SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

SHAKE TABLE TEST FOR ASSESSING THE SEISMIC BEHAVIOR OF WAFFLE SLAB STRUCTURES WITH NON-ORTHOGONAL COLUMNS DISTRIBUTION

Jesús DONAIRE-ÁVILA[1] and Amadeo BENAVENT-CLIMENT[2]

ABSTRACT

This paper presents an ongoing experimental study aimed at improving the knowledge on the seismic behaviour of waffle slab structures. To this end, shake table tests have been carried out on a test structure that represents a scaled portion of a prototype 3-story waffle flat slab structure (WFS) supported on three columns. This prototype building was located in the highest seismicity zone of Spain, Granada, fulfilling the current Spanish seismic code NCSE-02 (Ministerio Fomento, 2003). The specimen was subjected to successive seismic simulations of increasing intensity that reproduced different seismic hazard levels at the city of Granada (Spain). This paper describes the experiments and presents the overall response obtained at the end of each seismic simulation, in terms of interstory drifts, lateral stiffness, level of damages, etc. It is concluded that WFS structures designed as a primary structural system to carry the seismic actions, present a poor seismic behavior. For lower seismic hazard levels, the building could be out of service owing to the breakage of claddings and/or partitions caused by the large values of the interstory drift. Moreover, the damage concentrates at the ends of the columns and at the spandrel beams of the exterior waffle slab-column connection. This situation could jeopardize the stability of the structure and bring the structure to collapse.

INTRODUCTION

Reinforced concrete (RC) flat slab structures are widely used for both office and residential buildings in moderate-seismicity southern European countries, such as Spain, owing to their architectural advantages. To reduce the self-weigh (and consequently to increase the spans up to about 10-12m) voids are sometimes included in the slab. This solution is known in the literature as “waffle flat slab” (WFS). In Spain, the WFS supported on RC columns is nowadays one of the most commonly used structural system, and constitutes the object of this research.

Beside the architectural benefits, WFS structures have significant drawbacks when used as main lateral resisting system in earthquake-prone regions. The main drawbacks of the WFS systems are: (i) the low lateral stiffness of the structure owing to the low torsional stiffness of the plate; and (ii) the low ductility and energy dissipation capacity. These shortcomings were clearly revealed by seismic events such that the Michoacan earthquake (México, 1987) or the more recent earthquake in Lorca (Spain, 2011). The former promoted significant research on flat slab structures, with improvements in both seismic and concrete standards such as ACI 318-11 (2011). In Europe, however, Eurocode8

[1]Dr. Civil Engineer, University of Granada, Granada (Spain), donaire@ugr.es
[2]Professor, Department of Mechanics of Structures and Industrial Constructions. Polytechnic University of Madrid, amadeo.benavent@upm.es

(CEN, 2004) does not give provisions for slab structures. In Spain, the Spanish Seismic Code NCSE-02 (Ministerio Fomento, 2003) permits the design of WFS structures in seismic zones but limiting the ductility-based reduction factor for seismic forces up to 2.

PROTOTYPE STRUCTURE AND TEST SPECIMEN

The three-story WFS prototype with non-orthogonal distribution of columns shown in Fig.1 is considered as the prototype in this study. It is representative of buildings with irregular column distribution. It was designed with the limit state design method considering gravity loads (dead loads of 3.13 kN/m^2 for floors and 3.46 kN/m^2 for the roof; live loads of 2 kN/m^2 for floors and 1 kN/m^2 for the roof) and seismic loading fulfilling the provisions of the current Spanish seismic code NCSE-02 (Ministerio Fomento, 2003). The prototype is located in Granada (Spain), the highest seismicity area of Spain with a design ground acceleration a_b= 0.23g (g is the gravity acceleration). The design concrete compressive strength was f_c =25 MPa and the yield strength for the rebar steel was f_y = 500 MPa. The WFS consists of 35 cm depth waffle flat slabs with a 83x83 cm centre-to-centre distance grid of ribs supported on isolated 40x40 cm^2 RC columns. On the exterior perimeter of the slabs, spandrel beams of 20x35 cm^2 are embedded on the plate. It is not considered the presence of hollows necessaries to exploit the building such us lift-facility shafts, stairwells or interior courtyard of the waffle slab. RC solid slabs around the column were included to improve the transmissions of shear and flexural stresses between slab and columns, with minimal size of 1/6 of the length of the next free span measured from the face of each column. These solid slabs were reinforced against punching failure with stirrup-type slab shear reinforcement.

