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EXPERIMENTAL STUDY OF A TWO-STOREY FLAT SLAB BUILDING UNDER SEISMIC AND GRAVITY LOADS

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Abstract

Flat slab buildings for commercial, office and residential use are built in many countries. Yet, their behaviour under seismic and gravity actions is still not very well understood. Many studies have been carried out in North America but European research is lagging behind and currently Eurocode 8 does not cover the design of buildings with flat slab frames. The SlabSTRESS project (www.slabstress.org) was therefore undertaken at the ELSA Reaction Wall of the Joint Research Centre, within the Transnational Access activities of the SERA project (www.sera-eu.org).

The test specimen is a full-scale two-storey flat slab structure with three bays in the direction of loading and two in the orthogonal one. This layout allows testing interior, edge and corner slab-column connections. The plan dimensions are 9.0 m ×14.5 m. The slab of the second storey has punching shear reinforcement, while the one of the first storey does not. In addition, half of the slab at each floor is constructed with uniformly distributed horizontal reinforcement and the other half with the same amount of horizontal reinforcement mostly concentrated close to the columns.

Pseudo-dynamic tests have been performed first on the specimen treated as part of a building with two sub-structured (numerically simulated) ductile shear walls to study its seismic performance at the Serviceability and Ultimate Limit States. Quasi-static tests have been performed on the specimen – without the numerical shear walls – subjected to cyclic loading with increasing displacements and varying the magnitude of vertical loads.

The aim is twofold: first to verify the seismic performance of flat slab frames in a structure with earthquake resistant ductile walls; secondly to study the performance of the system beyond the design displacements.

The results of the project will ultimately provide the basis for a deformation-based design procedure and will allow the improvement of the Eurocode and *fib* Model Code punching shear models and reinforcement detailing rules to account for seismic loading.

Keywords: flat slab; large-scale tests; pseudo-dynamic testing; retrofit

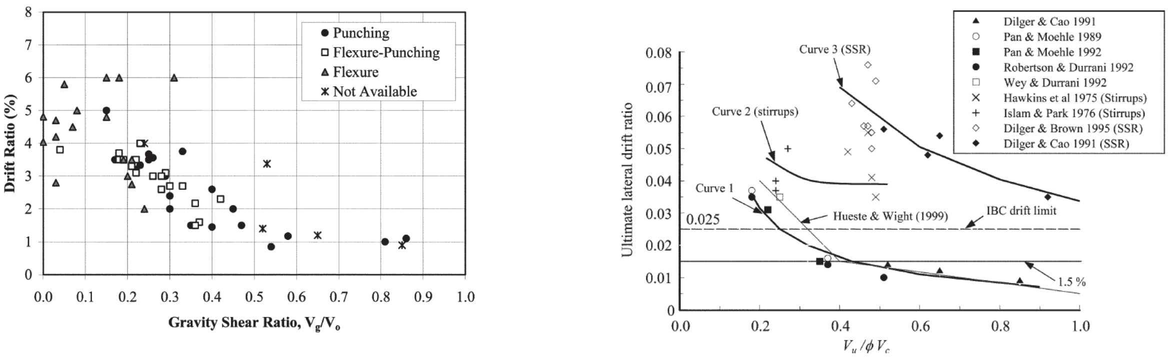
1. Introduction

Flat slab concrete buildings for office, commercial and residential use are built in many countries, but their behaviour under seismic and gravitational action is not yet fully understood. Many studies have been undertaken in North America and Asia, but European research is lagging behind and the current version of Eurocode 8 [1] does not cover the design of buildings with flat slab frames used as primary seismic elements. The SlabSTRESS (www.slabstress.org) project was therefore launched at the ELSA Reaction Wall of the Joint Research Centre, within the Transnational Access activities of the SERA project.

Design of flat slab frames in Europe developed mainly in North-European non-seismic countries [1][3][4]. The specifications of Eurocode 2 "Design of concrete structures" [5] consider the design of flat slabs and punching verifications for the effects of gravity loading. Eurocode 8 "Design of structures for Earthquake resistance" [1] does not include specific rules for flat slabs. The scientific community has expressed the aspiration to advance the knowledge and develop adequate code provision [6][7].

For the time being, design is carried out considering the provisions given by Eurocode 8 for secondary elements coupled with a primary dissipative earthquake resistant system; the former must bear gravity loads at the maximum design lateral deformations reached by the latter. These deformations are calculated for the design actions on the primary system, multiplied by the behaviour factor. In addition, the code specifies that the secondary elements must give a contribution lower than 15% of the total stiffness of the structure.

Research in North America produced a wide database of tests and code specifications for flat slab design for gravity combined with seismic loads. A set of results is shown in Fig. 1 for tests on interior slab-columns connections.



(a) (b)

Fig. 1 – Ultimate drift capacity and gravity shear ratio: (a) Test results for interior connections without transverse reinforcement [8], (b) Test results without and with transverse reinforcement [9]

Experimental activity in Europe started on slab-column connections under cyclic loading. Research at EPFL [10] tested full-scale slab-column connections without transverse reinforcement, comparing the effects of monotonic and cyclic loading, different gravity shear ratios and reinforcement ratios. For slabs subjected to low gravity loads, and for lower reinforcement ratios in particular, lateral drift cycles led to reduction of flexural strength and ultimate drift capacity when compared to monotonic tests.

