FLAT SLAB DESIGN AND EXPERIMENTAL RESPONSE

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Abstract: *Flat slab frames are characterized by a concentration of bending moments and shear forces near the column and by a complex structural behaviour under lateral loading. For this reason, it is important to have adequate code provisions that cover the seismic design of these structures. In Europe, current seismic design code Eurocode 8 does not cover flat slabs that are part of the primary load resisting system, but it does contain general guidance and rules for secondary seismic members. On the contrary, the draft of the new Eurocode 8 for buildings contains provision to verify flat slabs considered in the design as primary seismic elements, but no explicit guidance is provided to verify flat slabs considered as secondary elements (where horizontal actions are primarily carried by a bracing system). In this context, this paper aims to outline a design approach for flat slabs considered as secondary elements and to show its application to the design of a two-story building with primary ductile walls and secondary flat slab frames as a case study. The structure designed in this manner is the object of a full-scale testing program carried out at the ELSA Laboratory at JRC in Ispra, Italy, with the purpose of providing the basis for improved design rules and code development. The results of the design example are verified using nonlinear numerical modelling. An overview of the testing program is presented and the research objectives are described. In designing this structure in accordance with Eurocode 2 and 8, several aspects are discussed and areas for potential improvement of the codes are pointed out*

1. General considerations

Flat slab design in a seismic zone is advancing in the recent research (Fardis, 2022; Muttoni et al. 2023). Flat slabs are not covered by the current codes EC8: as primary systems . Flat slab frames are not covered in Eurocode 8 (CEN, 2004) as part of the seismic-action-resisting system of Ductility Class (DC) M or H buildings. In such buildings they can only be considered as secondary seismic elements, without a contribution to earthquake resistance, and can be designed according Eurocode 2 provisions for flat slabs. In buildings of DC L in low seismicity areas, by contrast, they may be considered as a part resisting the seismic-action, designed to resist the seismic action effects according to Eurocode 2, with a behaviour factor q lesser or equal to 1,5 (Fardis, 2022).

Consequently flat slab frames are typically considered as a secondary system in buildings where the primary elements, e.g. shear walls, resist the seismic action in medium-high seismicity regions. Fardis (2009) proposes the design of flat plate frames as secondary systems, based on resistance verifications for the structure bearing gravity loads at the deformations imposed by the design earthquake. Linear elastic analysis of the effects of lateral deformations on the flat slab frame leads to high effects at the connections and design problems to provide the design resistance. The verification of the structure bearing the gravity loads at the lateral design drift can be carried out based on the deformation capacity of the connections. An approach based on this concept is included in North American regulations (ACI 318, 2019, ACI 421, 2010).

Even though the secondary system is not designed to seismic action, the slab-column connections should be able to resist the gravity loads after the earthquake with a damage level allowing for a possible repair. A situation after punching as described in Figure 1, although allowing to avoid a total collapse, can be considered as a near collapse limit state and would be hardly reparable.

For this reason, it should be prevented as a design situation so that the slab-column connection should be designed to avoid punching by limiting the gravity shear ratio to guarantee adequate deformation capacity. This can be determined based on tests results. The approach presented in ACI318 (2019) was adapted to European codes by Ramos et al. (2017) calculating the shear ratio according to EC2.

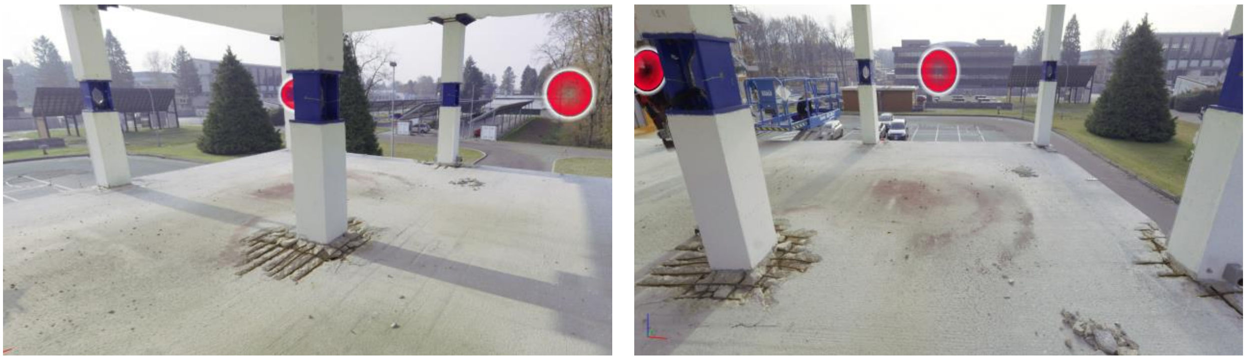

Figure 1. Damage induced by lateral drift in a full scale test (Coronelli et al., 2021).