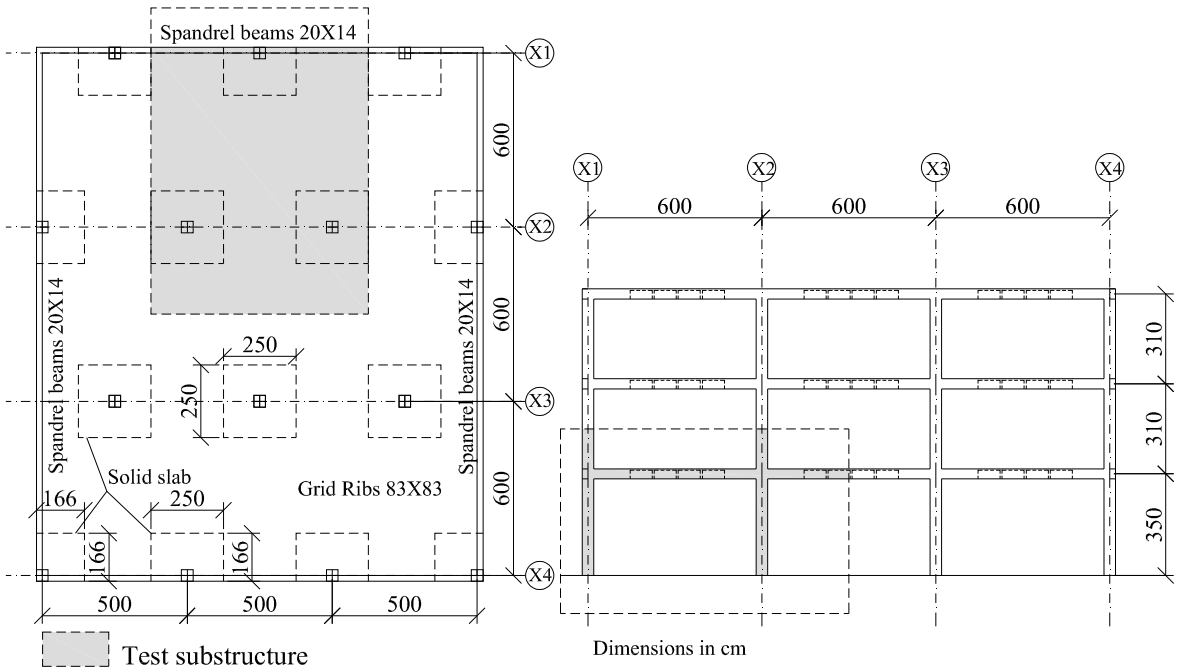


Figure 1. Prototype structure (left: plan; right: elevation in the Y direction)

The behavior factor adopted for the seismic design was q = 2.0, according to the Spanish seismic code that considers WFS systems as low ductile structures. The resulting base shear force coefficient of the prototype was 0.33.

A sub-structure of the prototype formed by a portion of the first floor slab and columns from the first (ground floor) and second floor (Fig.1) was selected, cutting the plate through points where the bending moment under lateral loads is approximately zero. From this sub-structure, a test specimen was defined by applying scale factors of λ_L= 2/5 for length, λ_a= 1 for acceleration and λ_σ= 1 for stress. The reinforcing details of the tests specimen are as follows. The area of longitudinal

reinforcement A_{sl} to the gross section A_g of the columns, A_{sl}/A_g, was 0.031 and 0.024 for the exterior and interior columns of the first floor, respectively, and 0.018 for all columns of the second floor. The volumetric ratio of shear reinforcement cast through stirrups, $\rho_{st} = V_{st}/V_{cc}\rho_{st} = V_{st}/V_{cc}$, was 0.03 for interior columns of the first floor and 0.02 for the rest, where V_{st} is the volume of the shear reinforcement and V_{cc} is the volume of the concrete confined by the shear reinforcement. The plate was designed with a base reinforcement of 1Ø6 (one rebar of 6 mm diameter) at top and bottom of the slab and in both directions. Some additional reinforcement with Ø6 and Ø8 mm of diameter was considered on the solid slab around each column. The yield stress of the rebars obtained through tension tests was 525 MPa for Ø8, 543 MPa for Ø6 and 656 MPa for the stirrups. For concrete, the compression tests carried out on the 28th day and the day of the shake-tests gave 39 and 43 MPa, respectively. This overstrength respect to the design value (25 MPa) is probably due to the chemical admixtures used to make the concrete more fluid and facilitate the casting.

The specimen was placed on the uniaxial MTS 3x3 m^2 shake-table of the University of Granada (Spain) as shown in Fig.2 and Fig.3. To represent the gravity loads acting on the floors and to satisfy the similitude requirements between prototype and test model, steel plates were placed at the top of the plate and at the top of the half columns of the second story (Fig.2). Pin joints connections were used at the top of the columns and at the extreme of the plate where the bending moments are considered zero with lateral loads.