Researchers at FCT/UNL in Portugal have developed a test setup [11] to test flat slab-column connections under realistic conditions of slab continuity under combined gravity loading and reversed horizontal cyclic drifts. Besides specimens without punching shear reinforcement, a series of specimens with punching shear reinforcement and solutions to enhance the deformation capacity , such as stirrups [12], headed studs [13][14], post-installed bolts [15], fibre-reinforced concrete [16] and high-strength concrete [17] have been tested with promising results. Isufi et al. [18] studied the numerical models of flat slab frames calibrated on these experimental results. Although the test setup of FCT/UNL overcomes some of the limitations of past tests on specimens that represent only the hogging moment region of the slab, testing full-scale specimens is more realistic. Furthermore, the tests at FCT/UNL have been limited to interior slab-column connections.

Previous tests of real-scale multi-storey flat-slab buildings are very limited. Coelho et al. [19] carried out pseudo-dynamic tests at the ELSA laboratory on a three-storey building with only one bay in each direction (7.0 m and 4.0 m respectively). The connections had overhangs along two of the four sides (1.5 m and 1.25 m respectively) and the floors were 0.3 m thick waffle slabs with thick slab around the columns extending four times the slab thickness in plan. The columns were rectangular, 0.3 m × 0.5 m reduced to 0.3 m × 0.4 m at the last floor. For the 475 years return period earthquake (Ultimate Limit State) the structure reached a displacement of 162 mm at the second storey (1.64% drift) with cracking around the columns. In a following test for the 2000 years return period earthquake test a failure in the pseudo-dynamic testing caused the jacks to pull the structure to failure and a drift capacity of 4.3% was reached. Heavy damage in the slab around the columns was accumulated at different floors and the top floor slab was nearly detached from the column at two connections.

Fick et al. [20] tested under cyclic lateral loading a flat-slab structure with two floor panels in plan and three storeys (plan dimensions: 9.1 m × 15.2 m, height: 9 m, slab thickness: 0.18 m). The spans measured 6.1 m in each direction, with 1.5 m overhangs all around the perimeter. The columns were square (46 cm × 46 cm). This resulted in a structure with only two types of connection, edge and corner, with an important influence of the overhangs. The test was stopped after a connection punched at 3.3% drift.

The North-American code ACI318 [9] and design philosophy [8] are based on a database of results mainly on individual connections, with or without shear reinforcement (Fig. 1). A central aspect for slabs without transverse steel is that ultimate drift ratio reduces with increasing gravity shear ratios (GSR). The GSR is the ratio between the acting vertical shear force and the punching shear resistance according to ACI318 [8]. Transverse steel increases the ultimate drift ratio of connections, with some reduction of ultimate drift capacity with the gravity shear ratio. Hence, the results for the two tests above must be compared considering the gravity shear ratio ensuing from the specimen design and the gravity loading. Fick et al [20] report a value of 0.21 and a nominal value close to 0.4 is calculated for [19]. These values are not in full accordance with the database of experimental results [8]. For a gravity shear ratio of 0.21 in [20], ultimate drift values in the database range from 2.7 to 3.6%. For gravity shear ratio 0.4 in [19], the ultimate drift is between 1.5 and 2.6%. It should be considered that the latter structure had a particular geometric configuration and a part of waffle slabs.

This experimental background shows the need of a comprehensive experimental study on a real scale structure. The SlabSTRESS program was proposed with the aim of providing support for the European design codes by studying the response of a full-scale two-storey building for seismic and gravity actions, different types of connections (corner, edge and interior), the redistribution of load effects in floors with realistic boundary conditions, different longitudinal reinforcement layouts, with and without transverse steel reinforcement. The aim of the testing phases presented here is twofold: first to verify, for actions corresponding to the Serviceability and Ultimate Limit States, the seismic performance of flat slab frames in a structure with earthquake resistant ductile walls (test A); secondly to study the performance of the system beyond the design displacements (test B). The first aim corresponds to verifying the requirement that the structure should bear gravity loads in correspondence of the maximum lateral displacement reached for the design action. The latter provides understanding of the deformation capacity of the system.

2. Design of the test mock-up

The structure is a reinforced concrete (RC) building with two 0.2 m thick flat slabs supported by columns, as shown in Fig. 2. Preliminary design was carried out according to the Italian national design code NTC 2018 [21] and was finalized with code provisions compatible with Eurocode 2 [5] and Eurocode 8 [1].