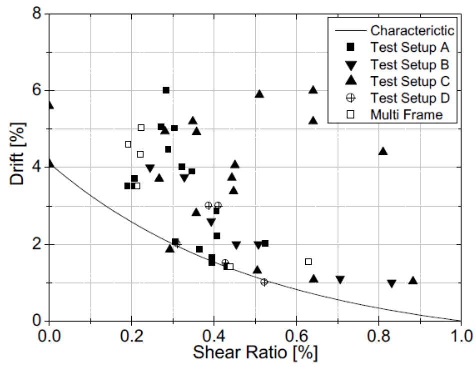

Figure 2. Drift capacity as a function of shear ratio calculated per EC2:2004 (Ramos et al., 2017).

2. Outline of the design approach

The secondary flat slab and primary wall design can be carried out as follows (see also chart in Fig.3):

- A limit of inter-storey drift d_R for the structure is chosen in a first step by the designer, complying with the code specifications.

- The flat slab frame is designed for gravity loads, accounting for a gravity shear ratio (GSR) which corresponds to the drift limit d_R previously set. The relationship between drift ratio and GSR can be that based on test results (Ramos et al. 2017) or using the formulation proposed by Muttoni et al. (2022). As an alternative, the designer can also provide shear reinforcement to increase the drift capacity.

- The primary elements are designed for the seismic design situation, where the q factor and the ductility class are chosen according the type of primary system, as for any structure according to EC8:2004.

- The deformations d_e of the primary elements are calculated, using a model considering only the primary elements with a cracked stiffness, and multiplied by the q factor to obtain the design drift $d_s=q_d\ d_e$. This is compared to the limit established at the first step.

- The design drift demands d_s are verified to be smaller than the ultimate drift capacity d_R of the flat slab frame.

- If this is not verified, there are two options: either to reduce the drift of the structure by re-dimensioning the primary system to be stiffer or to improve the deformation capacity (e.g. by increasing the slab thickness, increasing the column cross-section or by providing shear reinforcement).

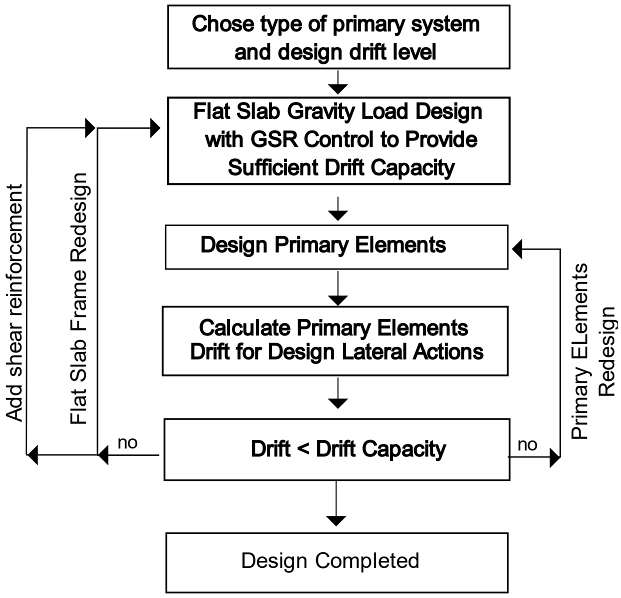

Figure 3. Design flow chart.

The prescriptive provision of EC8:2004 that the secondary elements can bear the gravity loads at the deformation imposed by the design earthquake is verified explicitly by limiting the effects of the gravity loads so that the ultimate deformation capacity is higher than the deformations imposed by the seismic actions.

EC8:2004 indicates that the total contribution to lateral stiffness of all secondary seismic members should not exceed 15% of that of all primary seismic members. This non-prescriptive indication was taken into consideration in the design of the building presented in the next section, with the aim to understand its implications in design and the response.