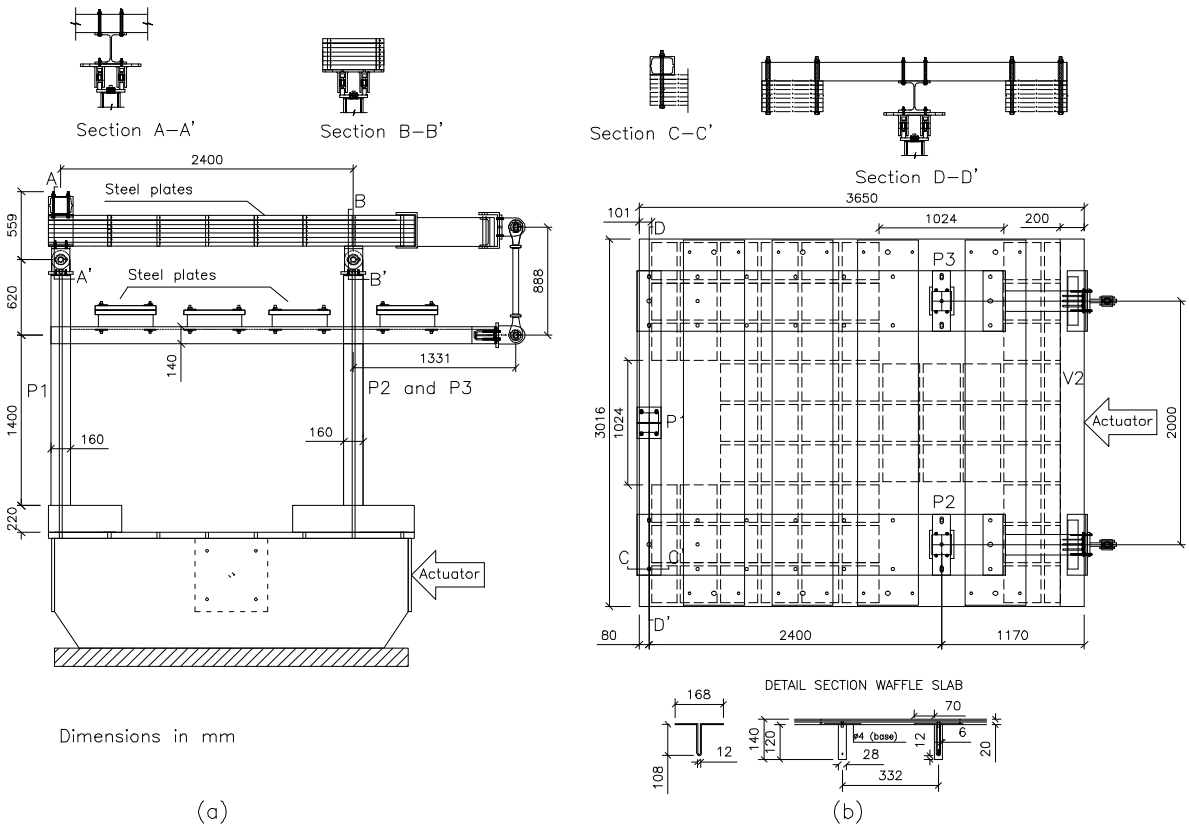


Figure 2. Test setup: (a) elevation; (b) plan

Figure 3. General view of the specimen on the shake-table

The total mass of the test specimen (without the foundation) was 11005 kg. The specimen was instrumented with 204 strain gauges, 11 uniaxial accelerometers and 20 displacement transducers (LVDTs). The strain gauges were located at the end of the columns and at the sections of the plate with references shown in Fig.4.