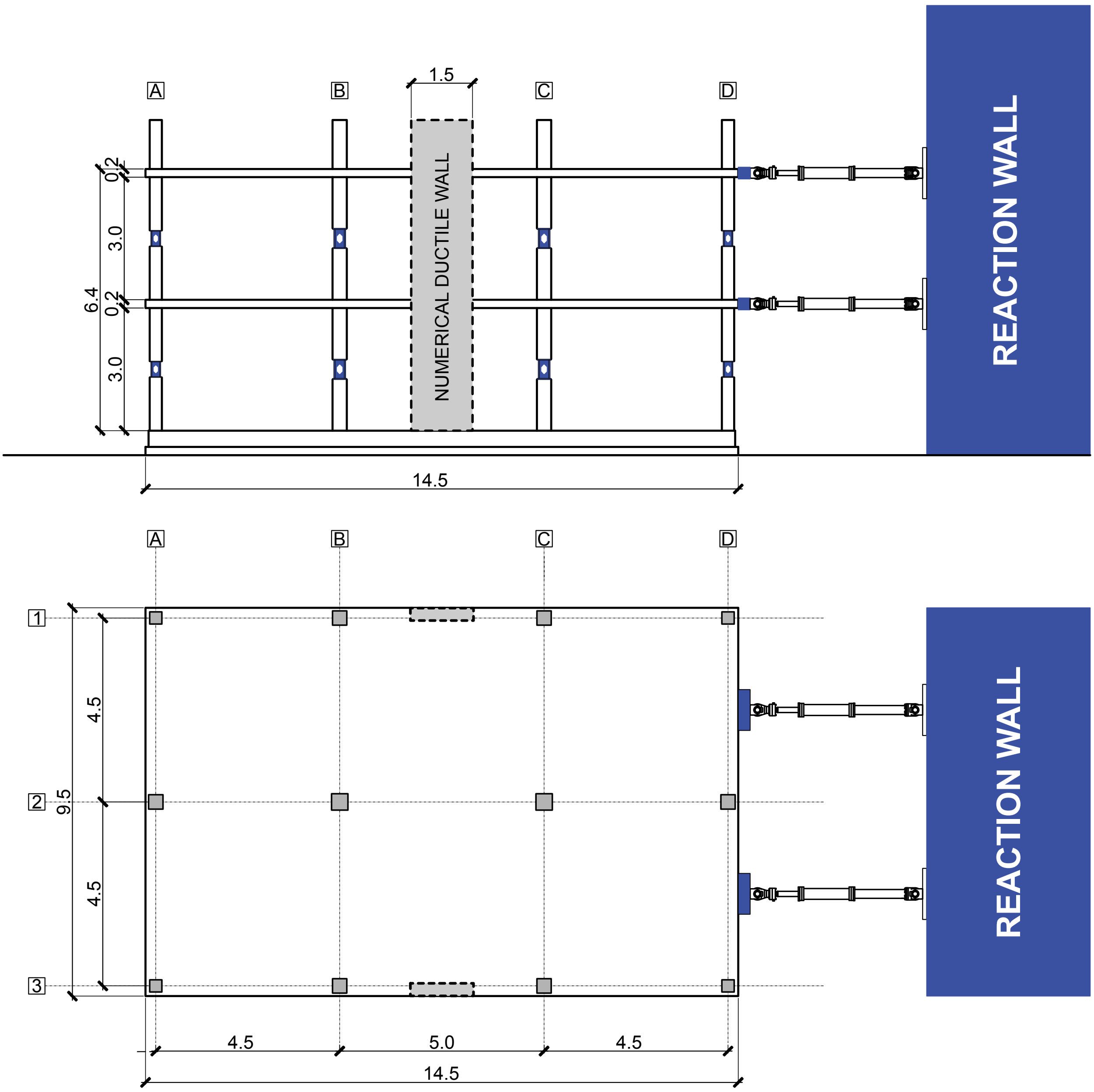


Fig. 2 – The Mock-up for the experimental campaign

The structure was conceived with primary seismic RC walls while the columns and the slabs were assumed as secondary seismic elements, not forming part of the seismic action resisting system of the building. The strength and stiffness of the secondary elements against seismic actions is neglected and their stiffness may not exceed 15% of the stiffness of primary members [1]. The structure belongs to the "ductile wall system" structural type, with uncoupled walls. A basic value of the behaviour factor $q_0 = 4$ is assigned to this type both by Eurocode 8 and NTC 2018.

The building includes six panels with spans on column axes 4.5 m, 5 m and 4.5 m in the long direction and 4.5 m and 4.5 m in the short one. One of the two slabs is provided with transverse reinforcement, although not requested from the design. Total dimensions of the floors are 14.5m x 9.5m. The slab thickness is 20 cm. Columns dimensions are 0.40 m × 0.40 m, 0.35 m × 0.35 m and 0.30 m × 0.30 m for the interior, edge and corner columns respectively. Column height is 3.0 m between each slab. A specific feature is that the columns are made of RC portions above and below the slab, and have a structural steel stub at mid height to carry out measurements of internal column forces. The walls are rectangular with cross-section dimensions 1.5 m × 0.32 m and are designed as ductile walls according to NTC 2018 and EN 1998-1-1. Finally, the structure has a reinforced concrete foundation with a slab and an orthogonal grid of stiffening beams in correspondence of the column lines.

The following loads, usual for residential buildings, were considered in the design of the slab and of the building: self-weight of the slab = 5 kN/m^2, permanent non-structural load = 3 kN/m^2, live loads = 2 kN/m^2. The load combinations were according to EN 1990.

