Analysis of the slab-column connections of flat slabs for seismic action

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Abstract: The performance of slab-column connections has been studied over the last several decades by researchers, aiming to better understand the behavior of flat slabs subjected to punching shear loading conditions. As a result, the use of slab shear reinforcement has emerged as a practical strategy to improve both the strength and ductility of reinforced concrete flat slabs.

For this, studies are carried out on the influence of the ductility of the slab-column connections to resist seismic actions, considering the gravity loads and the drift capacity of the structure, following some indications of the american regulation.

The deformation limitation of the structure can be analyzed through damage control for serviceability conditions (ULS) defined on EuroCode8-1, or the ultimate imposed displacement limitation present on EuroCode8-3.

In order to apply the previously mentioned code outlines, 5 models were developed with structural elements with different geometric characteristics and lateral stiffness, to evaluate the seismic displacements of the structures and to verify the need to adopt punching shear reinforcement in order to increase the slab-column ductility connection during earthquakes.

Keywords: Seismic action, Drift capacity, Ductility, Punching shear, Slab-column connection, Shear reinforcement

1. Introduction

Reinforced concrete flat slabs are widely used in modern infrastructures due to their relatively simple construction.

For the dimensioning of reinforced concrete structures, the punching shear failure is one of the most complex behaviors because it occurs in a slab-column connecting zone, where there is a great concentration of bending and cutting efforts. However, this behavior is not properly studied in slab-column connections subjected to seismic actions.

EuroCode8-1 does not classify this structural system as a primary system resistant to seismic actions, so it is assumed that it must be dimensioned as a secondary structural system. In this way, the slab-column connection must have enough shear strength to withstand large deformations without punching shear [1].

The behavior of the flat slab system subject to lateral cyclic loads greatly depends on the hysteretic properties of the slab-column connection, according to the characteristics of the column and the slab. In the case of the column, it is recommended that the same type of ductility requirements is fulfilled for the primary elements, and for the slabs to have enough deformation capacity to ensured that the axial load transmitted during the earthquake.

2. PUNCHING SHEAR

In recent years, the mechanisms of punching shear resistance have been researched in order to better understand the mechanism of degradation of the slab-column connection when subjected to horizontal cyclic actions. An example of this is the Model Code 2010 (MC2010) [2] which has alternative approaches to Eurocode 2 (EC2) [3] regarding the evaluation of shear strength in flat slabs.

2.1 Eurocódigo 2

The punching shear resistance is based on an empirical formulation given by the equation:

$$v_{Ed} = \beta \frac{V_{Ed}}{u_1 \cdot d} \leq v_{Rd,c} \qquad (1)$$

Where, V_{Ed} is the acting shear force; u_1 is the basic control perimeter; d is the effective depth of the slab; β represents the non-uniformity of load transmission for the support.

The punching shear resistance [MPa], without shear reinforcement, may be calculated as follows:

$$v_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} \qquad (2)$$

Where, $C_{Rd,c} = 0{,}18/\gamma_c$ (γ_c=1,5); k is a factor that takes into account the scalte effect; ρ_l is the reinforcement ratio for longitudinal reinforcement; f_{ck} is the characteristic compressive cylinder strength of concrete at 28 days.

When shear reinforcement is required, it should be calculated in accordance with equation 3.

$$v_{Rd,cs} = 0{,}75 v_{Rd,c} + v_{Rd,s} \qquad (3)$$

The shear resistance provided by shear reinforcement may be taken as:

$$V_{Rd,s} = n_r \cdot A_{sw} \cdot f_{ywd,ef} \qquad (4)$$

Where, n_r represents the number of punching shear reinforcement lines; A_{sw} is the area of one perimeter of shear reinforcement around the column; $f_{ywd,ef}$ is the effective design strength of the punching shear reinforcement.

2.2 Critical shear crack theory and fib Model Code 2010

Muttoni et al. [4], based on work done by Nylander and Kinnunen [5], formulated the critical shear crack theory (CSCT) according to levels of approximation, in order to study the punching shear resistance behavior for flat slabs. This approach can be better estimated if devoted more time to their analysis [6].