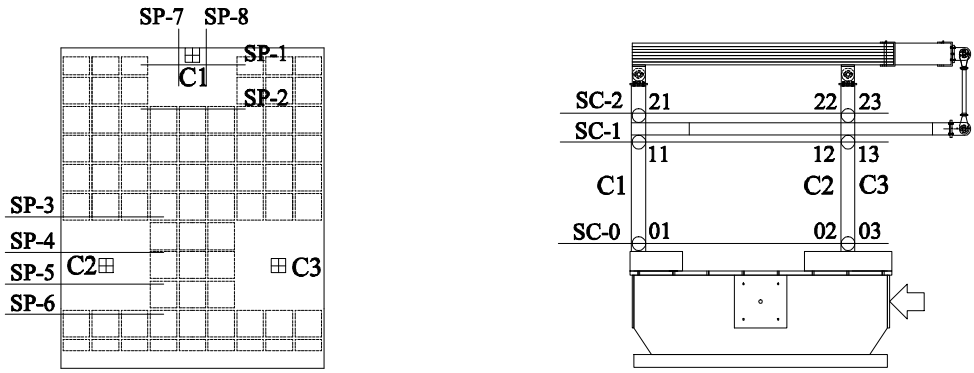


Figure 4. Sections with strain gauges

SEISMIC SIMULATIONS

The specimen was subjected to the Campania-Lucania earthquake (1980) recorded at the Calitri station scaled in time by the factor $\lambda_t = \sqrt{\lambda_L/\lambda_a} = 0.63$. The specimen was subjected to four seismic simulations in which the accelerations of the record were scaled to 100%, 200%, 300% and 350% of the original values. The corresponding seismic simulations are referred to as C100, C200, C300 and C350, respectively, hereafter, and their PGAs are 0.16g, 0.31g, 0.47g and 0.55g respectively. Fig.5 shows the accelerogram for C100. Each seismic simulation C100, C200, C300 and C350 represents a different seismic hazard level, SHL, at the location of the prototype (Granada), that will be referred to hereafter as SHL-1, SHL-2, SHL-3 and SHL-4, respectively. SHL-1 represents a 'frequent' earthquake, SHL-2 a 'rare earthquake' and both SHL-3 and SHL-4 represent a 'very rare earthquake'. The return period, P_r associated to each SHL is 139, 726, 2985 and 4421 years, respectively. These return periods are calculated according to a relation between P_r and PGA proposed by the Spanish code NCSE-02

(Ministerio Fomento, 2003) for soft-stiff soil with 400 m/s ≥ v_s ≥ 200 m/s (Type III), where v_s is the shear-wave velocity in the upper 30 m of ground.

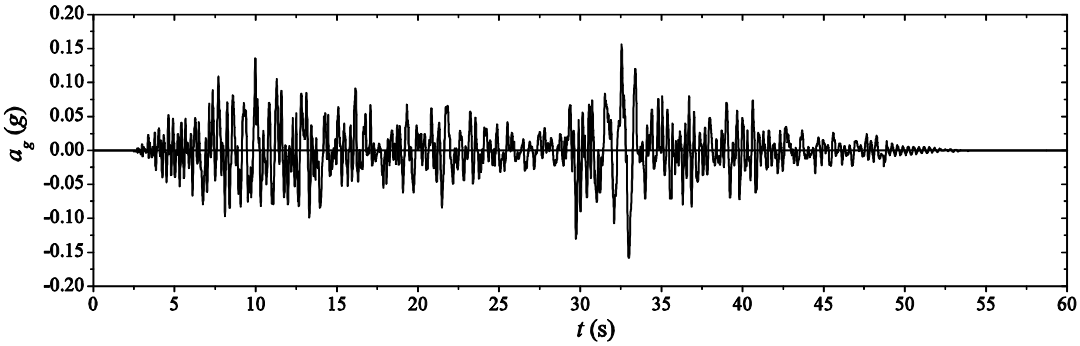
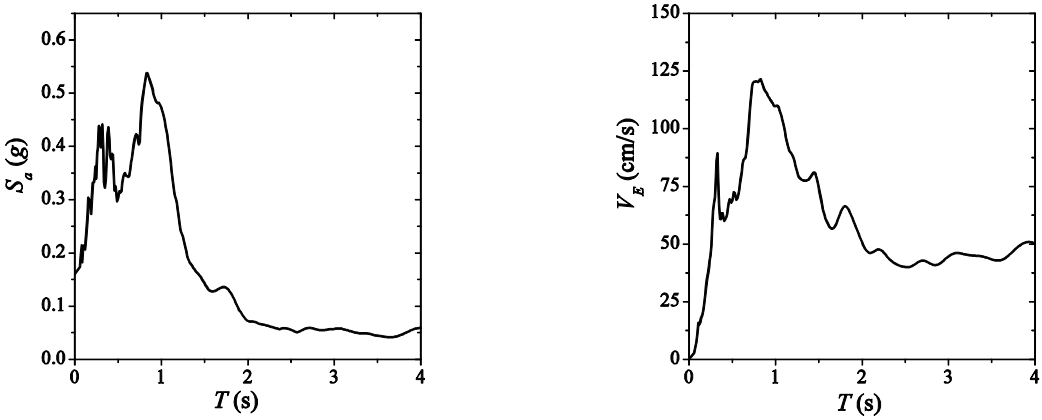


Figure 5. Accelerogram for the seismic simulation C100

Fig.6 shows the 5% damped spectra of the (time-scaled) accelerogram used in seismic simulation C100, in terms of acceleration response, S_a and relative energy input, E_I, (Uang and Bertero 1990) expressed as equivalent velocity, V_E defined by:

$$V_E = \sqrt{2E_I/M} \tag{1}$$

where M is the total mass of the structure.