For the flat slabs and the columns concrete of Class C30/37 was selected. The slab longitudinal reinforcement steel was of Class B450C (typically prescribed by the Italian code when ductile behaviour is required). Welded studs with yield stress $f_{yk} \geq 500$ MPa were used in the slab with transverse reinforcement. The column longitudinal reinforcement was B500 Class B with the exception of column bases where steelB450C (according to NTC 2018) was used, since plastic hinging was expected. The column transverse reinforcement was of Class B450A according to NTC 2018.

The design seismic action was obtained adopting a behaviour factor $q = 4$ and considering the structure as located in the city of Gemona, region of Friuli-Venezia Giulia (Italy), that was struck by a 6.5 MW strong earthquake in 1976. The design spectra of NTC 2018 at Ultimate Limit State (ULS) and Serviceability Limit State (SLS) were used. From the modal analysis of the structure considering cracked sections, the first vibration period in the direction of the walls is $T_1 = 0.315$ s. This value falls on the constant acceleration part of the ULS spectrum (spectral ordinate of the design spectrum $S_d = 153.06$ cm/s^2, for $q = 4$). For the SLS spectrum, the spectral ordinate is $S_a = 182.3$ cm/s^2. It is worth recalling that NTC 2018 prescribes verification at SLS for an elastic response spectrum, differently from Eurocode 8 where the inter-storey drifts at SLS are directly derived from those computed using the design spectrum at the ULS.

The ductile structural walls, which make up the primary seismic structure, were not part of the constructed building, but were numerically simulated during the testing in a pseudo-dynamic procedure. A linear equivalent model of the nonlinear response of these walls was developed to this end with the following procedure. A nonlinear model of the walls was developed inside the research computer code NONDA [22] with two (one per storey) RCIZ [23] wall elements adopted to model each wall. These elements include shear-bending-axial force interaction. The nonlinear numerical model of the structure was used to compute the time-history response at the ULS and SLS to an earthquake compatible with site spectrum, and to obtain a pushover curve for a linear distribution of lateral forces that approximates the inertial forces when the structure vibrates in its first mode. The pushover curve was used, along with the displacement coming from the time-history analysis to identify the stiffness and damping of the equivalent linear model used during the pseudo-dynamic testing.

The signal selected for the numerical analyses at ULS and SLS is the Y component of signal 007142ya form the MW 6.3 Bingöl earthquake of 01/05/2003. The signal has been selected with the REXEL computer code [24] as one best matching the code spectrum for ULS and SLS at the site. Matching the site elastic response spectra requires the original peak ground acceleration (*PGA*) of 2.92 m/s^2 to be scaled at 87% and 31%, respectively. The pseudo-acceleration response spectra of the selected accelerogram and the design spectra at ULS and SLS are shown in Fig. 3 together with the time history of the selected signal.

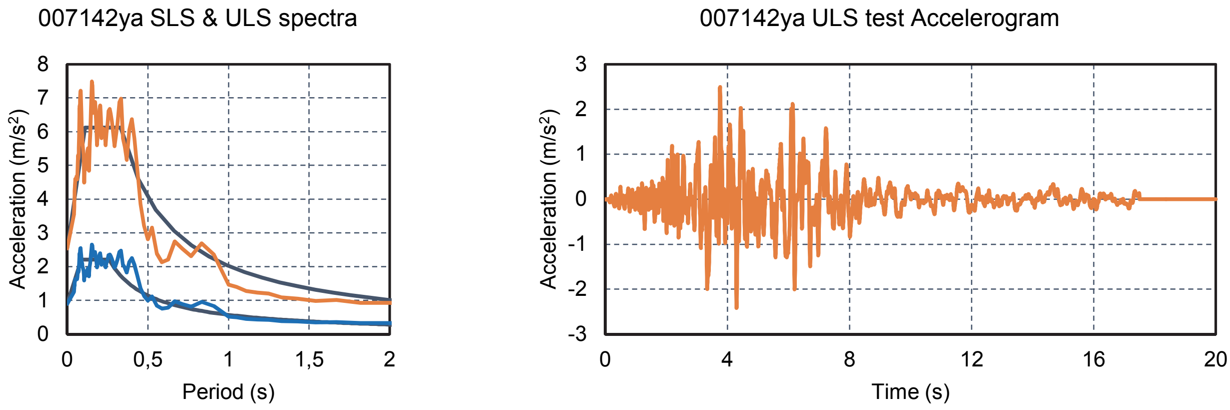


Fig. 3 – Pseudo-acceleration response spectrum of the selected signal and design spectra at SLS and ULS (left) and time-history of the selected signal (right)

As previously mentioned, the slab-column connections of the first storey had no punching shear reinforcement, whereas the connections in the second storey were reinforced with headed studs. Headed studs were chosen for the second storey as one of the most efficient and practical methods of enhancing the deformation capacity of slab-column connections [25] [26] [13]. Since punching shear reinforcement was not required to carry the gravity loads at the ultimate limit state, the purpose of providing transverse reinforcement at the second storey was to enhance the deformation capacity of the slab-column connections under lateral loading by preventing punching shear failures at low drifts. Commercially available studs were used. The layout and the diameter of studs, as well as the extension of the shear-reinforced zone, were determined to avoid punching failure for the gravity loads outside the studs-reinforced zone. The layout presented in Fig. 4 was chosen; with five rows of 10 mm studs spaced at 90 mm. Detailing requirements of the respective European Technical Approval were followed.

