VULNERABILITY / SEISMIC PERFORMANCE OF OLDER RC WAFFLE FLAT-PLATE SYSTEMS IN A LOW-TO-MODERATE-SEISMICITY REGION

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ABSTRACT :

The objective of this research is to investigate the seismic performance of waffle-flat-plate floor systems engineered to resist low earthquake excitations in low to moderate seismicity regions. The models investigated represent structures built between the seventies and nineties in Spain, according to earlier seismic codes that did not provide any requirement for attaining ductility. Scaled test specimens constructed and tested under quasi static cyclic loading made it possible to clarify the hysteretic behavior of this type of structure. Based on the experimental results, a numerical model is proposed and calibrated, approximately representing the hysteretic behavior of the waffle flat-plate column connections. Using this numerical model, the seismic behavior of a 6-storey, 3-bay and 3-span building is investigated through nonlinear dynamic response analysis. A Performance Based Earthquake Engineering (PBEE) approach is applied, and the response of the building under five historical accelerograms is discussed. The results of the analysis confirm the excessive lateral flexibility and the limited usable ductility of this type of system.

KEYWORDS: waffle flat-plate structure, PBEE, hysteretic behavior, seismic resistance; column-slab connection.

1. INTRODUCTION

This article is part of a very extensive investigation targeted at evaluating the seismic behavior of waffle flat-plate structures constructed in the area of Granada (Spain) before 1994, according to the former seismic code PDS 74 [1]. The complete description of the prototypes and test models together with the experimental results can be found elsewhere [2,3].

Reinforced concrete (RC) structures with waffle flat-plate systems have been utilized (and still are at present) extensively in the field of construction in Spain. Studies and laboratory tests focusing on this type of system are very limited, yet they a have demonstrated in general that these structures wield a low capability of energy dissipation and a high lateral flexibility when subjected to earthquakes [4,5].

One very important disadvantage of these structures in comparison to conventional RC frames is the reduced stiffness of the plate-column connections, which intensifies problems deriving from the P-δ effect. Moreover, the fact that shear is concentrated excessively around the columns (even if solid zones are used) can cause local punching shear failure and induce the collapse of the floor slab (soft floor).

Many of the waffle flat-plate structures built in Spain were designed according to the earlier seismic code PDS 74. This Code required low lateral forces as compared to current codes, and did not provide any requirement for attaining ductility. The evaluation of the ultimate earthquake resistance of existing waffle flat-plate systems is a very important issue. Also important is the core decision of whether they need to be seismically upgraded, and which is the true level of safety. Besides, the excessive lateral flexibility of this type of structure can damage the non-structural elements. In this study, a 6-storey waffle flat-plate system is modelled based on the results of the test accomplished at the Laboratory of Strength of Materials and Structures of the University of Girona. Next, nonlinear analyses are carried out applying the Performance Based Earthquake Engineering (PBEE) approach, which considers distinct levels of earthquakes according to its return period. This methodology permits evaluating the behavior of the structure under different load conditions, and leads to important conclusions with regard to its ultimate strength, deformability and damage level.

2. MODELLING THE WAFFLE-FLAT-PLATE COLUMN CONNECTIONS

Two waffle-flat-plate column connections tested in the Laboratory of Strength of Materials and Structures of the University of Girona [2,3] were modelled as a grid of virtual T-beams spanning in both directions. A lumped plasticity model was used for every bar (columns and beams). The predicted bending moments and curvatures at cracking, yielding and ultimate states were obtained from approximate formulas proposed by Kunnath and Reinhorn [6][7][8]. The hysteretic model adopted was defined by four parameters [6] that account for the degradation of stiffness (K_1), the degradation of strength controlled by ductility (K_2), the degradation of the strength controlled by dissipated energy (K_3), and the slip effect or pinching (K_4).
The adoption of different values for these parameters determining the hysteretic model for the T-beams and columns lead to the predicted hysteretic curves shown in red in Fig. 1. The prediction is compared with the experimental curves in black.