Column design procedure

The columns can be designed based on a collapse mechanism with plastic hinges at the base of the column and in the slab at the connection with the column. The following workflow is proposed:

1. ULS gravity load design of slab.

2. Estimation of maximum unbalanced moment transfer on the column $M_{unb,i}$ at floor i, under assumption of flexural yielding of the slab (ACI421, 2010).

3. Design of the column base:

3a. Selection of reinforcement ratio and/or moment resistance $M_{col,pl}$ for the base cross section of the column (taking into account the design value of the axial force in the seismic configuration).

3b. Check that sufficient curvature ductility is available and eventually add the required confinement reinforcement.

3c. Solution of the column internal forces and moments with a simplified scheme where the column is hinged at the base and laterally simply supported at the floors level. The column is loaded with $M_{col,pl}$ at the base of the column and $M_{unb,i}$ at the floor levels.

4. Final design of the column for internal forces calculated at point 3 and the axial force for the seismic configuration.

The reinforcement along the column can be designed considering biaxial moment effects (30% reduction of resistance) and an overstrength ratio of 1.3.

3. Nonlinear numerical modelling

The design approach outlined in the previous sections has been applied to a two-story building with primary ductile walls and secondary flat slab frames. The structure designed in this manner has been the object of a full-scale testing program (“SlabSTRESS”, Coronelli et al., 2021) carried out at the ELSA Laboratory at JRC in Ispra, Italy, with the purpose of providing the basis for improved design rules and code development. A schematic 3D view of the building model is reported in Figure 4.

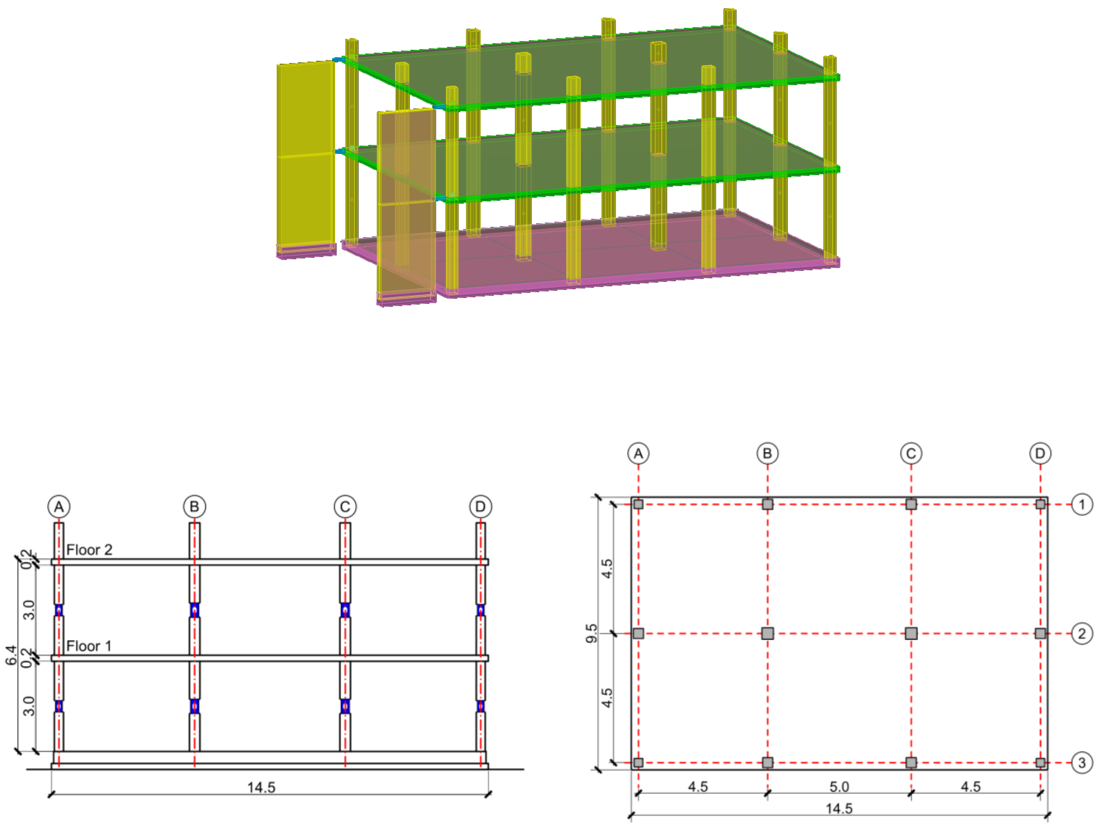

Figure 4. (a) 3D scheme of the building; (b) geometry (units m).