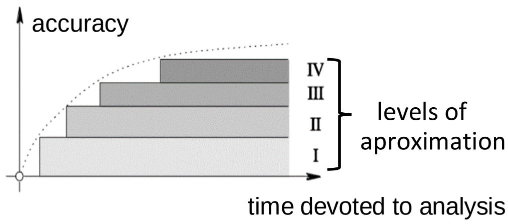

Figure 1 Expected accuracy as a function of time developed to analysis [6]

According to this theory, the concrete contribution is estimated from the assumption that the critical shear crack develops within a 'failure zone'. The contribution of the shear reinforcement is determined using the main hypothesis of this theory, which states that the width of the critical shear crack (w) is proportional to the product of the effective depth (d) of the specimen times the rotation of the slab (ψ) [4], [5], [6], [7].

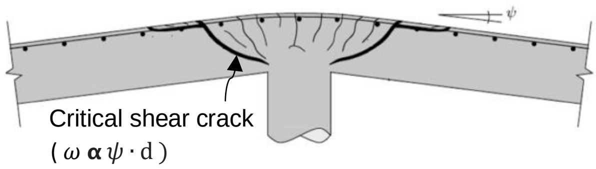

Figure 2 Assumed slab kinematics with opening of critical shear crack [7]

Based on this approach, Muttoni proposed a failure criterion [6]:

$$\frac{V_R}{b_0 \cdot d_v \cdot \sqrt{f_c}} = \frac{3/4}{1 + 15 \frac{\psi \cdot d}{d_{g0} + d_g}} \qquad (5)$$

Where, V_R is the shear strength; b_0 is a control perimeter; d_v is the effective depth of the slab; f_c is the compressive strength of concrete; the term $d_{g0} + d_g$ represents the roughness of the lips of the cracks, which depends on the maximum size of the aggregate.

The punching shear strength of a slab without shear reinforcement can be directly calculated using the CSCT failure criterion (equation 5).

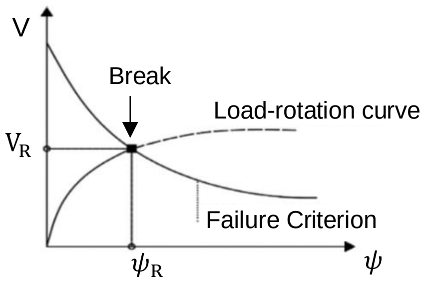

Figure 3 Calculation of punching shear strength for members without shear reinforcement [6]

To do so, the intersection between the failure criterion and the actual behavior of the slab, characterized by its load-rotation curve, must be calculated [2]:

$$\psi = 1{,}5\frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{sd}}{m_{rd}}\right)^{1{,}2} \qquad (6)$$

Where, r_s refers to the distance from the column axis to the line of contraflexure of radial bending moments, m_{sd} is the average moment per unit length for calculation of the flexural reinforcement in the support strip; m_{rd} is the average flexural strength per unit length in the support strip; E_s is the value of the modulus of elasticity of steel.

The *fib* Model Code 2010 [2] is a code-like formulation of CSCT, where the punching shear strength of a flat two-way slab is assumed to be the summation of the resistance provided by concrete and the resistance provided by shear reinforcement (equation 7).

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \geq V_{Ed} \qquad (7)$$

$$V_{Rd,c} = k_\psi \cdot \frac{\sqrt{f_{ck}}}{\gamma_c} \cdot b_0 \cdot d_v \qquad (8)$$

$$k_\psi = \frac{1}{1{,}5 + 0{,}9 k_{dg} \cdot \psi \cdot d} \leq 0{,}6 \qquad (9)$$

Where, $k_{dg} = 32/(16 + d_g) \geq 0{,}75$.

The nominal shear strength provided by shear reinforcement is taken as:

$$V_{Rd,s} = \sum A_{sw} \cdot k_e \cdot \sigma_{swd} \qquad (10)$$

Where, k_e is the coefficient of eccentricity; σ_{swd} refers to the stress that is activated in the shear reinforcement.

3. DEFORMATION OF FLAT SLABS SUBJECT TO CYCLIC ACTION

In the FLAT project developed by António Ramos et al. (2014) [8], 6 slabs subjected to cyclical lateral actions were analyzed. The objective of these tests was to study the maximum drift capacity of each slab when subjected to the seismic actions and, consequently, the evaluation of energy dissipation based on the hysteretic properties of the slab-column connection.