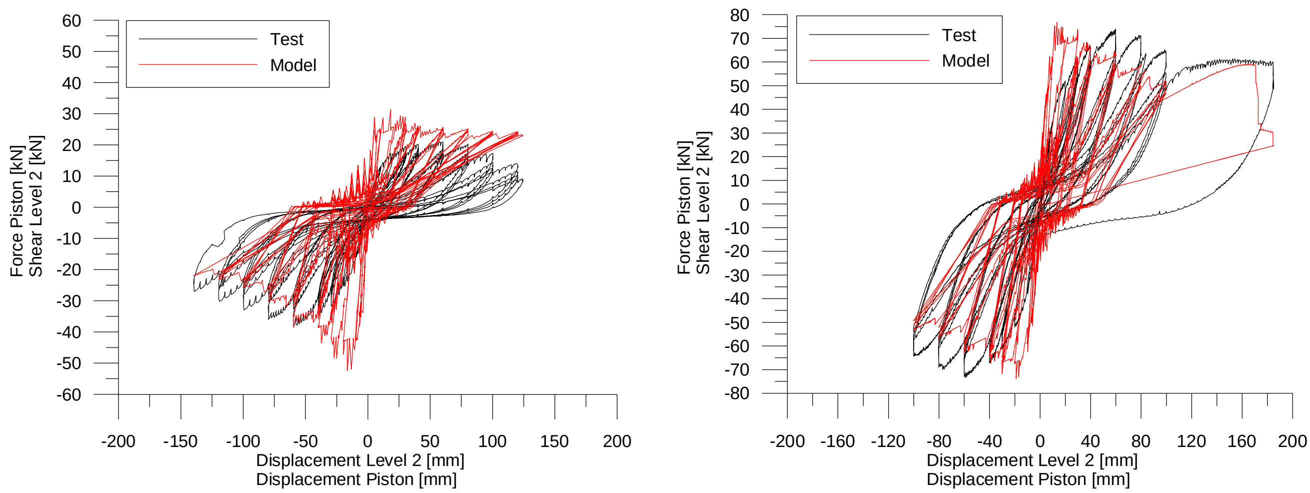


Fig. 1. Outside connections (left) and inside connections (right)

3. ANALYSED BUILDING MODEL

The numerical model developed and calibrated with the experimental results was used to model a prototype building. The prototype represents an RC waffle-flat plate structure designed to sustain the gravitational, seismic and wind loads prescribed by earlier Spanish codes PDS 74 and EH-91 [9] (Fig. 2). These codes were replaced by the current NCSE 02 [10] and EHE [11] Codes. The building is located in the region of Granada, upon medium soil conditions. It comprises six storeys 3m high, and 16 square columns supporting six waffle flat-plates. The sections of columns are progressively reduced in height from 55x55 cm to 30x30 cm. Besides the self-weight, uniformly distributed dead loads and live loads of 1.0 kPa and 3 kPa respectively, were applied in all stories except in the roof where these values are 3.0 kPa and 1.5 kPa, respectively. To be consistent with PDS 74, the structure was detailed without any special provision for attaining ductility. Structural concrete was specified for a 28-day compressive strength of 17.5 MPa and steel type AEH 400 (yield strength of 400 MPa). The structure will be examined only for the earthquake acting in the X direction (X). The elevation and plan of the building can be seen in Fig. 2. Although the building is analysed in one direction (X), a 3D model is used to represent the structure. An equal displacement condition may be imposed at each floor level.
Initially a thorough modal analysis of the structure was conducted using the software IDARC6.1 [6]. The eigenvalues for the X direction of study are shown in Table 1. In order to verify and to calibrate relevant parameters of the model (effective slab width of the beams, stiffness of connection beams, etc.), an independent modal analysis was carried out using another Finite Element Method (FEM) program. The results are shown in Table 2 for three modes, evidencing reasonable adjustment.