(a) (b)

Figure 6. Elastic spectra for C100 (ξ = 0.05): (a) acceleration response; (b) input energy

TEST RESULTS

Dynamic characterization

For each seismic simulation, Table.1 shows the first and second vibration periods, T_1 and T_2, and the damping ratio, ξ, at the end of the test. The damping ratio was obtained from the displacement history of each story when the shake-table acceleration was zero but the test specimen was still moving in free vibration, using the logarithmic decrement method. T_1 and T_2 were calculated idealizing the test

specimen with a lumped system with two masses, each one with a single degree of freedom (the horizontal displacement in the direction of the shaking), through an eigenvalue analysis using the tangent stiffness matrix. It can be seen in the Table.1 that during seismic simulations C100 and C200 T and ξ remained basically constant, very light or light minor damage. Nevertheless, after seismic simulation C300, the periods enlarged to 70% and ξ about 3 times. After seismic simulation C350 the period enlarged two times and ξ four times. Therefore, the meaningful damage occurred at the two last simulations, with a drop in the lateral stiffness of up to about 25% (=100x$0.33^2/0.65^2$) respect to the initial value.

Table 1. Response parameters of the specimen

					Story 1			Story 2			Top			
Seismic simulation	SHL	T_1 (s)	T_2 (s)	ξ (%)	$\ddot{u}^t_{max}$ (g)	ID (%)	ID_r (%)	$\ddot{u}^t_{max}$ (g)	ID (%)	ID_r (%)	ID (%)	ID_r (%)	SPL	SEAOC ID (%)
Prior test		0.33	0.09	1.78										
C100 (end)	SHL-1	0.33	0.09	1.78	0.34	0.60	0.00	0.46	0.31	0.00	0.51	0.00	O	< 0.5 ±
C200 (end)	SHL-2	0.38	0.12	2.47	0.55	1.12	0.01	0.72	0.78	0.12	0.95	0.03	LS	< 1.5 ±
C300 (end)	SHL-3	0.53	0.16	6.29	0.61	1.93	0.07	0.95	1.33	0.22	1.62	0.02	NC	< 2.5 ±
C350 (end)	SHL-4	0.65	0.19	7.82	0.83	3.49	0.21	0.98	2.12	0.36	2.88	0.04	C	≥ 2.5 ±

Overall response

To analyze the dynamic response of the test specimen it was modelled as a two-degree-of-freedom system of concentrated masses in which the waffle-slab, m_1, and the block steel plates located at the upper part of the columns, m_2, are considered as rigid diaphragms with a mass of m_1 = 4782 kg and m_2 = 6223 kg, respectively. A degree of freedom consisting of the horizontal translation in the direction of shaking, u_i, is assigned to each concentrated mass. The areas of possible plastic deformations are located: (i) at the end of the columns (labelled with an identification number k in Fig.4), (ii) at the slab in the vicinity of the columns, and (iii) at the connection of the ribs with the solid slab. With this model, the equation of dynamic equilibrium of the specimen is as follow:

$$\mathbf{m}\ddot{\mathbf{u}}^t + \mathbf{c}\dot{\mathbf{u}} + \mathbf{F_s} = \mathbf{0} \qquad (2)$$

where $\mathbf{m}$ is the mass matrix, $\ddot{\mathbf{u}}^t$ is the vector of absolute accelerations, $\mathbf{c}$ is de damping matrix, $\dot{\mathbf{u}}$ is the vector of relative velocities, and $\mathbf{F_s}$ is the vector of restoring forces exerted by the structure. The total shear force $F_{I,B}$ exerted by the inertial forces $\mathbf{F_I} = \mathbf{m}\ddot{\mathbf{u}}^t = -(\mathbf{c}\dot{\mathbf{u}} + \mathbf{F_s})$ at the base of the structure can be calculated as $F_{I,B} = \mathbf{F_I^T}\mathbf{1}$, where $\mathbf{1}$ is the unit vector.

Fig.7 shows the $F_{I,B}$-δ_T curve for each simulation, where δ_T is the top displacement of the structure respect to the shake-table. The curves corresponding to seismic simulations C100 and C200 (Fig.7 (a) and (b)) show a stable response with minor plastic excursions and a low amount of energy absorbed/dissipated by the structure. This is consistent with the observation of low values for ξ, and constant values for T. In contrast, the curves corresponding to seismic simulations C300 and C350 (Fig.7 (c) and (d)), show that the structure entered in the plastic range. In these cases, it is observed a degradation of the lateral stiffness, and higher values for both $F_{I,B}$ and δ_T. The maximum value of the shear base $\alpha_B = F_{I,B}/\sum m_i g$ attained by the specimen was 0.83 at the simulation C350, where $\sum m_i = 11005$ kg is the total mass of the specimen.