Through experimental tests, Ramos et al. proves that in models without punching shear reinforcement, the drift capacity and the stiffness of the connection are inversely proportional to the gravitational load applied. Thus, the rotation capacity and, consequently, the slab-column connection ductility decreases, while gravitational charge increases. In the models with punching shear reinforcement, there was an increase in the capacity of resistance to the horizontal loads, leading to an increase of the capacity of dissipation of energy. In this way, it is proven that the reinforcement of the slabs with punching reinforcements provides a bearing capacity of higher drifts, compared to a slab without reinforcement (Figure 4).

This has already been proven by Megally and Ghali in 2000 [9], who concluded that the inclusion of punching reinforcement significantly increases the ductility of the slab-column connection.

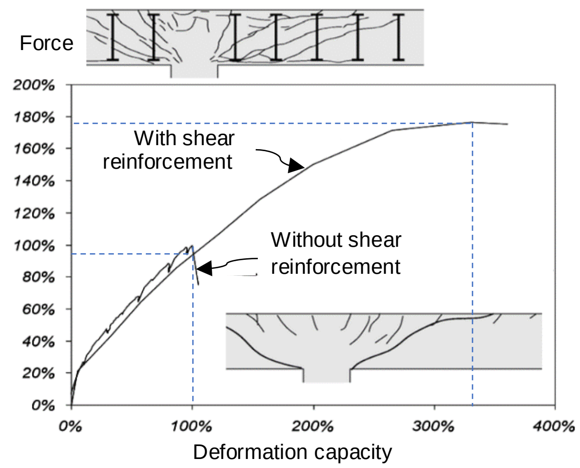

Figure 4 Comparison of the behavior and strength of two slabs with and without shear reinforcement [10]

4. DIMENSIONING OF THE SLAB-COLUMN CONNECTIONS SUBJECT TO SEISMIC ACTION

4.1 Ductility Requirement – ACI318-14

Pan and Moehle (1989) found that the gravity shear ratio between gravitational loads and the shear strength provided by concrete for monotonic loads (V_g/V_C) had an influence on punching shear breaking. Through experimental tests, these authors found that the level of gravity load carried by the slab is a primary variable affecting the apparent lateral ductility. As a result of these tests, it was noticed that the gravity shear ratio should be kept below 0,4, to ensure some minimal ductility, and for values of V_g/V_C exceeding approximately 0,4, there is no lateral displacement ductility. In addition, Pan and Moehle have concluded that for high gravitational loads there is a decrease in drift capacity [11], [12], [13], [14].

Thus, ACI 318-14 [15], which corroborates the ideologies derived from the studies developed by these authors, develops an empirical formulation (equation 11) that relates the drift capacity that the structure is subjected, during the seismic action, to the ratio V_g/V_C. The application of this criterion is done for $\psi_2 \cdot Q_k + G_k$ of the seismic combination.

$$\frac{\Delta_x}{h_{sx}} \geq 0{,}035 - \left(\frac{1}{20}\right) \cdot \left(\frac{V_g}{V_c}\right), mas \ \frac{\Delta_x}{h_{sx}} \geq 0{,}005 \quad (11)$$

This criterion analyzes the ductility of the structure and also the ductility of the slab-column connections and indicates the need to use punching shear reinforcement.

When the drift to which the structure is subjected (Δ_x/h_{sx}) during cyclic lateral loading exceeds the maximum capacity presented in the criterion of equation 11, the slab-column connection must be reshaped so that its ductility improves. To do this, the dimensions of the column and/or slab must be altered or reinforce the structure in order to reduce displacements between floors or to introduce punching shear reinforcement. The latter suggestion of change is the most reliable since the use of punching shear reinforcement increases the ductility of the column-slab connections. This ductility requirement, expressed in equation 11, is plotted in Figure 5.