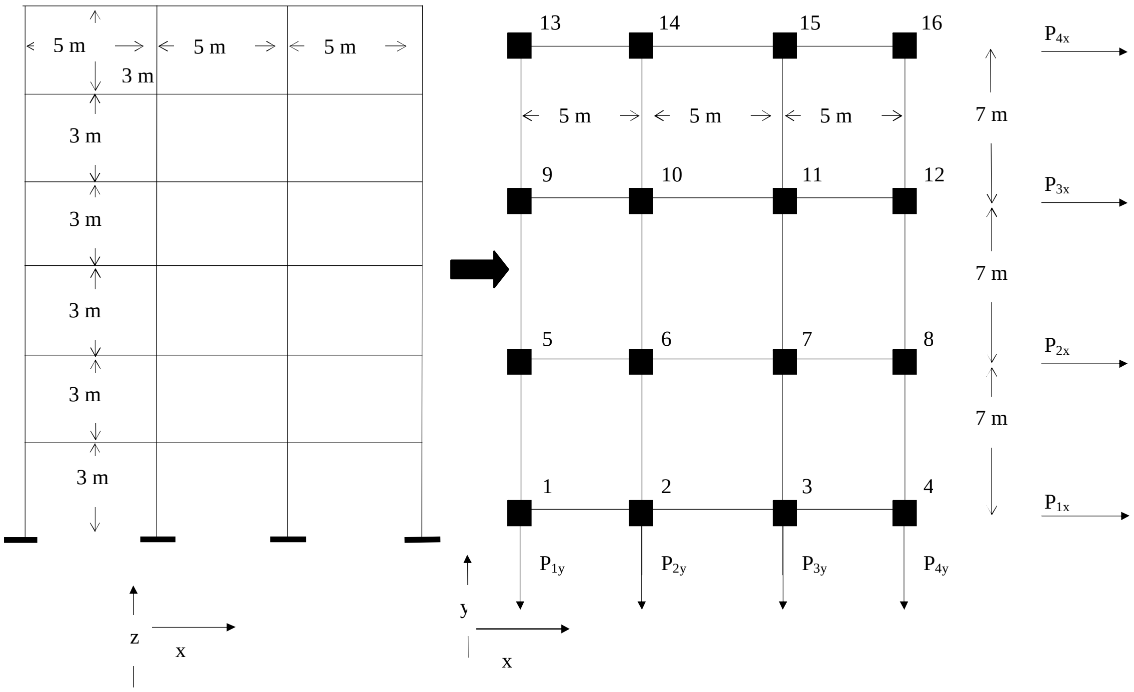


Fig. 2. Structural scheme. Frame type (left) and plant (right)

Table 1. Modal analysis of the structure (at brute sections)

Mode	Frec. [Hz]	Period [seg]	Participation modal factor	Weight Modal [kN]	Rel. weight Modal [%]
1	0.77221	1.29510	0.8797	7591.612	71.866
2	2.64240	0.37844	0.3671	131322.063	13.689
3	4.47435	0.22350	0.2605	665.556	6.388
4	6.03238	0.16577	0.1821	325.407	3.123
5	7.32609	0.13650	0.0846	70.217	0.674
6	11.45722	0.08728	0.2127	443.705	4.259

Table 2. Model verification with modal analysis of the structure

Mode	Period [seg]	Period FEM [seg]	Diference [%]
1	1.29510	1.20	7.90
2	0.37844	0.45	18.90
3	0.22350	0.21	6.43

4. THE PERFORMANCE-BASED EARTHQUAKE ENGINEERING CHARACTERIZATION

Herein, a methodology based on the well-known Performance Based Earthquake Engineering (PBEE) approach is utilized. To this end, it is necessary to establish certain probabilistic seismic levels, so as to apply coherent loads, obtain structural responses, and analyze the response in relation to control limits. The first problem is to establish the level of study, the load corresponding to each seismic level, and an index of control that might be used to evaluate whether the response is satisfactory or not. The situations can be different in one case or another. For example, in a region of low-to-moderate seismicity such as Spain, no damage should be tolerated in the event of frequent earthquakes. The objective for the engineer may be different from that of the builder or owner. One point of view encompasses the engineer's perspective and another the owner's [12]. For the former, what matters are the structural yielding, buckling, cracking, permanent or no permanent deformations and the general damage that the structure experiences. For the owner, the most important problems surround occupancy and the cost of repairs (including the time involved). Following FEMA [12,13,14] guidelines, the PBEE is based on four limit states or performance levels defined as follows:

1. Operational or Serviceability [O]: Elastic behavior. Plastic hinge is not produced. Little cracking, and limited displacements.
2. Immediate Occupancy [IO]: Plastic hinges are produced but they are few and occur only in the beams. Cracking is limited. Moderate displacements. Minor damage. Immediate occupation is permitted.
3. Life Safety [LS]: Plastic hinges are produced in beams and columns, but many affect the beams. Generalized cracking. Important displacements. High damage. Immediate occupation is not permitted. Structure may be repairable (long-term). The structure can not collapse. This limit state is the level foreseen in the current Spanish Code NCSE 02 for a return period of 500 years.
4. Collapse Prevention [C]: Final level of collapse. Exceptional earthquake. Ductile and progressive collapse.

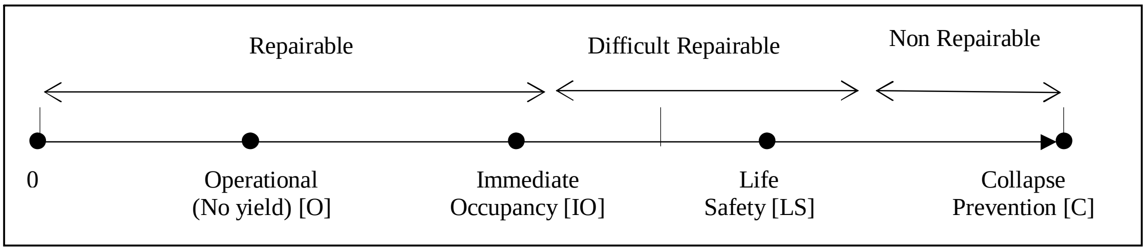
It is then necessary to characterize each level in order to quantify the corresponding seismic load. Figure 3 shows the characterization adopted for this study. It is based on two peak ground accelerations, a_b, and a_c , determined by the Spanish Seismic Code NCSE 02 for the region of Granada:

$$a_b=0.24g \quad ; \quad a_c= 0.245g \tag{1}$$

and the following relationships between intensity, I, a_b and the magnitude M of the design earthquake inferred by the Spanish Instituto Geográfico Nacional:

$$I=[3.2233+\log(a_b/g)]/0.30103 \tag{2}$$

$$M=0.552\cdot I+1.34 \tag{3}$$



Earthquake type	Very frequent	Frequent or Occasional	Rare	Very Rare
Probability	100%/50 years	50%/50 years	10%/50 years	2%/50 years
T_r (Return Time)[year]	50	100	500	2500
DI $_{PA}$*	0.10	< 0.40	> 0.40	1.00
Acceleration (a_c) NCSE 02	0.10 g	0.135 g	0.245 g	0.444 g
I (Intensity)	5.84	6.28	7.14	8.00
M (Magnitude)	4.56	4.81	5.28	5.76

* DI_{PA}: Damage index Park-Ang

Fig. 3 Applied levels of PBEE

Once the levels of seismic loads are established, the response limits adopted for each level must be set. These limits are determined by a Control Index. The Operational level (O) should not admit any damage, and the structure should always be in the elastic range. In Collapse (C), all kinds of damage are allowed and a resistance limit for the structure is fixed. However, the IO and LS levels give rise to further alternatives and it is more complex. The control indexes adopted here for defining the IO and LS levels reflect the yielding or plastic behavior, the Park and Ang damage index (DI_{pa}), the roof drift (Δ_{roof}), the interstory drift (Δ_{di}) and the global ductility (μ). In this study the values adopted for each performance level are summarized in Table 3. Some of these values are from Eurocode 8; others come from FEMA.

Table 3. Considered seismic levels and Control index.

State	Yielding behavior	ID_{pa}	Δ_{roof} [$\%H_t$]	Δ_{di} [$\%h_i$]	$\mu = \delta_{6m}/\ \delta_{6y}$
O	No	≤ 0.10	≤ 0.10	≤ 0.20	1.00
IO	No or very few in beams	≤ 0.15	≤ 0.20	≤ 0.30	1.00-1.50
LS	Yes, many distributed in beams	≤ 0.60*	≤ 0.45	≤ 0.50**	1.50-2.00^{+}
C	Yes, distributed everywhere	$\leq 2.00^{++}$	≤ 5.00	≤ 0.80	$> 2.00^{++}$

* Assuming the situation of DIpa > 0.40 (difficult reparation)

** Depending on the type of partitions used: values until 0.2-0.3 % for brick masonry infill and until 0.5 % for panels light or plywood or plaster or similar.

