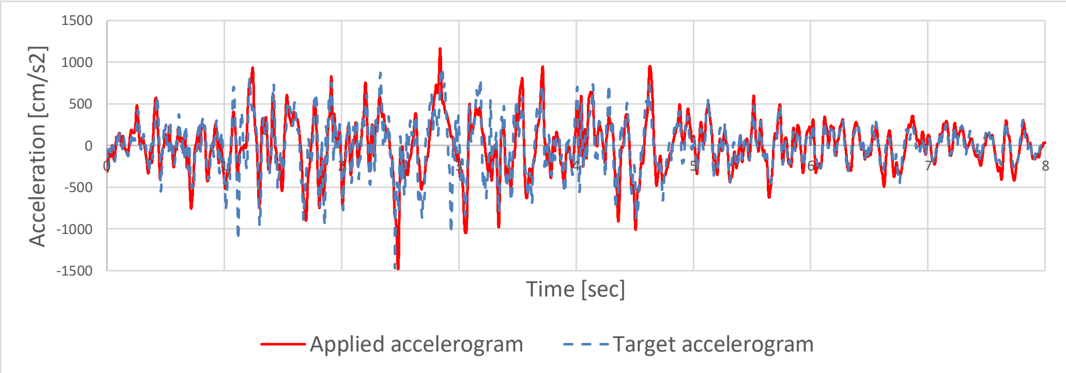
The slabs were reinforced by two reinforcing meshes Q-131, providing 1,31 cm$^2$/m for the top and the bottom reinforcing layers. The yielding and the ultimate stress of the corresponding steel were 500 MPa and 560 MPa, respectively.

3.3 Seismic excitation

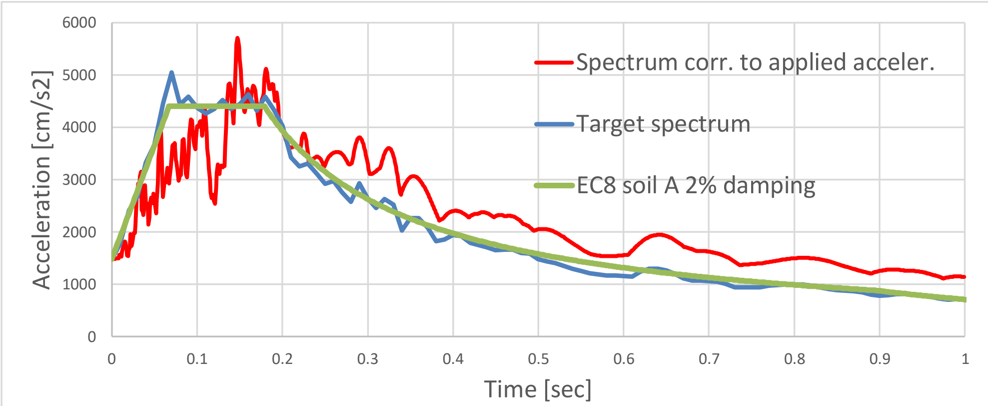
The shake table was excited by an artificial accelerogram (see Fig. 4a), which was generated modifying an accelerogram Petrovac N-S registered during the 1979 Montenegro earthquake. This accelerogram was modified to match the EC8 acceleration spectrum corresponding to soil site type A and 2% damping. The target accelerogram and the accelerogram actually applied during the tests as well as the corresponding acceleration spectra are presented in Fig. 4a and 4b, respectively.






a)




b)

Fig. 4 – Seismic excitation: a) target and applied accelerogram, b) corresponding acceleration spectra for PGA = 1,5 g (Note: The time axis is scaled considering time scale factor from Table 1).

A series of uniaxial tests was preformed, with gradually increasing the intensity of the seismic excitation in the direction of walls (N-S – see Fig. 5). All runs are summarized in Table 2. The testing was concluded when the displacement capacity of shake table was exhausted (12 cm). In between the tests, the periods/frequencies of the structure were measured. The measured values were 0.14 sec, 0.20 sec, 0.32 sec, and 0.32 sec before the first test R010, after R060(2), after R150(1), and after R150(2), respectively.