The results of the design example were verified using nonlinear numerical modelling. Two different models were developed, one for the primary ductile walls and one for the secondary flat slab frames. These numerical models were set up in different computer codes following modelling established procedures of the literature (Mulas et al. 2007a, b; Martinelli 2008; Coronelli 2010).

The use of two models complies with the Eurocode design approach, with primary walls and secondary flat slab frames reaching the same displacements, with the primary elements of the system providing most of the seismic resistance and the secondary ones bearing the gravity loads.

The design example was assessed by comparing the capacity curves of the primary walls and of the flat slab frames, as obtained from a push-over analysis.

The nonlinear numerical model of the primary walls was set up within the research computer code NONDA (Martinelli et al. 1996). The primary walls were modelled using RCIZ (Reinforced Concrete Inelastic Zone) column fibre element (Martinelli 2008). The RCIZ element is well suited to reproduce the response of R/C shear walls and its performance has been extensively tested against well documented experimental cyclic tests of R/C walls designed for different ductility levels (Martinelli 2002, Mulas et al. 2007b). Each wall has been modelled using one RCIZ finite element one per storey following procedures of proven accuracy (Mulas et al. 2007a; Martinelli et al. 2013).

The RCIZ element is based on Timoshenko's beam theory and accounts for the axial force-shear force-bending moment interaction. The internal shear force is computed considering the two load carrying actions associated to a 30° compression field and a to concrete contribution. The latter includes aggregate interlock and the compression of concrete in the cross-section of an element under axial force and bending, that in the formulation of the RCIZ element includes the effects of arch action.

The element is stiffness-based, and integration along the element adopts a five-point Gauss-Lobatto scheme to include among the integration points the element the end sections, as in these zones the largest inelastic behaviour is expected.

The uniaxial material model for steel by Monti and Nuti (1992) has been adopted for longitudinal and transversal reinforcement, while the model proposed in Stevens et al. (1987), restricted to fixed principal strain directions, has been adopted to describe the concrete fibres. Mean values of the material properties are used in the input. The cross-section was discretized in 68 concrete fibres and 20 steel fibres (each representing a reinforcement bar). The model adopted in the NONDA code is schematically depicted in Figure 5.1.

The capacity curve from the pushover analysis for lateral loading is shown in Fig.7.

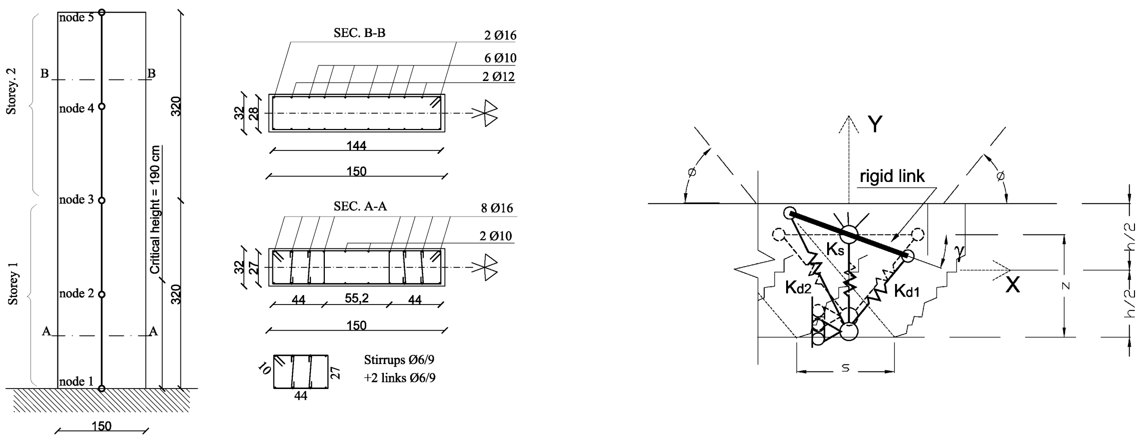

Figure 5. Schematic representation of the FE model of the primary walls part (left); reinforcement layout (center); shear truss mechanism (left).