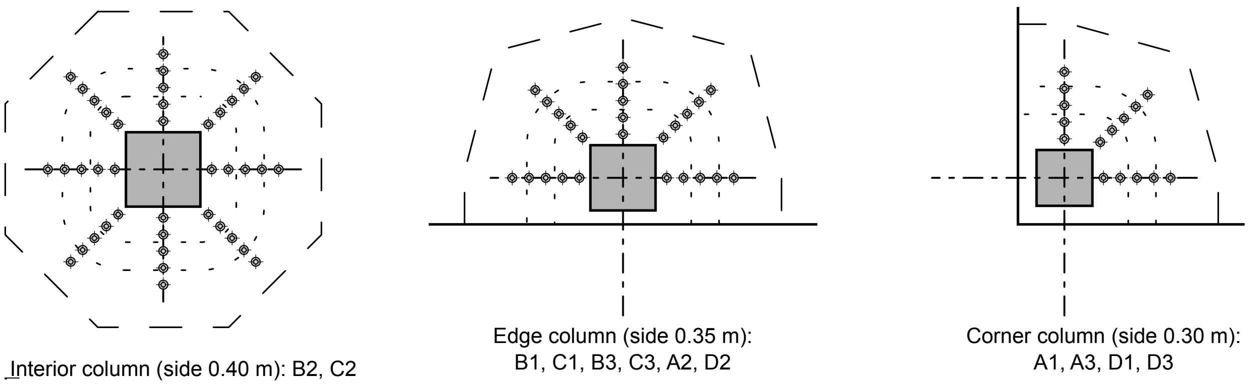
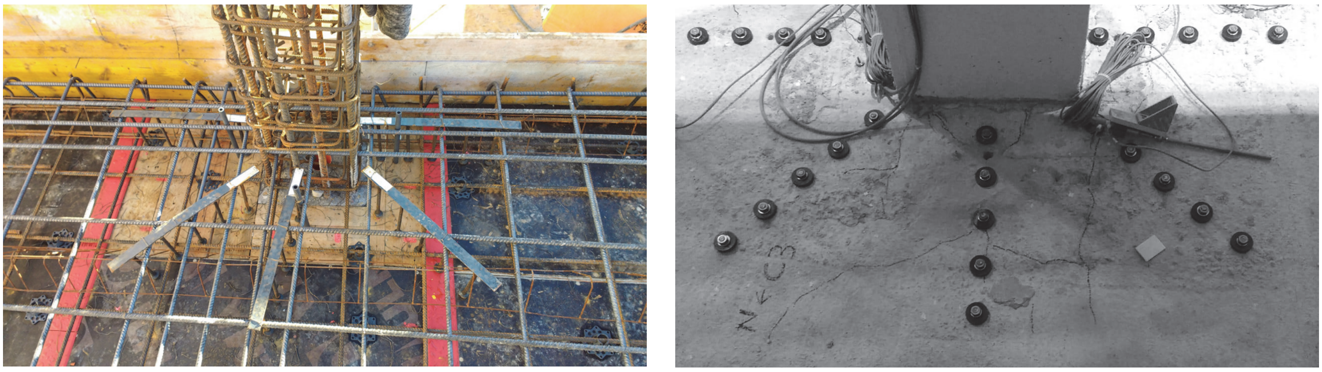


Fig. 4 – Punching shear reinforcement layout

After finalising test B1 (see Table 1), and before test B2, the first-floor slab was strengthened for punching at connections B1, C2 and C3, using 10 mm post-installed shear bolts (Fig. 4). In the past, this strengthening solution has been tested under seismic-type loading only on interior or edge slab-column connections [27][28][12]. The remaining connections will offer valuable information on the performance of slab-column connections that have not undergone repair. Past studies dealing specifically with this topic are limited, and they only cover isolated interior connections [14].

To facilitate the interpretation of the results, the layout of the post-installed bolts was kept the same as the layout of the studs used in the second floor. Fig. 5 shows the punching shear reinforcement units used (studs and post-installed bolts).



(a) (b)

Fig. 5 – Shear reinforcement on edge columns: a) pre-installed at 2nd floor and b) post-installed at 1st floor

To study the effect of flexural reinforcement distribution over the column, two different flexural reinforcement layouts were used in slab-column connections in symmetrical locations, with regard to the central axis along the shortest direction of the specimen.

3. Experimental programme and setup

According to the objectives described in the previous paragraphs, the test sequence has been planned to assess the seismic-design parameters used in the mock-up employing a limited number of tests, in order to achieve a progressive and controlled damage of slab-column joints. The test programme is summarised in Table 1. Two pseudo-dynamic tests were performed on the physical specimen coupled to virtual shear walls (numerically simulated) to assess the seismic performance at SLS and ULS (A1 and A2 respectively). A cyclic quasi-static test (B1) followed until punching was observed on the first floor. A number of slab-column connections at the first floor were repaired as described above and a cyclic test was performed until the most of connections were damaged (B1-R). The aim of the final cyclic test was to examine the ultimate capacity of the structure (B2).