Table 2 – The list of the performed tests

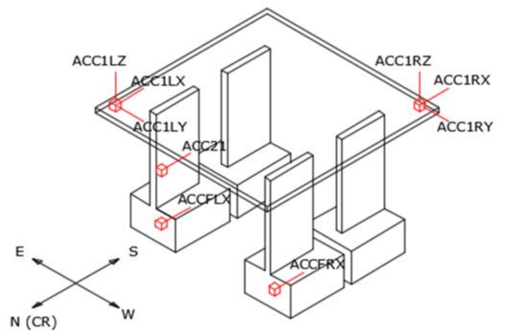
| Test | Maximum acceleration at the shake table |
|---|---|
| R010 | 0.1 g |
| R020 | 0.2 g |
| R030 | 0.3 g |
| R050 | 0.5 g |
| R060(1) | 0.6 g |
| R060(2) | 0.6 g |
| R080 | 0.8 g |
| R090 | 0.9 g |
| R120 | 1.2 g |
| R150(1) | 1.5 g |
| R150(2) | 1.5 g |



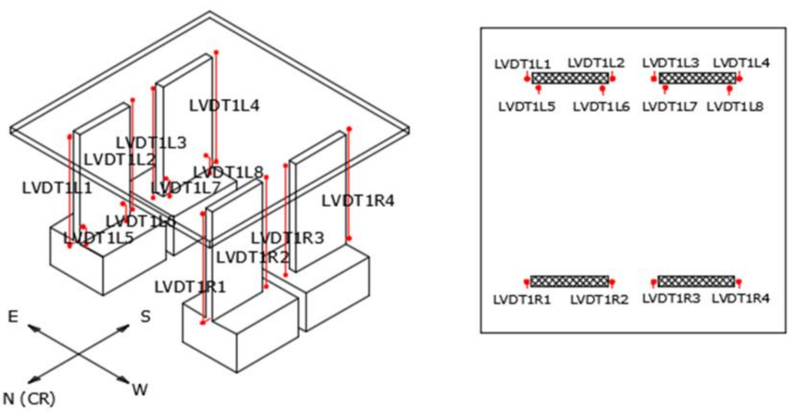
 

3.4 Instrumentation

The instrumentation is summarized in Fig. 5




a) Accelerometers were installed at all slabs and at the foundation level (only the scheme of the first story is presented)




b) LVDT'S were used to measure relative vertical displacements (deformations) along all stories and at the bottom of the walls (only the first story is presented)

d) Optical measurements of deformations were performed at outer faces of bottom story of all walls

Fig. 5 – An overview of the instrumentation

## 4. Response of the tested specimen

4.1 Observed response

The response of the tested building was essentially elastic up to the test R120. The first cracks were observed at the bottom of the wall piers and in the 1st story slab near the joints with the walls, after the test R030. When the seismic intensity was increased, the cracks appeared also in the second and the third slab. The cracks in the slabs were first limited to the area near the joints with the walls. When the seismic intensity was increased, they were gradually expanded to the whole width of the slabs between the two rows of wall piers (see Fig. 6). The cracks were clearly visible at the top and at the bottom surfaces of the slabs.



a)  b) 

Fig. 6 – Cracks were formed at a) the top and b) at the bottom surfaces of the slabs, all over their width between two rows of wall piers

The damage in the wall piers was initiated at the very bottom cross-section near the foundations. Later on, additional cracks were gradually formed up to the approximately 100 cm from the foundation level (see Fig. 7a). The cracks were initiated at the outer edges of each wall pier. When the seismic intensity was increased, they were extended toward inner edges (see Fig. 7). The crack pattern was considerably different from the cross-shaped damage pattern, which is typical for cantilever walls (compare the crack patterns, presented in Fig. 7a and 7b).

a) 

Fig. 7 - a) Cracks, which were observed in the wall piers,

b) 

Fig. 7 – b) Crack pattern typical for cantilever walls (courtesy of Tran and Wallace [9])

In the test R120 the response of the building entered the nonlinear range. The cracks in the slabs were spread over the full width of the slab in between the two rows of the wall piers (see Fig. 6). The width of the cracks in the slabs was considerably increased. The yielding of the reinforcement in the slabs was achieved. The effective width of slabs was equal to their total width. The flexural strength of slabs was fully activated, generating considerable axial forces in wall piers (see Fig. 2). The frame effect, generated by the slabs was considerable (see also the discussion in section 4.3).