The nonlinear model for the flat-slab frames has been developed following the approach proposed in Coronelli (2010) implemented within the SAP2000 computer code (CSI 2019). The columns are modelled with one elastic beam element per storey, with one flexural plastic hinge at the base of the members at the ground floor. The slab is modelled by a two-way grid of elements. The model of the building, modelled with the grid of elements as shown in Fig.6.a

Each grid element has an elastic part and point plastic springs for the flexural, torsion and shear response (Figure 6, b-d). Each grid element has two flexural hinges at each end, to capture biaxial bending, and one torsion hinge and one shear spring in the centre. The flexural, shear and torsion hinges of the elements, framing into the column, model the most relevant inelastic phenomena that might occur. The shear model considers punching resistance on a critical perimeter at *d*/2 from the column face, using the ACI318 model. Torsion resistance is based on the models proposed by Park and Choi (2006), including the interaction with bending moment and shear in the slab at the column face. The nonlinear flexural moment-curvature and force-strain response for shear is described using a sectional model proposed by Bentz (2000). The moment-twist relation is based on an analytical formulation by Collins and Mitchell (1980). The ultimate inelastic shear strain and torsion twist angle are based on formulations verified with experimental tests on slab-column joints.

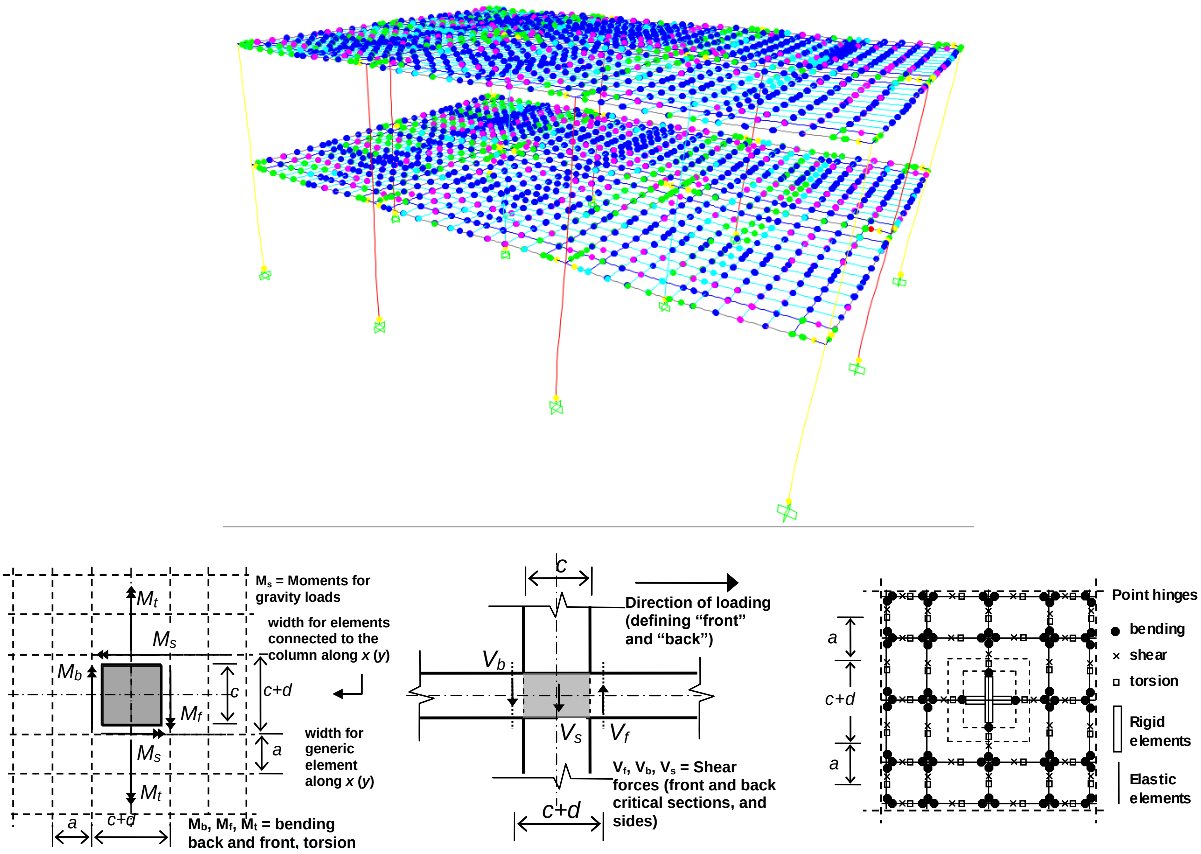

Figure 6. FE model of the flat slab frame.