$^{+}$ It depends on the stated value in structure's design.

$^{++}$ Not very important values

The values of roof drift (Δ_{roof}) of Table 3 were reduced in relation with those obtained in the tests so as to consider the influence of damage in the infill panels and to guarantee the global structural stability (second order effects). Table 3 points to a general criterion to be followed in relation to structural and non-structural damage for waffle-flat-plate structures: i.e. it is not a general criterion for all type of structures.

5. INCREMENTAL DYNAMIC ANALYSIS AND RECORDS

Various approximations may be applied in order to accomplish the PBEE study. It is commonly accepted nowadays that Incremental Dynamics Analysis (IDA) is one of the best methods. It entails an important computational cost, however, and it is not perfect. Each earthquake level can only be attained by applying records with a certain maximum acceleration. This involves scaling the real records and preserving, approximately, the distribution of frequencies). Minor error is permissible if the scaling constant, k, is within the 0.3-3.0 (recommended) range, a range sometimes surpassed in this study by some weak tremors used (Spain accelerograms). It is known that magnitude is a function of time and of maximum acceleration. In this work, all records of one same earthquake were considered in conjunction with equal time. The quantity of records is also a very important matter in this type of analysis. The non linear behavior of a structure is very dependent on the type of record used, for which reason, it is recommended to utilize several records. These records must represent “possible” accelerograms for the structure. Assuming that the structural response is always of a probabilistic nature, in general, for a minimum of 3 records, the worst value of the responses can be utilized; and from 7 or more records it is considered appropriate to take the statistical media response of all of them [15]. Habitually it is preferable to utilize many records and to find the response on statistical bases [16][17]. There are sound studies in the literature involving 160 records. In this study just five records were used: three from Spain and two from California, as summarized in Table 4. The last two records try to cover the entire frequency range imposed by the design spectrum of Spanish Code NCSE 02. Al records used are real scaled records. No artificial records (spectrum compatible) were employed because it is know that they can produce excessive response when matching the envelope spectra of the codes .

Table 4. Input earthquake used for assessment

Earthquake	*Station*	*Comp.*	*Duration [sec]*	*Soil type*	*Observations*
Granada (1984)	Santa Fe	NS	10.06	Stiff soil	
Huelva (1989)	Cartaya	NS	21.29	Rock	Impulsive ground motion
Murcia (1999)	Lorquí	EW	21.00	Stiff soil	
ImperialValley (1940)	El Centro	NS	32.00	Stiff soil	Near Fault
Kern County (1952)	Nº 475	S90W	77.32	Soft soil	Very long duration

The chosen records have very different characteristics, representing the diverse possibilities that can present in a “possible earthquake allowing us to assess the response in statistical form. The data on their magnitude, acceleration, velocity, displacement, energy, etc. were left out because all of them will be scaled for utilization in the context of values of acceleration corresponding to each status or study level.

6. RESULTS AND DISCUSSION

Calculations were accomplished through a nonlinear dynamic response analysis. A great dispersion of results is observed. The earthquakes that induce highs loads at dangerous frequencies in the structure (El Centro and Kern County) cause the linear tendency to be lost in the interstorey drift. High modes of vibration might be also be present, as can be deduced from the storey-drift curves shown in Fig. 4. It is important to point out that the pushover method with a triangular load distribution, corresponding to the 1º mode of vibration, would not be adequate in this case.