The response of two wall piers located at the same side of the specimen was considerably different. This is evident from Fig. 8a, where the response (obtained with optical measurements) of two piers is presented. In the left pier, where the tensile axial force was generated, the considerable cracks were formed approximately up to the 1m from the foundation level (see the orange areas surrounded by red circle, which indicate cracks). In the right wall pier, which was subjected to compression, the damage was located mostly at the bottom of the wall.

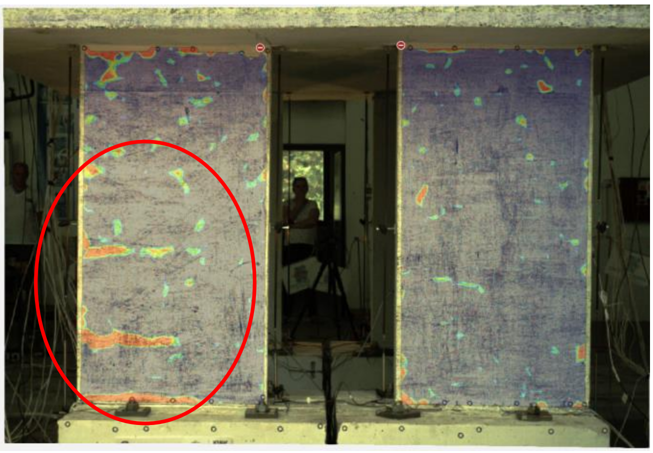
a)  b) 

Fig. 8 – a) Response of two piers was considerably different, b) Buckling of the longitudinal bars was observed in the outer edge of one pier

In the last two tests (R150(1) and R150(2)), where the nonlinear deformations were noticeable, the differences in the response of two piers were visible to the naked eye. Considerable rocking of the wall subjected to tension was observed. In the last test, the buckling of the longitudinal reinforcement at the outer edge of one of the piers was observed (see Fig. 8b) indicating that this pier was subjected to relatively large compression stresses.

4.2 Global parameters of the response

The envelopes of horizontal story accelerations and the envelopes of horizontal displacements in the direction of the seismic excitation (N-S see Fig. 5) are presented in Figure 9. The presented accelerations are the average values of the accelerations, measured at two stations (see Fig. 5a) at each slab. The hysteretic response throughout all tests, expressed in terms of displacements and the base shear, is presented in Fig. 10. The base shear is estimated taking into account the measured average story accelerations.

Maximum acceleration of 3.4 g was registered at the top of the building at test R150(2). It corresponds to the acceleration excitation of the shake table of 1.5 g. Note, however, that seismic excitation of 1.5 g was applied only in one single time step (see Fig. 4a). The most of the local maximums corresponded to the acceleration excitation of about 1 g. This acceleration corresponds to acceleration of 0.4 g in the prototype structure (see Table 1).

In both directions (N-S and S-N) the maximum displacement of 53 mm was obtained at the top of the building during the last test R150(2). This value corresponds to 1.1 % drift.

The top displacement - base-shear relationship, presented in Fig. 10c, confirms the visual observations from the experiment, that the structure entered the nonlinear range in the test R120. The gradually decreased stiffness of the structure (see Fig. 10 a – c), is in a good agreement with the measured increasing periods of vibrations (see section 3.3).