Rigid elements are used around the columns to enforce the kinematic effects of the column dimensions, the length of these rigid elements then corresponds to half the column dimension *c*. The slab element grid spacing is approximately *c+d*, where *c* is the column size and *d* the effective depth of the slab.

The modelling approach has been verified for the nonlinear static analysis of internal, edge and corner slab-column connections tested under gravity and lateral loading, with different values of gravity shear ratio and reinforcement details with and without shear reinforcement (Coronelli 2010). The efficiency of the formulation has been verified by Coronelli and Corti (2014) for the pushover analysis of a flat slab floor tested with cyclic loading by Hwang and Moehle (2000).

The capacity curve from the pushover analysis for lateral loading is shown in Fig.7.

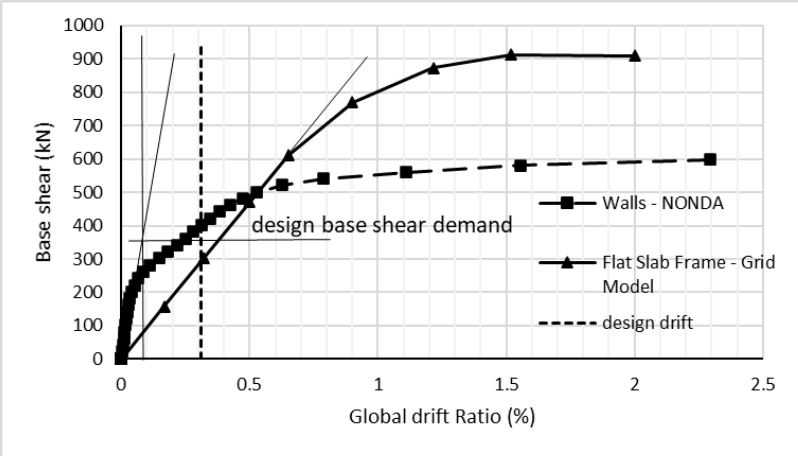

Figure 7. Comparison in terms of the drift ratio (displacement at the top divided by the total height) versus the base shear for the primary walls part and flat slab part, obtained from the two numerical models.

4. Full scale testing program

The testing programme was defined within the transnational access activities of the SERA project (www.sera-eu.org) to study with realistic loading conditions both the global and local response of flat slab frames under seismic actions. The experimental activities encompassed full-scale testing, at the ELSA Reaction Wall of the European Commission's Joint Research Centre, of a two-storey building designed according to the procedure previously outlined. The aim of the test campaign is to assess the seismic performance of the building, its ultimate capacity beyond the design displacement and the performance of the building after repair of damaged joints. For practical purposes a hybrid (physical-numerical) specimen of the building was tested. In this hybrid specimen the primary walls of the building were simulated numerically during the testing while the secondary flat slabs frame part was physically tested in 1:1 scale.

The testing programme includes two sets of tests: seismic pseudo-dynamic tests (Pegon et al. 2008) and cyclic quasi static tests. The seismic pseudo-dynamic tests were carried out on the hybrid (physical-numerical) specimen. The cyclic tests were carried out on the (physical) flat slab frame part only.

The seismic pseudo-dynamic tests comprised two seismic tests: one at the serviceability (damage) and one at ultimate (life safety) seismic limit state. The second test aims in particular to verify the requirement of EC8:2004 (CEN 2004a) that the flat slab frame maintains the capacity to bear gravity loads when subjected to the maximum deformations reached for the seismic design action. The same time history, scaled in acceleration, of a natural ground motion compatible with the elastic response spectrum at the design site, was used for both tests.