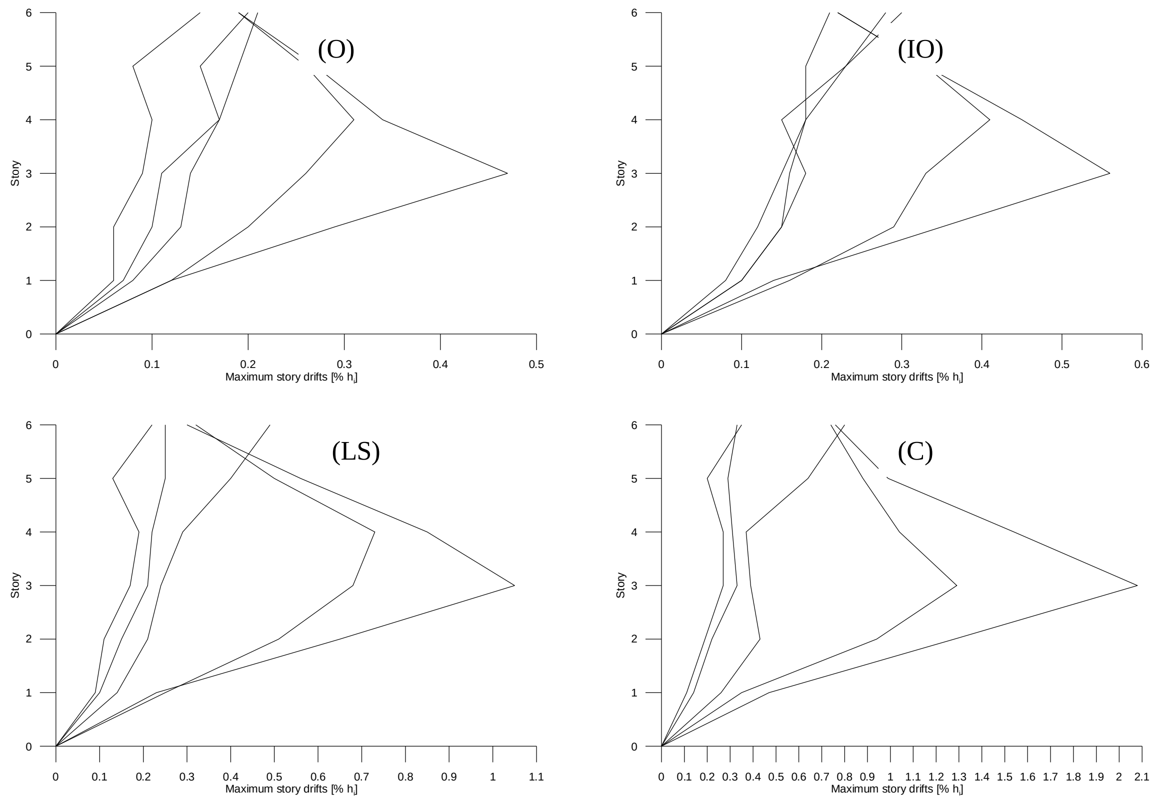


Fig. 4. Maximum interstorey drift in each seismic level

The yielding behavior obtained from the response analyses for each level is summarized in Table 5:

Table 5. Yielding behavior for each seismic level

State	Yielding behavior
O	Cracking. Yes, very few in beams
IO	Yes, very few in beam
LS	Yes, many distributed in beam
C	Yes, distributed everywhere

The results evidence that, for the earthquake level corresponding to an Operational state, a generalized yet limited in yielding (only in two hinges) is observed, which should not be permitted. In other levels, the behavior is adequate and relatively few plastic hinges exist.
Table 6 shows the statistical treatment of the results for each seismic level. The values that reach the limits are indicated in bold. From Table 6 important conclusions can be obtained. The values of DI_{pa} seem lower than expected, suggesting that the Park and Ang damage index is not well calibrated to the type of structure analysed in our study.