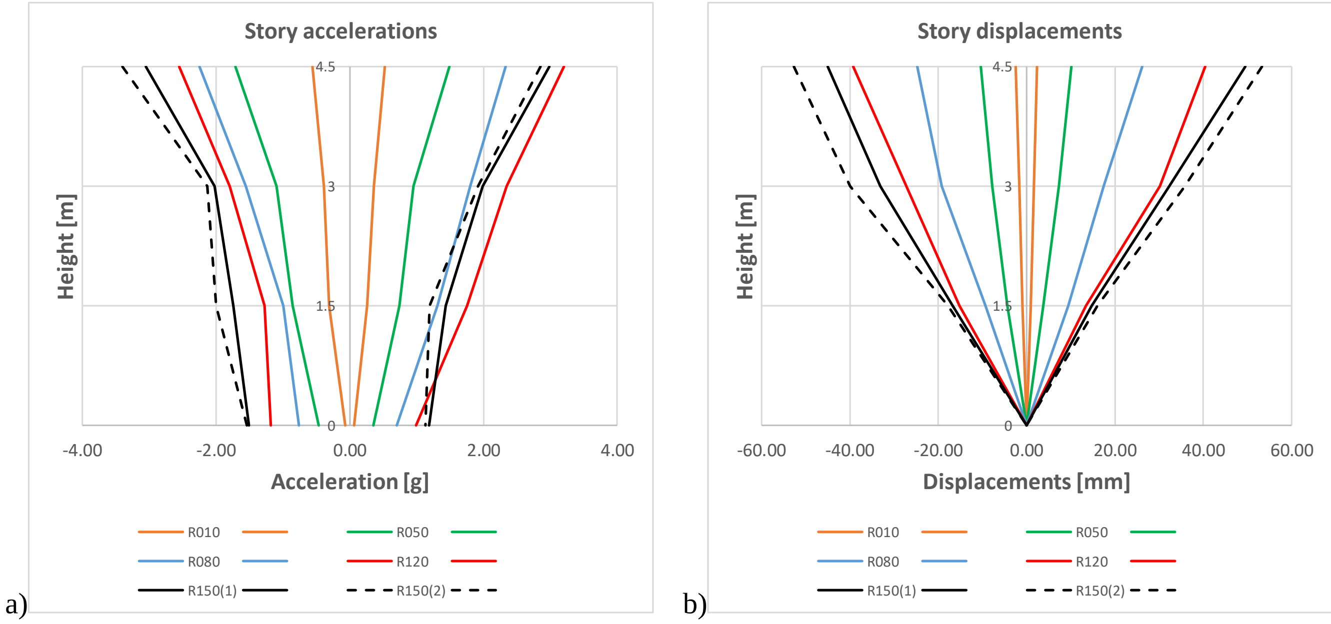



Fig. 9 – Envelopes of a) horizontal story accelerations, b) horizontal story displacements in the direction of excitation

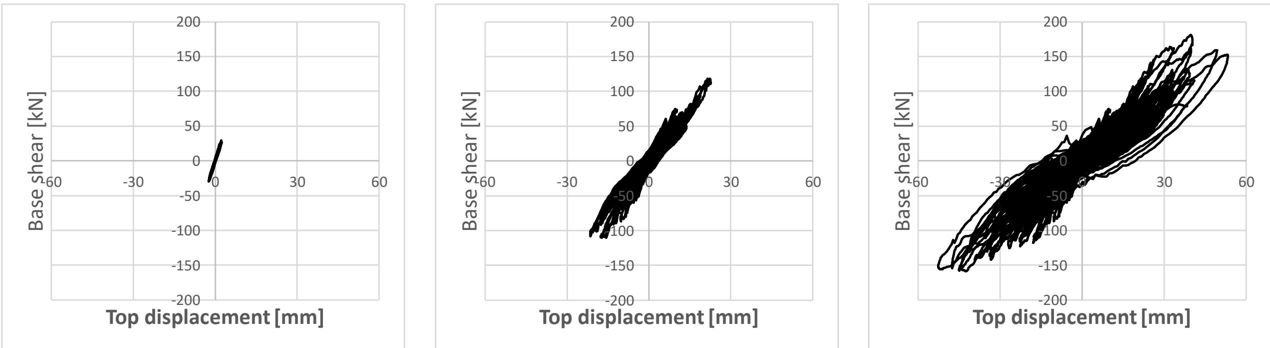



Fig. 10 – Hysteretic response at three different stages of testing: a) after R010, b) after R060(2) and c) after the last test R150(2)

### 4.2 Estimated level of coupling

The level of coupling was estimated considering the ratio of the overturning moment resisted by the flexural response of piers and the share resisted by the frame action of slabs (moment due to the axial forces in walls resulting from the accumulated shear in slabs – see section 2).

The level of coupling was analyzed considering the response of the two wall piers at the east side of the tested building (see Fig. 5), which was damaged more than the west part. The representative example of this analysis is provided in the following paragraphs, considering one of the peak excitations during the last test R150(2) in which yielding of wall piers was observed and their flexural capacity was achieved.