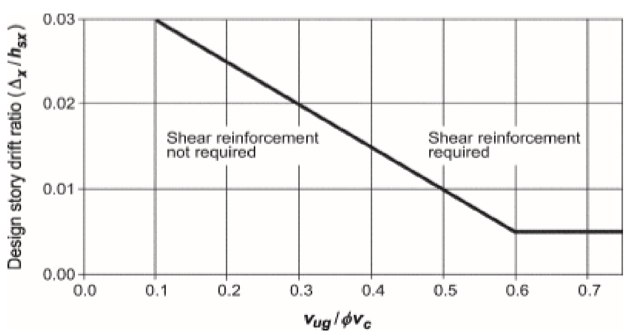

Figure 5 Illustration of the criterion of equation 11 [15]

4.2 Damage Limitation - Eurocode 8-1

The EC8 [1] admits that non-structural elements can suffer some damage without the operationality of the structure being compromised. This requirement is ensured on EC 8 by limiting the relative displacement between floors, giving minimum stiffness to the structure.

For buildings having non-structural elements of brittle materials attached to the structure, the interstorey drift is limited to:

$$\Delta_x \cdot \nu = 0{,}005 \cdot h \qquad (12)$$

The damage limitation expressed in EC8-1 can be considered as parameter to indirectly measure the ductility of the slab-column connection in accordance with the ductility requirement in the american regulation, since both methodologies have limitations for the relative displacement between floors.

4.3 Displacement imposed limitation – Eurocode 8-3

In order to complement the ductility verification in the slab-column connections, another approach can be considered in EC8-3 [16], where the capacity of the structural elements to withstand the earthquake is evaluated through an analysis of the displacements imposed on the structure.

$$\delta_{imposed}{}^{maximum} \leq \delta_{supported}{}^{maximum} \qquad (13)$$

Where, $\delta_{imposed}{}^{maximum}$ is the displacement imposed by the seismic action on the structure or structural element; $\delta_{supported}{}^{máximo}$ is the maximum displacement supported by the structure or structural element.

This methodology is used to analyse the need of confinement reinforcement in columns.

5. CASES STUDIES DESCRIPTION

The analyzed structures are located in Lisbon, belonging to the seismic zone 1.3 and a type B foundation soil. The gravity loads considered are 3 kN/m^2 for dead and live loads, on all the floors. The structures of the various cases studies have 6 raised floors and a symmetrical geometry with 30mx30m, composed by flat slabs with a thickness of 0,20m. The general characteristics of the analyzing structures are summarized in Figure 6 and Figure 7.

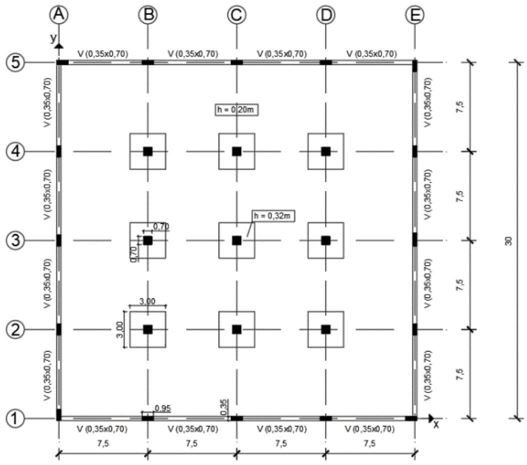

Figure 6 Plan dimensions of the analyzing of the first case

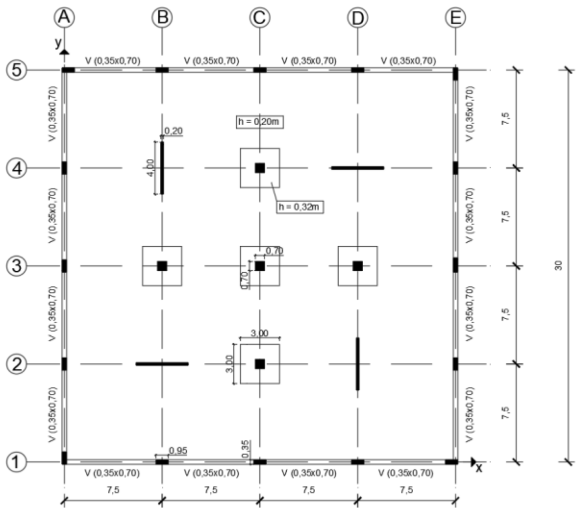

Figure 7 Plan dimensions of the analyzing of the second and third cases

The behavior coefficient adopted was 3,9 and the slab flexural reinforcement on the capitals is ϕ12//0,1+ ϕ12//0,1 for all the cases studies.