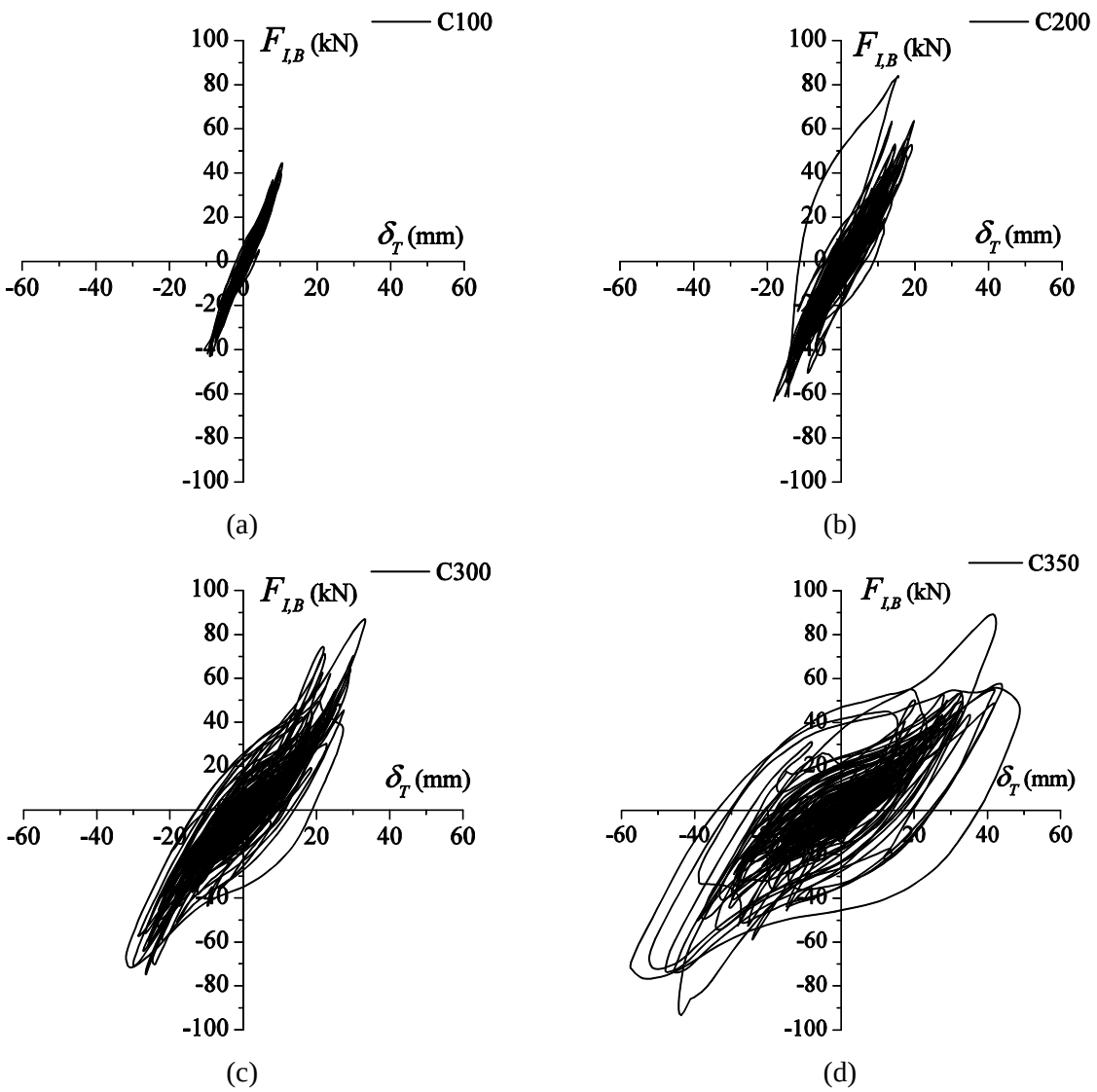


Figure 7. Base shear force versus top displacement: (a) C100; (b) C200; (c) C300 and (d) C350