The overturning moment was estimated based on the inertial forces, calculated at all stories taking into account the accelerations measured at the east side of slabs (see Fig. 5a) and the tributary mass (half of the mass of the tested specimen). The bending moments at the level of foundations, caused by these forces, were summed to obtain the total overturning moment.

At the beginning of the analysis, the axial forces in piers were not known. Thus their flexural capacity was estimated taking into account the axial force caused by the gravity load $N_g$ = 20 kN per wall pier. The corresponding total flexural capacity of both piers was $M_{FC}$ = 140 kNm (70 kNm per wall pier).



The overturning moment $M_{over}$ was 290 kNm. Considering the flexural capacity of piers ($M_{FC}$ = 140 kNm) the part of the overturning moment resisted by the frame action was defined as $M_{FA}$ = 290 – 140 = 150 kNm.

To obtain the axial forces $N_E$ in wall piers caused by the seismic excitation, $M_{FA}$ was divided by the axial distance of wall piers (1.25m). In this way, $N_E$ was estimated to 120 kN. In one wall pier, this force was tensile in the other compressive (see Fig 2).

In the next step $N_E$ and Ng were summed to obtain the total axial forces in piers (due to the gravity and the seismic load). In the pier subjected to tension, the axial force was $N_t$ = 100 kN (tensile force). In pier subjected to compression the axial force was $N_c$ = 140 kN (compressive force).

Taking into account forces $N_t$ and $N_c$, the flexural capacity of each pier was defined. It was 30 kNm and 105 kNm, in the pier subjected to tension and compression, respectively. Thus, the total flexural capacity of both piers was $M_{FC}$ = 135 kNm. Consequently, the value of the overturning moment, resisted by the frame action amounted to:

$$M_{FA} = M_{over} - M_{FC} = 290 - 135 = 155 \text{ kNm} \tag{1}$$

$$M_{FC}/M_{over} = 155 / 290 = 0.53 \tag{2}$$

Note that in spite of the considerable changes of the axial forces in wall piers and considerable changes of their flexural capacity (compared to that corresponding to $N_g$), the total flexural capacity of both piers was only slightly changed. This is not surprising, considering that the flexural capacity of the piers is changing proportionally to the changes of the axial force. In pier subjected to tension the flexural capacity was reduced. At the same time the flexural capacity in the pier subjected to compression was increased for the approximately same amount.

In the analyzed case, the part of the overturning moment resisted by the frame action was 53 % of the total overturning moment $M_{over}$ (see Eq. 2). Note that in Eurocode 8, the coupled walls are defined as walls where the frame action contributes more than 25 % of the total overturning moment. Considering this definition, the analyzed structure should be designed following the rules for buildings with coupled walls.

As it was mentioned before (e.g. in Section 2) the studied walls are typically designed as cantilever walls, neglecting the frame action induced by slabs. In the studied case, this would lead to considerable underestimation of the compression stresses and shear forces in the piers subjected to compression. This would further lead to brittle failure of the wall and the damage, which is similar to that observed in the recent earthquakes (e.g. buckling of the longitudinal bars, which was observed in the presented experiments).

## 5. Conclusions

The half-scale shake table tests of the three story RC coupled wall building were conducted in order to study the slab-to-wall interaction. The specimen consisted of four rectangular walls connected only by the slabs.

The considerable coupling of wall piers was provided by the slabs. The ratio of the overturning moment resisted by the frame effect induced by the slab was larger than 50 %.

All slabs were fully activated. They were considerably cracked over the entire width between two rows of piers. The response of the wall piers was considerably different from that typical for the cantilever walls. The considerable rocking was observed in the piers subjected to relatively large tension, which was induced by the frame effect. In the piers subjected to the compression, the buckling of the longitudinal bars was occurred due to the relatively large compressive stresses also induced by the frame action of the slab.

The presented experiment confirmed the indications of some other experiments that can be found in the literature, that for certain building configurations only the slabs without beams can provide considerable coupling of wall piers. In such cases, the common design, based on the assumptions that the walls respond as cantilever walls, can lead to significant underestimation of the demand in piers. This can further lead either to brittle shear failure of walls or to their failure caused by the buckling of the longitudinal bars induced by large compression stresses, which were underestimated in the design.

## 6. Acknowledgements

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