5.1 Classification of internal columns

To classify the inner pillars as primary or secondary seismic elements, the EC8-1 defines an analysis based on the contribution of these elements to the lateral stiffness of the structure through the percentage of basal shear effort.

Table 1 Contribution of the columns for lateral stiffness

Model	Peripheral Columns	Walls	Internal Columns	Δ
A	61,53%	---------	38,47%	62,53%
B	49,43%	32,80%	17,77%	21,61%
C	35,03%	52,45%	12,52%	14,32%
D	25,36%	65,48%	9,16%	9,92%

Analyzing Table 1, it can be seen that Models B, C and D, the inner pillars can be classified as secondary elements since their contribution to the lateral rigidity of the structure does not reach 15% of the contribution of the primary seismic elements, despite in model B, the limit imposed is exceeded by a very small margin.

5.2 Damage Limitation

The analysis shows that the damage control requirements stipulated in EC8 are guaranteed loosely for all floors and for the two directions of action of the seismic action, since all models have maximum deformations between floors below 0,5%. As shown in Figure 8, model A is the one with the highest drift, as would be expected, since it is the model that has the lowest stiffness and as such, will be submitted to greater lateral displacements.

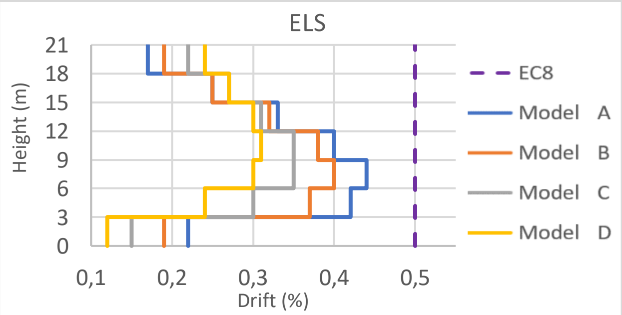

Figure 8 Drift on height for all cases of study

5.3 Case Study 1 – Model A

5.3.1 Resistance Analysis

In Table 2 are represented the shear strength values in the slab-column connection for the fundamental and seismic combinations following the y axis, in order to verify the punching shear resistance according to EC2-1 and MC2010.

Table 2 Design shear and shear strength according do EC2 and MC2010

		EC2-1	MC2010	
Combination	V_{Ed} (kN)	$V_{Rd,c}$ (kN)	ψ	$V_{Rd,c}$ (kN)
Fundamental	977,9	1174,1	0,0121	974,1
Seismic y	586,7	1174,1	0,0144	705,5

Observing the results presented in the Table 2, the punching shear resistance is verified, except for the fundamental combination according to the MC2010, creating the necessity to adopt punching reinforcement.

5.3.2 Ductility analysis

The american regulation (ACI318-14) approaches the resistance to punching by ductility through a dimensioning criterion given by equation 11. In this way, it becomes necessary to verify if the drift which the structure is subjected respects this condition.

The results obtained by applying EC2-1 and MC2010 are shown in Table 3 and plotted in Figure 9.

Table 3 Design drift and gravity shear ratio for model A

	$EC2-1$	$MC\,2010$
V_g (kN)	575,3	547,8
$V_{Rd,c}$ (kN)	931,9	899,2
$V_g/V_{Rd,c}$	0,62	0,64
$(\Delta_x/h_{sx})_{máx}$	0,0041	0,0030
Δ_x/h_{sx}	0,0110	0,0110

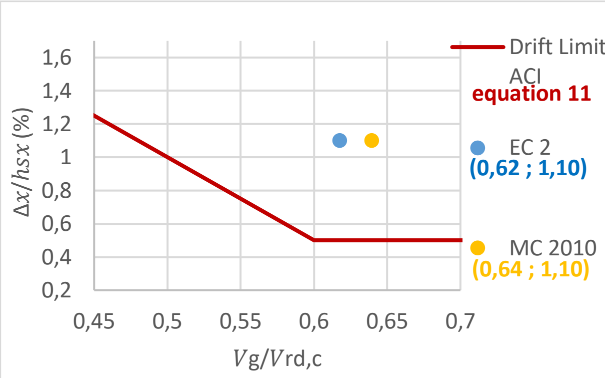

Figure 9 Illustration of ductility requirement for model A

Figure 9 shows that the ductility requirement proposed by ACI 318-14 is not checked, applying EC2 and MC2010, so it will be necessary to adopt punching reinforcement.