The experimental program included two different setups to verify the effect of transverse punching reinforcement: the first floor without transversal reinforcement and the second floor with cast in situ steel studs as in Fig. 5a. In turn, each floor has two different longitudinal slab reinforcement layouts: the West side (alignments A and B) with rebars spread throughout the slab, while the East side (alignments C and D) with rebars concentrated close to columns, yet maintaining the same overall reinforcement ratio. The inserting of transverse reinforcement in slabs that were not originally equipped with was also verified, even after minor earthquakes (retrofitting). Reinforcement bars with the same specifications and in the same positions as those on the second floor were post-inserted the first floor slab, Fig. 5b.

Table 1 – Rationale for the test program

Test id.	**A1**	**A2**	**B1**	**B1-R**	**B2**
Test type	Pseudo-dynamic	Pseudo-dynamic	Cyclic	Cyclic	Cyclic
Assessment	SLS Design	ULS Design	1st floor punching	1st floor after retrofitting	2nd floor punching

3.1 Pseudo-dynamic tests

The pseudo-dynamic method mates the properties of the structure as a physical quasi-static model tested in the laboratory with a numerical model representing the inertia [29]. The motion equation for a model idealized in this form may be expressed as an ordinary differential second-order equation:

$$\boldsymbol{M} \cdot \boldsymbol{a}\,(t) + \boldsymbol{C} \cdot \boldsymbol{v}\,(t) + \boldsymbol{R}\,(t) = \boldsymbol{P}_{eff}\,(t) \qquad (1)$$

This implies that the structure can be analysed as if it was supported on a fixed foundation and subjected to an effective force vector $\boldsymbol{P}_{eff}\,(t) = -\,\mathbf{M} \cdot \mathbf{I} \cdot a_g\,(t)$, where $\boldsymbol{I}$ is a vector of zeros and ones and $a_g\,(t)$ is the ground acceleration time history. The mass matrix, $\boldsymbol{M}$, the viscous damping matrix, $\boldsymbol{C}$ (typically null) [29], and the excitation force vector, $\boldsymbol{P}_{eff}\,(t)$, are all numerically specified. The restoring force vector, $\boldsymbol{R}\,(t)$, which is, in principle, nonlinear with respect to the displacement vector, $\boldsymbol{d}\,(t)$, is experimentally measured. At each time instant t, the equation is numerically solved, from the restoring forces, $\boldsymbol{R}\,(t)$, measured at time t by the actuator load cells, to obtain the acceleration response, $\boldsymbol{a}\,(t)$,velocity, $\boldsymbol{v}\,(t)$ and displacement, $\boldsymbol{d}\,(t + \Delta t)$, at the next time step. The displacements calculated at instant $t + \Delta t$ are then imposed on the structure by means of actuators, at the end of the step their load cells measure the restoring forces $\boldsymbol{R}\,(t + \Delta t)$ to be used for the calculation of the response to the next time step. The test program uses a motion equation with two degrees of freedom, with the roof displacement x, parallel to the E-W direction for the mock-up. The actuators use two high-resolution optical encoders mounted on two reference frames as feedback for the proportional-integral-derivative controller (PID), to impose the calculated displacements of each floor without torsion. The mass for tests A1 and A2 was assumed to be 137 tonnes at the 1st floor and 139 tonnes at the 2nd floor. The reference input motion used in the pseudo-dynamic tests was a unidirectional 15 s-long time history, shown in Fig. 3b. A zero-acceleration signal was added after the end of the record, to allow a free vibration of the test structure.

3.2 Cyclic tests

The cyclic displacement protocol was composed of increasing steps, in turn made of three cycles. The displacement history is presented in Fig. 6. At the end of the test, in case a residual load occurred, the system was manually returned to a zero-load position through small cycles around the zero displacement.