Table 6. Statistically significant values found for each seismic level

St.	Statistical	DI_{pa}	$\Delta_{roof}\%H_t$	$\mu = \delta_{6max} / \delta_{6y}$	$\Delta_{di\ 6}$	$\Delta_{di\ 5}$	$\Delta_{di\ 4}$	$\Delta_{di\ 3}$	$\Delta_{di\ 2}$	$\Delta_{di\ 1}$
O	*Mean*	0.055	0.11	1	0.188	0.19	0.218	0.214	0.156	0.09
	SD	0.0278	0.098	0	0.0228	0.0791	0.102	0.158	0.0907	0.0283
	Mean+SD	0.0828	**0.208**	1	**0.2108**	**0.2691**	**0.32**	**0.372**	**0.2467**	0.1183
IO	*Mean*	0.066	0.142	1	0.242	0.26	0.28	0.273	0.212	0.116
	SD	0.05	0.125	0.2	0.035	0.067	0.138	0.178	0.102	0.033
	Mean+SD	0.116	**0.267**	1.2	0.277	**0.327**	**0.418**	**0.451**	**0.314**	0.149
LS	*Mean*	0.13	0.25	1.496	0.316	0.368	0.456	0.47	0.326	0.162
	SD	0.117	0.248	0.56	0.1	0.177	0.31	0.384	0.24	0.074
	Mean+SD	0.247	**0.498**	**2.056**	0.416	**0.545**	**0.766**	**0.854**	**0.566**	0.236
C	*Mean*	0.223	0.44	2.33	0.596	0.6	0.706	0.872	0.672	0.266
	SD	0.22	0.45	1.88	0.235	0.35	0.56	0.79	0.479	0.149
	Mean+SD	0.443	0.89	4.21	**0.831**	**0.95**	**1.266**	**1.662**	**1.151**	0.415

Δ_{di} : interstorey drift of the i-th story [% h_i] H_t: total height roof

Another surprising result is that the ductility developed is small. This was obtained from the displacement (maximum/yield) of the higher level of the structure. However, for structures of the type investigated in this study (for which there is more than one important mode of vibration) estimating ductility from the displacement of the top storey) they may not be correct. Even so, the obtained values provide a reference as to the progressive variation of ductility for different seismic levels (about 2.00-2.10 on LS).

Roof drift and interstorey drift give values that reach the maximum adopted for each performance level. This occurs for all the seismic levels adopted, yet interpretation depends on the each case. For O and IO, it can be clearly seen that the computed drifts indicate more damage than could be accepted. In the first case, practically only non-structural damage is evidenced, and in the second one, structural and non-structural damage are expected. In both cases, this would have economical connotations in terms of the necessary reparation costs. In LS and C levels, the implication is different. In addition to the generation of damage (which is not the critical aspect now) the great story deformability endangers the stability of the global structure due to the development of the significant P-δ effects. The collapse associated with the development of P-δ effects must be always avoided. This becomes a crucial aspect in very flexible structures such as that investigated in this study. Under the current Spanish Seismic code [10] beyond Δ_{roof}=0.2 % H_t the P-δ effect must be of special concern. It is important to underline that in our study this value is reached in all cases. An alternative evaluation in terms of interstorey drift would give rise to the same conclusion [10,15].

8. CONCLUSIONS

A prototype waffle-flat-plate structure with 6 stories, 3 bays and 3 spans —representing buildings now standing in Spain– was designed according to earlier seismic codes. The corresponding 3D nonlinear numerical model was developed, adapted to the software program IDARC6.1. The parameters governing the hysteretic behavior of the waffle flat-plate column connections were calibrated on the basis of experimental results reported elsewhere. Nonlinear dynamic response analysis was carried out, the numerical model being subjected to five historical accelerograms. The application of Performance Based Earthquake Engineering (PBEE) helped us corroborate high level of storey deformation and poor usable ductility.
A preliminary conclusion of this ongoing research would be:

1. The hysteretic general model used in this study for representing the waffle-flat-plate column connections provides a reasonably good approximation to experimental results. Under this model, waffle flat-plate systems were represented by virtual T-beams and effective slab width methods were applied.
2. The roof drift and the inter-storey drift obtained in the analyses exceeded the maximum values adopted for each performance level. In the low levels (Operational and Immediate Occupancy), the drifts obtained would produce severe damage to both structural and non-structural elements. In highs levels of performance (Life Safety and

Collapse) the drifts obtained can lead to global unstability of the structure owing to P-δ effects.
3. Due to a very low lateral stiffness, these structures reached a generalized yielding state at very high deformations (i.e. at roof drifts of about 1.00-1.50 %), as reported on previous studies.
4. The most adequate calculation method for this type of structures should be based not on strength but rather on deformation, for all seismic levels considered. To this end, a spectrum of displacements and accelerations should be put forth through future research efforts.

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