In order to complement the ductility verification in the slab-column connection, the philosophy of EC8-3 can be adopted to provide enough ductility to the connection to withstand displacements imposed by the earthquake. Thus, the concrete was confined in order to increase the ductility of the connection.

Since the bending moments on the column, for seismic action, are greatly reduced, a reinforcement ratio of 1,1% was adopted. Table 4 summarizes the values of the effective critical strain $(\varepsilon_{cu,cf})$ and the effective lateral compressive stress at the ULS due to confinement $(\boldsymbol{\sigma_2})$, for the seismic combination.

Table 4 Effective critical strains and effective lateral compressive stress due to confinement

Strains due to confinement reinforcement					
A_{sw}	w_{wd}	σ_2 (MPa)	$fck_{,ef}$ (MPa)	$\varepsilon_{cc,cf}$ (‰)	$\varepsilon_{cu,cf}$ (‰)
ϕ8//0,10	0,1572	1,60	38,00	3,21	14,16

Calculated the strain of the concrete, the steel strain and the critical curvature of the section were determined (Table 5).

Table 5 Steel strain and critical curvature

N (kN)	$\varepsilon_{cu,cf}$ (‰)	ε_s (‰)	$(1/R)_u$ (cm^{-1})
4327	14,16	33,91	7,17x10^{-4}

Once the curvature value was obtained, the equation 13 was verified, considering the maximum relative displacements between floors.

$$\delta_{imposed} = \Delta_x \leq \delta_{supported} = \left(\frac{1}{R}\right)_u \cdot c_{column} \cdot h_{sx}$$

$$\delta_{imposed} = \Delta_x \leq \delta_{supported} = 7{,}17\text{x}10^{-4} \cdot 70 \cdot 300$$

$$\delta_{imposed} = \Delta_x = 3{,}30cm \leq \delta_{supported} = 15{,}06cm$$

By analyzing the obtained results, it is concluded that the percentage of confinement reinforcement adopted will provide enough ductility to the slab-column connection.

5.4 Case Study 2 – Model D

Since in the case of a previous study, the ductility requirement is not satisfied, by application of both regulations, the slab-column connection must be reshaped in order to improve its ductility. For this purpose, interior walls have been introduced, giving a greater stiffness to the structure and, consequently, decreasing the displacements between floors.

5.4.1 Resistance Analysis

As can be seen from the results presented in the Table 6, the punching shear resistance condition for this model is checked for all combinations, according to both regulations. Therefore, according to a resistance analysis it would not be necessary to adopt punching reinforcement for this case of study.

Table 6 Design shear and shear strength according do EC2 and MC2010

		EC2-1	MC2010	
Combination	V_{Ed} (kN)	$V_{Rd,c}$ (kN)	ψ	$V_{Rd,c}$ (kN)
Fundamental	960,3	1174,1	0,0119	1025,4
Seismic y	569,4	1174,1	0,0134	691,8

5.4.2 Ductility analysis

The structural performance used to define the behavior of the structure in ductility is done by limiting the displacement imposed according to the american regulation (equation 11) and the european regulation (equation 13).

Following the ACI ductility requirement, by application of the MC 2010, it is noticeable that it is not necessary to use punching reinforcement in the slab, even though it exceeded the limit imposed by a very small margin. Which is no longer the case with EC2 as the Figure 10 shows, formulation that maintains the shear strength value constant and increases the design shear.

Table 7 Design drift and gravity shear ratio for model D

	$EC2-1$	$MC\ 2010$
$V_g\ (kN)$	548,5	547,0
$V_{Rd,c}\ (kN)$	931,9	988,4
$V_g/V_{Rd,c}$	0,59	0,55
$(\Delta_x/h_{sx})_{máx}$	0,0056	0,0073
Δ_x/h_{sx}	0,0079	0,0079

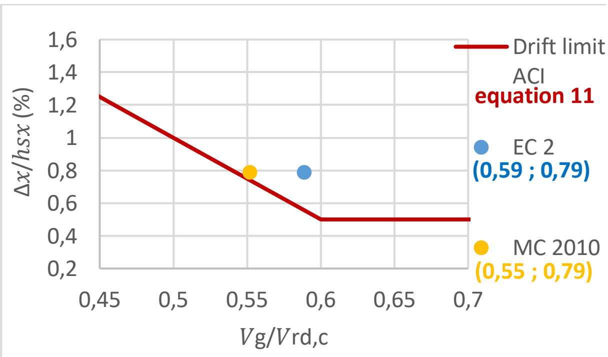

Figure 10 Illustration of ductility requirement for model D

Thus, this model shows, compared to the previous one, a greater capacity of control of the deformations induced in the slabs indicating that, at least in terms of resistance, it would not be necessary to adopt transverse reinforcement.