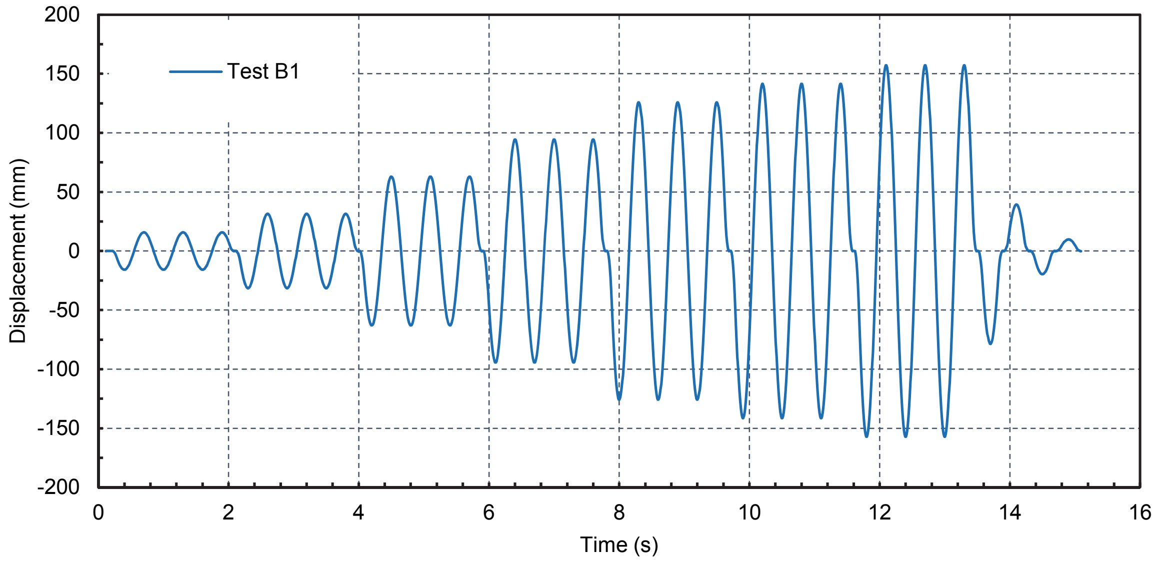


Fig. 6 – Displacement history for cyclic test

3.3 Loading system

A pair of hydraulic actuators, connected to the ELSA reaction wall, imposed the displacements to each floor. Each jack had 500 kN of working load and was equipped with a load cell for continuous measurement of the supplied load. The total load capacity was 2000 kN. The actuators were connected to steel beams that were in turn bolted to rebars cast into the slabs. Reinforcing bars, post-tensioned at over 500 kN for each actuator, were used to avoid the gap between the metal beams and the slabs and allow a two-sided coupling of the structure with the actuators. Two high-resolution (2 μm) displacement transducers continuously measured the drift at each floor level. These optical encoders were mounted on two reference frames and served as feedback for the PID controller for each actuator, Fig. 2 and Fig. 7.

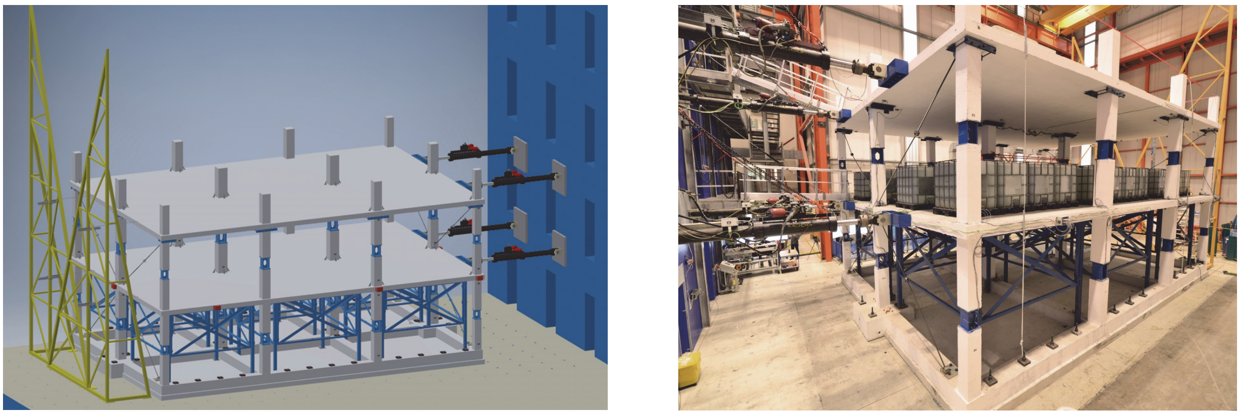
3.4 Transducers for local measurements

Local measurements of displacement, rotation and strain were acquired using a network of electrical transducers. For each column, from the foundation to the roof, a control unit for the analog-to-digital conversion collected a cluster of signals continuously streamed by different types of electrical transducers: displacements, rotations and deformations. The signals were digitally converted and routed through a local data network to a central unit, together with the signals (load and displacement) from the actuators.

In order to analyse each slab-column connection individually, it is necessary to know its internal actions. Since it is not possible to do it directly, to be able to reconstruct the actions within the joints, sensors have been specially designed and positioned in the middle-height of each column to measure its internal actions: shear and axial forces and bending moment. These devices have provided accurate shear and bending moment measurements in all columns during the experimental campaign.

A reduction of the measure points was allowed by the double-symmetry of the mock-up. The sensors to measure the kinematics (displacement, rotation and local deformation) were installed in all columns of the central alignment (A2, B2, C2 and D2) and only on two columns of each external alignment (A1, B1, C3 and D3). Differently, the devices for measuring internal actions were placed on each column; otherwise, it would be impossible to reconstruct the internal equilibrium conditions of the structure.

Given the briefness of this paper, the local measures obtained cannot be presented; therefore, they will be subsequently omitted.