The seismic simulation C100 corresponds to a moderate level earthquake (SHL) for the building site, that is, an 'occasional' earthquake, with a P_r between 75 and 200 years. After this level, referred to as SHL-1, the specimen showed light cracking on the top of the plate, flexural cracks at the ribs-solid slab connection of the exterior column-plate connection and tension cracks on the exterior column. The strains in the reinforcing bars at the base of the columns remained below $0.9\varepsilon_y$, where ε_y is the yield strain of the steel . Maximum strains in the rebars in the slab (solid slab, ribs and spandrel beams) was $0.4\varepsilon_y$ for all members. The maximum inter-story drift *ID* reached 0.60% of story height at first floor and the maximum acceleration $\ddot{u}^t_{max}$ was $0.34g$ at second floor (Table.1). According to SEAOC (1995), the *ID* attained is in the boundary between the seismic performance level, SPL, of 'operational' (O) $(0.2 \leq ID < 0.50)$ and 'life safe' (LS) $(0.5 \leq ID < 1.50)$. It must be noted that the limiting values for the interstory drift given by SEAOC have a tolerance, ΔID (symbol ± in Table.1), which depend on the lateral stiffness of the structure. In this case a ΔID of +0.1% is judged reasonable owing to the known flexibility of the WFS under lateral loads. The value of P_r for the seismic simulation C100 (139 years) is close to the mean return period of seismic action (95 years) which is proposed by Eurocode 8 (CEN, 2004) for structures of ordinary importance. This code recommends that the structure should respond in the SPL, 'damage limitation' (DL), which is characterized by $ID \leq$ 0.50% if the story has brittle non-structural elements attached to the structure. Therefore, the specimen did not fulfill this requirement.

The seismic simulation C200 represents a strong ground motion (the design earthquake) at the building site, that is, a rare earthquake with a P_r of about 500 years. This level is referred as SHL-2. A visual inspection of the specimen after this test, revealed flexural cracks on the solid slab and torsional cracks on the spandrel beams both at the exterior column-plate connection. Moreover, the ribs-solid connections of the interior column-plate connections were slightly damaged too. Plastic hinges with ductile flexural yielding developed at the base of the interior columns, showing maximum strains that ranged between $1.16\varepsilon_y$ and $1.30\varepsilon_y$. However, significant damage was not observed, being repairable in all cases and keeping the structural capacity to carry the gravity loads perfectly. It is worth noting that beside the cracks on the spandrel beam, the longitudinal reinforcement remained within the elastic range, with maximum strain $<0.60\varepsilon_y$ though with higher values than the longitudinal rebars of the nearby solid-slab. Permanent deformation of ID_r= 0.12% was registered at the second floor and negligible at the first floor. Those values are within the range ($0 \leq ID_r < 0.50$), proposed by SEAOC for LS. The maximum values of *ID* and $\ddot{u}_{max}^{t}$ were 1.12% of story height at first floor and $0.72g$ at second floor, respectively. According to SEAOC, this *ID* is clearly within the range of the SPL of LS ($0.5 \leq ID < 1.50$), and far of 'near collapse' (NC) ($1.5 \leq ID < 2.50$). The SPL of LS and NC is the counterpart of the SPLs called 'significant damage' (SD) and 'near collapse' (NC) by Eurocode 8 (CEN, 2004).

The seismic simulation C300 represents a very rare or the maximum considered earthquake, with values of the mean return period in the order of more than 2000 years. Under this earthquake, referred to as SHL-3, the structure had significant damage. Cracks at the base of the columns were observed and even spalling of the concrete in one of the interior columns. Significant cracking was observed at top and bottom of the plate. The maximum strains of the longitudinal reinforcement at the base of the columns oscillated between $1.4\varepsilon_y$ and $3\varepsilon_y$. The damage on the spandrel beams of the exterior slab-column connection that initiated in previous simulations was widely increased. The maximum strains of the longitudinal rebars of these spandrel beams were near to ε_y. The low torsional capacity of the spandrel beams (T = 1.78 kN·m, of according to the ACI 318-11 (2011)) limited the strains in the longitudinal reinforcement anchored in the spandrel beam (Benavent-Climent et al. 2009). Damage concentrated on the exterior slab-column connection and at the base of the columns, while the interior columns remained apparently undamaged, with maximum strains of the rebars $<\varepsilon_y$. The maximum *ID* was 1.93% of the story height at first floor, within the limited range for NC performance levels proposed by SEAOC. The maximum acceleration attained $\ddot{u}_{max}^{t}$ was $0.95g$ at second floor. The permanent deformation of ID_r= 0.22% was registered at the second floor and 0.07% for the first floor, within the LS according SEAOC. According to the damage observed, the structure could be repairable again, although the cost of this action could advise the demolition of the structure and rebuild it.