Although the results obtained, by applying the ductility requirement of the american regulation, exempt the adoption of transverse reinforcement, it is recalled that Pan and Moehle found that the connection loses ductility as $V_g/V_{Rd,c}$ approaches 0,4, so it becomes necessary to provide ductility to the slab-column connection. Thus, the concrete was confined in order to increase the ductility of the connection, and to check its plastic rotation capacity.

Since this column has been characterized as a secondary seismic element, on section 5.1, the EC8 does not stipulate a minimum of reinforcement for this type of seismic elements. Due to this a 0,8% reinforcement percentage was adopted, close to the minimum limit of 1% stipulated in EC8 for primary seismic elements, since secondary seismic elements should only be dimensioned to resist gravitational loads. Table 8 summarizes the values of the effective critical strain and the effective lateral compressive stress at the ULS due to confinement.

Table 8 Effective critical strains and effective lateral compressive stress due to confinement

Strains due to confinement reinforcement					
A_{sw}	w_{wd}	σ_2 (MPa)	$fck_{,ef}$ (MPa)	$\varepsilon_{cc,cf}$ (‰)	$\varepsilon_{cu,cf}$ (‰)
ϕ8//0,10	0,1245	1,27	36,33	2,93	11,95

Continuing the analysis, the steel strain and the critical curvature of the section were determined.

Table 9 Steel strain and critical curvature

N (kN)	$\varepsilon_{cu,cf}$ (‰)	ε_s (‰)	$(1/R)_u\ (cm^{-1})$
4047	11,95	30,35	6,31x10^{-4}

$$\delta_{imposed} = \Delta_x = 2{,}36cm \leq \delta_{supported} = 13{,}25cm$$

It is verified that the amount of confinement reinforcement introduced to the column confers enough ductility to the slab-column connection to support, loosely, the deformations imposed by the earthquake.

5.5 Case Study 3 – Model D1

The simplification present in EC8-1, where a 50% loss of stiffness in all elements of the structure is assumed, can result in an excessively conservative design because it underestimates the imposed displacements, which is crucial for the verification of the service limit states.

According to recent studies [17], [18], it was verified that the stiffness to be adopted in each type of structural element may be different since the loss of stiffness by cracking of each structural element, during the seismic action, depends on the axial compression forces which it is subjected and the reinforcement adopted. Thus, since the walls are structural members subjected to large moments and reduced axial effects, their loss of stiffness due to cracking can be bigger.

Following this methodology, a model with the same geometric characteristics as model D was developed, only reducing the uncracked stiffness of the walls from $0{,}5E_cI_c$ to $0{,}25E_cI_c$.

5.5.1 Damage Limitation

With the reduction of the non-cracked stiffness of the section it is expected that the displacements between floors will increase but will be limited in order to verify the condition imposed on the EC8-1.

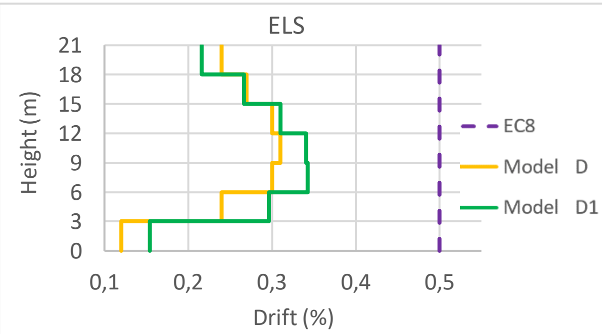

Figure 11 Comparison of drift on height between model D and model D1

Figure 11 shows that the reduction of wall stiffness actually led to an increase in the displacements between floors, but remained below the service limit, with the condition of 0,5% being loosely verified.