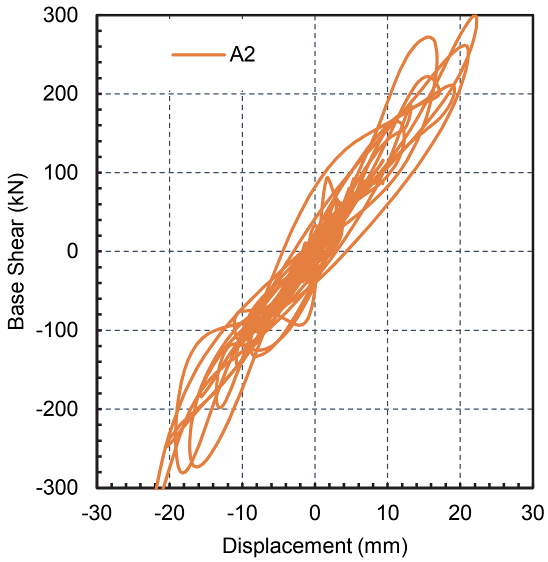


(a) South-West rendering view (b) North-East view

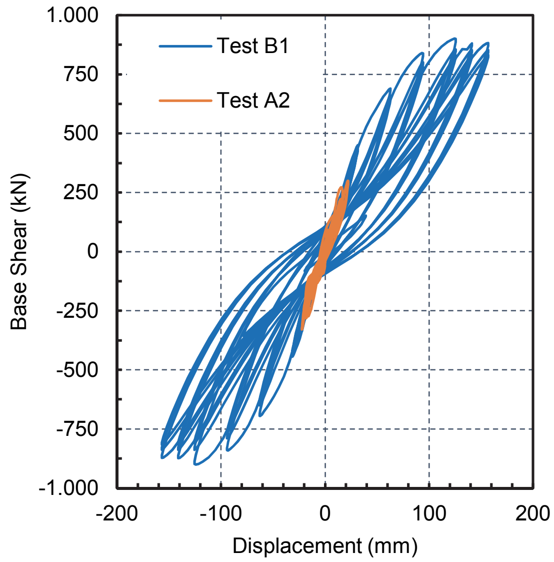
Fig. 7 – Test set-up

4. Experimental results

The global response in terms of base shear force versus the displacement recorded on the second floor are presented in the following. Fig. 8a shows the response of the flat-slab structure for test A2 using the ULS accelerogram. The base shear is the sum of the reaction forces of the actuators on the physical frame structure within the pseudo-dynamic response of a system with the shear resistant walls modelled with equivalent elastic stiffness and viscous damping. The maximum global relative drift reached approximately 22 mm, equal to a drift ratio of 0.35% (top displacement divided by the structure height = 6400 mm).



(a) Test A2



(b) Tests A2 and B1

Fig. 8 – Seismic response of the flat-slab frame (a) Test A2; (b) global results: Tests A2 and B1

Fig. 8b shows the results of test B1 compared with test A2. The structure reached a yield point between 1.5% and 2% drift. This can be related to yielding of the ground floor column bases. A progressive stiffness loss and pinching of the cycles with the increase of the displacement took place. The choice of the maximum relative inter-storey drift imposed in this test corresponds to the maximum value reached in slab-column connections in previous tests (Fig. 1). The SlabSTRESS specimen reached 2.5% drift (160 mm displacement) in Test B1. The base shear force reached for cycles of increasing amplitude remained nearly constant beyond the drift of 1.5%. In the repetition of cycles of equal amplitude starting from 1.5% (95 mm displacement) a small loss of resistance occurred, indicating a sound global structural response up to 2.5% drift. The superposition of the response in the two tests is shown in Fig. 8. The correspondence of the equal amplitude cycles up to 0.4% drift is evident.

5. Conclusions and outlooks

The results presented here are the first part of the SlabSTRESS test program. The seismic test at ULS and a cyclic test up to representative drift values are shown. The results of the former test indicate that flat slab frames combined with earthquake-resistant walls have nearly elastic response, with a maximum drift ratio of about 0.4 % for an accelerogram compatible with the ULS.

In the following cyclic tests, the response of the structure was explored up to the maximum drifts reached in the literature tests on individual connections (1.5-2.5%) for gravity shear ratio of 0.4. The mock-up showed a good overall response up to 2.5% drift ratio. The cyclic test has shown that the system can achieve high drift with limited or no strength deterioration and with satisfactory ductility and energy dissipation. No punching occurred in interior connections, while edge and corner joints were damaged. The results summarised above indicate that the combination of flat slab frames with different types of primary seismic systems can be explored. At the same time, these results confirm the need to perform large-scale tests on full-size buildings to understand the overall seismic behaviour of the building and to validate the experimental campaigns previously undertaken on sub-assemblies and small-scale tests.

The features of a seismic retrofitting system for flat-slab have been described. The program has shown that the system, previously used in individual connections, can be inserted in a building structure. Future testing phases will show the effectiveness of this solution.

Further research developments will analyse the damage accumulation, comparing the response of individual slab-to-column joints, to understand the role of different details of slab longitudinal and transverse reinforcement.

The outlooks of the SlabSTRESS program are the development of the procedure for the seismic design of a flat slab frames with better consideration of the role of primary and secondary seismic elements. The response of the slab-column connections extracted from the experimental results will be used to verify and improve, where needed, the punching shear provisions and detailing rules of Model Code and Eurocodes 2 and 8.

6. Acknowledgements

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