The quasi-static tests were performed with cyclic loading on the flat slab frame part only, without simulating the walls, to understand its deformation capacity beyond the design displacement. The interest was to study the redistribution of action effects within the floors; the response of different types of slab-column connections (corner, edge and interior) with realistic boundary conditions, the failure modes of the different connections, and the effect of different layout and detailing of longitudinal and transverse reinforcement using a realistic gravitational loading.

The programme addressed also the behaviour of existing structures. The detailing of reinforcement includes choices common in European design, such as anchorage with simple L shaped bends at the edges.

5. Test results

The test results are briefly summarized here. For a detailed account see Coronelli et al. (2021).

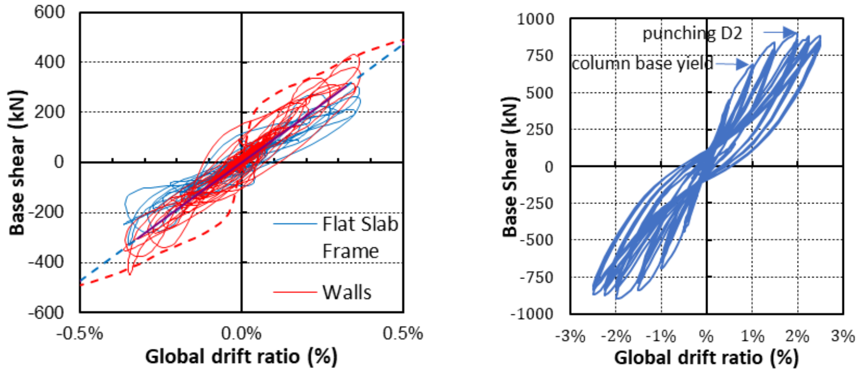

Figure 8. Test results for the design earthquake (left) flat slab frame global response in cyclic test (right).

The system of the secondary flat slabs coupled with primary walls in the pseudodynamic tests showed a very good performance with limited damage for a natural accelerogram corresponding to the design earthquake. In Figure 8 the base shear is reported in solid line for the primary walls part and for the secondary flat slab frame part, and is compared to the pertinent pushover curves which are reported in broken lines.

Under cyclic loading the flat slab structure showed a ductile response up to a global drift ratio of 2.5% (Fig.8). The connections along the edges suffered damage and failed for drifts ratios between 1.5% and 2.5%. In a following test the system reached 6% global drift ratio, with the integrity reinforcement supporting the failed connections; at this drift level the breaking of the reinforcement at the base occurred in one column, due to compression buckling and the nonlinear cycles in tension compression.

6. Discussion

The design approach proposed in the paper has been verified by the testing of a full-scale two storey structure and non linear analysis. The damage was concentrated in the flat slab connections and the ground floor plastic hinges in the columns. The columns did not show evident damage above this zone.

The design rules for secondary systems in EN1998 do not take into consideration the contribution of the flat slab frame in the response. The results in Figure 7 highlight the relevant contribution for the base shear of the system of the flat slab structure.

The deformation capacity shown by the system in cyclic tests indicates the path for the possible adoption of flat slabs as primary systems. The design did not include specific seismic design rules or detailing for the slabs.

The lateral and corner connections showed more damage at lower global ratios drifts between 1.5 and 2.5%. This was probably a consequence of the detailing of longitudinal reinforcement anchorage along the edges (Coronelli et al., 2021).

7. Conclusions

A design approach for secondary flat slab frames has been proposed. The concepts and steps shown comply with the current European seismic regulations.

The design approach proposed has been verified by the testing of a full-scale two story structure and nonlinear analysis. The results show the relevant contribution for the base shear of the system of the flat slab structure.

The system of the secondary flat slabs coupled with primary walls in a pseudo-dynamic test showed a very good performance with limited damage for a natural accelerogram corresponding to the design earthquake. The flat slab structure under cyclic loading showed a ductile response up to approximately 6%. This confirms the possibility to take into consideration the design of primary flat slab frames (Fardis, 2022).

Only nonlinear static analyses are shown here. The models presented here are currently under development for the prediction of the hysteretic response with energy dissipation and stiffness deterioration. Interesting highlights in this respect have been show by a blind competition (Coronelli et al., 2023).

The results are relative to a two storey building with seismic resistant shear walls. The developments of the research will explore the extension of the findings to a higher number of stories, as well as to different types of primary systems.

Acknowledgements

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