5.5.2 Resistance Analysis

In addition, reducing non-cracked stiffness will cause a redistribution of strengths by causing an increasing strength in the inner columns and at the same time strengths on the walls will decrease. Despite the increase of the strength in the inner columns, the results obtained in the analysis of the resistance, for this case study, do not present significant alterations, compared with the ones of the model D. This occurred because the increase of the strength in the peripheral columns was much more accentuated, compared to the increased strength in the inner columns.

5.5.3 Ductility analysis

By applying the ductility criterion of the ACI, it is verified that the $V_g/V_{Rd,c}$ relation takes the same value as the previous study case because the difference of the strengths in this model is not very significant, compared to the model D.

However, once it occurred an increase in the relative displacements due to the reduction of the stiffness of the walls, the ductility criterion would indicate, in this case, the need to adopt transverse reinforcement in the slab, in order to provide enough ductility to the slab-column connection, such as Figure 12 indicates.

This conclusion was expected since the $V_g/V_{Rd,c}$ ratio exceeds the limit of 0,4 defined by Pan and Moehle, as shown on section 4.1.

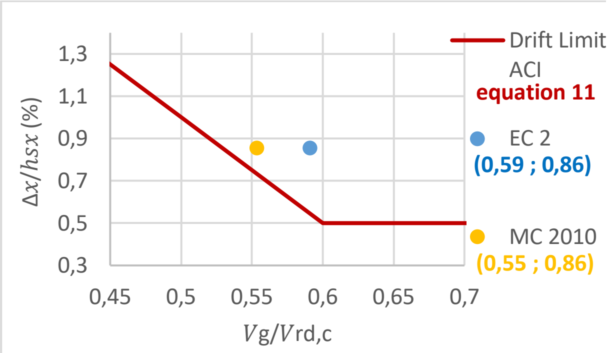

Figure 12 Illustration of ductility requirement for model D1

Since, in this model, there was an increase in the relative displacements, it becomes appropriate to verify if the condition of equation 13 is respected. In order to compare the results obtained in this model with the previous study case, the same reinforcement and the same confinement reinforcement were adopted in the column.

Considering the effective critical strain of the concrete, the steel strain and the critical curvature of the section were determined.

Table 10 Steel strain and critical curvature

N (kN)	$\varepsilon_{cu,cf}$ (‰)	ε_s (‰)	$(1/R)_u\ (cm^{-1})$
4333	11,95	23,16	5,90x10^{-4}

Through the curvature of this case study, the equation 13 was verified, taking into account the maximum relative displacement between floors.

$$\delta_{imposed} = \Delta_x = 2{,}57cm \leq \delta_{supported} = 12{,}39cm$$

Once again, it is concluded that the amount of confinement reinforcement implemented in the column gives enough ductility to the slab-column connection in order to support, loosely, the deformations imposed by the earthquake.

6. Conclusions

As the results obtained in the case studies demonstrate, the slab's rotation capacity (i.e., the ductility of the slab-column connection) decreases with the increase of gravitational loads. The same happens with drift capacity which has an inverse proportionality to the gravitational loads, as Ramos et al. had proven through their experimental tests on section 3.

As mentioned on section 4.1, there are several ways of improving ductility behavior to the slab-column connection.

- In model D, it is verified that the inclusion of the walls gives a high importance in the lateral stiffness of the building, reducing the drifts and, consequently, attenuating the ductility requirement of the slab-column connection. Thus, this model demonstrates the capacity to control the deformations in the slabs, is an efficient way to guarantee an acceptable seismic behavior.
- Other very effective and important way of increasing ductility to the slab-column connection is based on the introduction of transverse punching reinforcement that must be properly detailed.
- Adopting the confinement reinforcement on the column, is an important extra guarantee of providing capacity of supporting higher drifts, compared to a slab without reinforcement (Figure 4).

The ACI ductility design approach considers the contribution of all the structural elements in the consideration of the lateral stiffness of the structure. This hypothesis is questionable as it considers the columns connected to flat slabs as effective for seismic resistance.

On the other hand, EC8-1 dismisses these columns for seismic resistance but suggests that they should remain elastic (non-yielded) which leads to over resistant columns leading to high moment transmission in the connection, unfavorable for slab punching behavior.

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