The seismic simulation C350 represents a very rare earthquake with a P_r= 4421 years. The specimen was heavily damaged under this seismic action. Damage mainly concentrated on the plastic hinges of the column bases and at the spandrel beams of the exterior slab-column connection. The former, with clear spalling of the concrete (Fig.8 (a)), with strain reinforcement values attaining up to $7\varepsilon_y$. The last with generalized crushed and even the spalling of the concrete in the anchor area of the solid slab reinforcements, leaving the rebars of both the spandrel beams and the solid slab without concrete cover (Fig.8 (b)). The strain of the longitudinal rebars of the spandrel beams attained increased up to $1.2\varepsilon_y$. Some reinforcement of the solid-slabs and columns of the interior connections yielded, attaining maximum strain of $1.26\varepsilon_y$ and $1.30\varepsilon_y$. Yielding was also observed in some rebars at the connection of the ribs with the solid slab of the interior columns. After this simulation, the stability of the specimen kept jeopardized, though the collapse did not occur. The maximum *ID* attained was 3.49% of the story height on the ground floor, the ID_r = 0.36% at the second floor and the maximum value of $\ddot{u}_{max}^{t}$ was $0.98g$ at the second floor. This *ID* is associated with collapse (C) according to SEAOC (*ID* > 2.5% and ID_r > 2.5%).

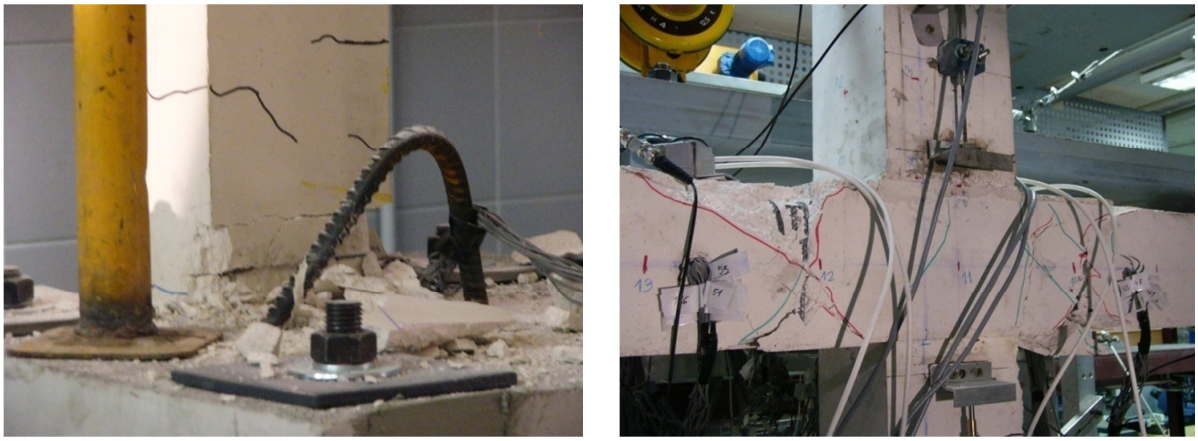


Figure 8. C350 damages: (a) base of exterior column; (b) spandrel beam

CONCLUSIONS

This study has shown the main results obtained from shake-table tests conducted on a two-fifths-scale RC waffle-flat plate structure designed according to current Spanish seismic code. The specimen was subjected to four seismic simulations representative of frequent, rare and very rare earthquakes associated with return periods of 139, 726, 2985 and 4421 years, respectively, in the Mediterranean area. The results of the tests lead us to put forth the following conclusions that are transferable to similar structures.

The WFS tested showed a satisfactory behavior for seismic hazard levels corresponding to 'frequent' and 'rare earthquake', for which damage concentrated at the base of the columns of the ground story. For 'rare earthquakes' the structure experienced large inter-story drifts and brittle breakage of the spandrel beams of the exterior slab-column connections. This can produce heavy damage on facilities and non-structural elements and jeopardize the stability of the structure.

ACKNOWLEDGEMENTS

This work received financial support from the Spanish Government under project BIA2011-26816 and from the European Union (FEDER).

REFERENCES

ACI (2011) Building Code Requirements for Structural Concrete (ACI 318M-11) and Commentary, Farmington Hills, MI (USA): American Concrete Institute

Benavent-Climent, A., Cahis, X. and Catalan, A. (2009) "Reinforced Concrete Exterior Waffle Flat Plate-Column Connections Subjected to Lateral Earthquake Loading" *Journal of Earthquake Engineering*, 13(3):275–292

CEN (2004) European Standard EN 1998-1:2004 Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, Brusells: Comite Europeen de Normalisation.

Ministerio Fomento (2003) Norma de Construcción Sismorresistente. Parte General y Edificación (NCSE-02), España: Centro Publicaciones. Secretaria General Técnica. M.Fomento

SEAOC (1995) Vision 2000: Performance Based Seismic Engineering of Buildings (VOL. 1), prepared by Vision 2000 Committee, Sacramento (USA)

Uang, C.-M. and Bertero, V.V. (1990) " Evaluation of seismic energy in structures" *Earthquake Engineering & Structural Dynamics*, 19(1):77–90